Composite Floors
A Theoretical Research into the Design of Steel-Concrete Composite Floors with a Bigger Unpropped Span of 7.2 m

J. van Blokland
COMPOSITE FLOORS

A THEORETICAL RESEARCH INTO THE DESIGN OF STEEL-CONCRETE COMPOSITE FLOORS WITH A BIGGER UNPROPPED SPAN OF 7.2 M

by

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Preface

Steel-concrete composite floors are applied in different types of multi-storey buildings, high-rise, and renovation projects. However, the application of this floor system is limited in the Netherlands, because the unpropped span capacity is relatively small compared to other competitive floor systems. This thesis is focused on finding a possible design for a steel-concrete composite floor that has a bigger span than available on the present day Dutch building market.

The thesis is written for the Faculty of Civil Engineering and Geosciences of the Delft University of Technology, as final challenge to prove worth the Master's degree in Structural Engineering. The research was supported by Dutch Engineering Raadgevend Ingenieursbureau B.V., which delivers different types of steel-concrete composite floors for the Dutch building market. In addition, the thesis is written for the personal interest of R. Stark, director of Imd Raadgevende Ingenieurs, which designs and develops load-bearing structures for all type of Dutch building projects. R. Stark is expert in steel-concrete composite structures, and keen advocate of the application of steel-concrete composite structures in buildings.

The design of the new floor is introduced as the 'JorFlor'. Lightweight fill elements form the main design principle of the floor. These elements increase the unpropped span capacity through a reduction of the self-weight of the floor. Furthermore, a steel deck design is made to resist shear, a new type of connector is developed to provide steel-concrete composite action, and a second load-carrying mechanism is analysed to achieve adequate performance during fire.

The committee that supervised my graduation project is formed by Professor ir. F.S.K. Bijlaard, ir. R. Abspoel and ir. S. Pasterkamp from the TU Delft, ir. H. Prins director of Dutch Engineering Raadgevend Ingenieursbureau B.V., and ing. R. Stark director of Imd Raadgevende Ingenieurs.

I would like to thank all committee members for their guidance, expertise, and support of my research. My sincere thanks goes to R. Stark for my position as intern and H. Prins for his effort in organizing two educative fun trips. Finally, I would like to thank Saar for the fun, laughter, love and good times, throughout the writing of my thesis.

J. van Blokland
Delft, September 2015
In the Netherlands buildings are designed using a grid with multiples of 3.6 meter. Within these designs floor spans of 7.2 meter are popular (double grid size). The deep decks of composite floors at this moment in time are designed to reach an unpropped span of 5.5 meter. This master research is focused on finding a possible deck design for a steel-concrete composite floor slab that can span 7.2 m and that is constructed without the need of temporary supports. This resulted in the JorFlor, a light-weight, big span steel-concrete composite floor that can compete with current floor systems.

Important aspects taken into account in the new design are the efficient installation of the decks, a low self-weight, and a 90 minutes fire resistance of the floor. This study is aimed at a steel thickness of 1.0 to 1.25 mm and a minimum amount of steel and construction height. The load carrying capacity of the floor is maximized, and a reduction of the self-weight is accomplished by lightweight fill elements. The shape of the deck is trapezoidal, with wide ribs and a thin concrete top slab.

Three critical aspects were studied to determine if the selected design is feasible: (1) the shear resistance of the steel web, (2) the shear connection, and (3) the resistance during fire. Transverse web stiffeners with a height of 150 mm, width of 17 mm, depth of 3 mm, and a c.t.c. spacing of 220 mm provide sufficient shear resistance. The top flanges of the decks are connected with a new design of shear connectors. 18 connectors per half-span are needed on the top flange of each deck panel to resist the maximum bending moment of 63 kNm/m. During fire the loads in the floor are carried by a concrete compression arch and a tensile tie. The ribs near the supports are fully cast to anchor the tension reinforcement, providing shear resistance, and to transfer the compression force to the support. A ø16 mm and ø20 mm bar are required for 60 minutes and 90 minutes fire resistance, respectively.

The total height of the JorFlor is 280 mm and the floor weighs only 300 kg per square meter. The deck panels are 7.2 m long, 220 mm high, 1.20 mm thick, 300 mm wide and weigh 50 kg. A first cast in the ribs is required to encase the bottom reinforcement. After that, fill elements are positioned in the ribs and the top reinforcement is installed. Different fill elements are suitable: PIR foam, rock wool, resol foam and foam glass. The floor is finished with a second cast. A prefabricated variant of the Jorflor is possible, where the concrete and tension reinforcement in the bottom of the ribs are precast. The load-carrying capacity of the floor slab during service life is determined with a model comparable to a concrete slab and the deck as a downstand truss beam.

The JorFlor meets the design specifications as formulated in the first part of the research. It can therefore be said that the design is successful. Based on a theoretical study it can be concluded that it is possible to design a steel deck for a steel-concrete composite floor slab, which can span 7.2 meter and can be constructed without the need of temporary supports. The JorFlor knows two practical applications. First of all, the ‘in-situ’ JorFlor can be applied in special circumstances where a big unpropped span is required, but prefabricated elements cannot be installed. Second of all, the ‘prefab’ JorFlor suits the current Dutch building practice. The installations process is simple and the construction speed is high. The installation of the decks requires light hoisting equipment at site. As a last note, it is recommended to conduct a cost analysis, whether the JorFlor is cheaper than other competitive floor systems.
5.1.7 Concept 7 .................................................. 33
5.1.8 Concept 8 .................................................. 33
5.1.9 Concept 9 .................................................. 34
5.2 Conclusion .................................................. 34
6 Steel-concrete composite floor with ribs 35
6.1 Design assumptions ........................................... 35
6.2 Deck design .................................................. 36
   6.2.1 Step 1: rough deck design ............................... 36
   6.2.2 Step 2: trapezoidal deck design ......................... 37
   6.2.3 Step 3: design of stiffeners and embossments .......... 39
   6.2.4 Step 4: redesign of the decks ......................... 39
   6.2.5 Conclusion ............................................... 41
6.3 Installation with a double cast .................................. 41
   6.3.1 Double cast ............................................ 41
   6.3.2 First cast up to centroid of deck ....................... 42
   6.3.3 First cast up to height of deck ......................... 43
   6.3.4 Conclusion ............................................... 43
6.4 Installation with a fill element .................................. 43
   6.4.1 Fill element ............................................ 43
   6.4.2 Concrete part ........................................... 44
   6.4.3 Double cast with fill element .......................... 45
   6.4.4 Conclusion ............................................... 46
6.5 Deck design for a double cast with fill element ............... 46
   6.5.1 Design concept ......................................... 46
   6.5.2 Critical aspects .......................................... 47
   6.5.3 Deck design ............................................. 48
6.6 Conclusion .................................................. 49
Part 3 51
7 Design of the web to resist shear 53
7.1 Tension field method .......................................... 53
   7.1.1 Model ................................................... 53
   7.1.2 Loads .................................................... 54
   7.1.3 Conclusion ............................................... 55
7.2 Tension field and capacity of the tensile tie ................. 56
7.3 Transverse stiffener and capacity of the compression strut ... 57
   7.3.1 Model ................................................... 57
   7.3.2 Section properties ...................................... 57
   7.3.3 Differential equation .................................. 60
   7.3.4 Euler buckling load .................................... 62
   7.3.5 Design buckling load .................................. 63
   7.3.6 Design of the web ...................................... 65
   7.3.7 Translational- and rotational stiffness ................. 66
   7.3.8 Conclusion ............................................... 68
7.4 Formation of the tension field ................................ 68
7.5 Conclusion .................................................. 69
8 Shear connection .................................................. 71
8.1 Shear connectors ............................................... 71
   8.1.1 Design of the shear connector ......................... 71
   8.1.2 Loads .................................................... 72
   8.1.3 Design shear resistance ................................ 73
   8.1.4 Rigid or flexible ........................................ 73
   8.1.5 Conclusion ............................................... 74
8.2 Partial shear connection ....................................... 76
   8.2.1 Internal stress distribution ............................. 76
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.2.2</td>
<td>Longitudinal shear forces</td>
<td>78</td>
</tr>
<tr>
<td>8.2.3</td>
<td>Design of connectors over the span</td>
<td>80</td>
</tr>
<tr>
<td>8.2.4</td>
<td>Conclusion</td>
<td>81</td>
</tr>
<tr>
<td>8.3</td>
<td>Moment capacity</td>
<td>81</td>
</tr>
<tr>
<td>8.3.1</td>
<td>Elastic moment resistance</td>
<td>81</td>
</tr>
<tr>
<td>8.3.2</td>
<td>Plastic moment resistance</td>
<td>82</td>
</tr>
<tr>
<td>8.3.3</td>
<td>Conclusion</td>
<td>82</td>
</tr>
<tr>
<td>8.4</td>
<td>Alternative shear connections</td>
<td>83</td>
</tr>
<tr>
<td>8.5</td>
<td>Design improvements for the connectors</td>
<td>83</td>
</tr>
<tr>
<td>8.6</td>
<td>Conclusion</td>
<td>85</td>
</tr>
<tr>
<td>9</td>
<td>Situation during fire</td>
<td>87</td>
</tr>
<tr>
<td>9.1</td>
<td>Floor slab during fire</td>
<td>87</td>
</tr>
<tr>
<td>9.1.1</td>
<td>Load-carrying model</td>
<td>88</td>
</tr>
<tr>
<td>9.1.2</td>
<td>Loads</td>
<td>88</td>
</tr>
<tr>
<td>9.1.3</td>
<td>Failure modes</td>
<td>90</td>
</tr>
<tr>
<td>9.2</td>
<td>Reduced material properties</td>
<td>90</td>
</tr>
<tr>
<td>9.2.1</td>
<td>Concrete slab</td>
<td>90</td>
</tr>
<tr>
<td>9.2.2</td>
<td>Tension reinforcement</td>
<td>90</td>
</tr>
<tr>
<td>9.3</td>
<td>Moment capacity</td>
<td>91</td>
</tr>
<tr>
<td>9.4</td>
<td>Shear capacity</td>
<td>92</td>
</tr>
<tr>
<td>9.5</td>
<td>Buckling of the concrete slab</td>
<td>93</td>
</tr>
<tr>
<td>9.5.1</td>
<td>Non-combustible fill element</td>
<td>94</td>
</tr>
<tr>
<td>9.5.2</td>
<td>Design of the concrete top flange</td>
<td>94</td>
</tr>
<tr>
<td>9.5.3</td>
<td>Conclusion</td>
<td>100</td>
</tr>
<tr>
<td>9.6</td>
<td>Anchorage of the tension reinforcement</td>
<td>100</td>
</tr>
<tr>
<td>9.6.1</td>
<td>The required anchorage length</td>
<td>100</td>
</tr>
<tr>
<td>9.6.2</td>
<td>Anchorage with a bend at the end of the bar</td>
<td>102</td>
</tr>
<tr>
<td>9.6.3</td>
<td>Design and installation</td>
<td>103</td>
</tr>
<tr>
<td>9.7</td>
<td>Conclusion</td>
<td>104</td>
</tr>
<tr>
<td>10</td>
<td>Design of the steel deck</td>
<td>105</td>
</tr>
<tr>
<td>10.1</td>
<td>Dimensions of the deck</td>
<td>105</td>
</tr>
<tr>
<td>10.2</td>
<td>Cross-sectional properties</td>
<td>106</td>
</tr>
<tr>
<td>10.3</td>
<td>Loads during construction</td>
<td>107</td>
</tr>
<tr>
<td>10.3.1</td>
<td>Self-weight</td>
<td>107</td>
</tr>
<tr>
<td>10.3.2</td>
<td>Loads</td>
<td>109</td>
</tr>
<tr>
<td>10.3.3</td>
<td>Deflection and ponding</td>
<td>110</td>
</tr>
<tr>
<td>10.4</td>
<td>Effective cross-sectional properties</td>
<td>111</td>
</tr>
<tr>
<td>10.5</td>
<td>Design verifications</td>
<td>111</td>
</tr>
<tr>
<td>10.5.1</td>
<td>Serviceability limit state</td>
<td>111</td>
</tr>
<tr>
<td>10.5.2</td>
<td>Ultimate limit state</td>
<td>112</td>
</tr>
<tr>
<td>10.6</td>
<td>Deck design</td>
<td>112</td>
</tr>
<tr>
<td>10.6.1</td>
<td>Structural optimization</td>
<td>112</td>
</tr>
<tr>
<td>10.6.2</td>
<td>Self-weight of the floor slab</td>
<td>114</td>
</tr>
<tr>
<td>10.6.3</td>
<td>Steel-concrete composite floor</td>
<td>114</td>
</tr>
<tr>
<td>10.6.4</td>
<td>Steel deck design</td>
<td>115</td>
</tr>
<tr>
<td>10.7</td>
<td>Conclusion</td>
<td>115</td>
</tr>
<tr>
<td>11</td>
<td>Steel-concrete composite floor</td>
<td>119</td>
</tr>
<tr>
<td>11.1</td>
<td>Load-carrying behaviour</td>
<td>119</td>
</tr>
<tr>
<td>11.2</td>
<td>Installation</td>
<td>121</td>
</tr>
<tr>
<td>11.2.1</td>
<td>In-situ</td>
<td>121</td>
</tr>
<tr>
<td>11.2.2</td>
<td>Prefabrication</td>
<td>121</td>
</tr>
<tr>
<td>11.3</td>
<td>Hogging moment</td>
<td>121</td>
</tr>
<tr>
<td>11.4</td>
<td>Local load</td>
<td>124</td>
</tr>
<tr>
<td>11.5</td>
<td>Deflections</td>
<td>125</td>
</tr>
<tr>
<td>11.6</td>
<td>Vibrations</td>
<td>128</td>
</tr>
</tbody>
</table>
H  Steel-concrete composite floor 209
   H.1  Shear capacity of the concrete cross-section 209
FINDING YOUR WAY IN THIS MASTER THESIS

The research thesis that you are about to read has the following structure.

Firstly, chapter 1 gives an introduction to the research. Here the thesis subject is explained, a problem statement given, and the research questions with corresponding research methodology are formulated.

The rest of the report is subdivided into four different parts, which corresponds to the different phases as formulated under 'research methodology' in chapter 1.

Phase one consists of chapter 2 and 3. Chapter 2 gives a literature review on the steel-concrete composite floor. The study takes a closer look at the technical aspects of the steel deck, and the steel-concrete composite floor slab in order to formulate the design specifications. These specifications will later on in the trajectory be used to steer the research design. Additionally, the review is used to visualize the verifications methods used and applicable by the Eurocode. In chapter 3 the design is specified by stating the design criteria and the boundary conditions, based on the information found during the literature study.

Phase two consists of chapter 4, 5 and 6. Chapter 4 provides a preliminary analysis on the designs possible success by investigating the load-carrying abilities and the deflection characteristics of the steel deck during construction. Chapter 5 focuses on the conceptual development of the steel deck design with a 'bigger' unpropped span. Based on a feasibility study using non-mathematical criteria is determined which design concepts are useful for further development. In chapter 6 is elaborated on these design concepts and a decisive design concept is chosen.

Phase three consists of chapter 7 till 11. Chapter 6 gave a decisive design concept, which in phase three will be further elaborated. In chapters 7, 8 and 9 a feasibility study is made of this particular concept, using the three most critical aspects: the shear resistance of the steel deck, the steel-concrete composite connection, and the design during fire. Chapter 10 will give a closer look into the design of the steel deck, and chapter 11 will provide the remaining checks according to Eurocode that are interesting to determine.

Phase four consists of chapter 12 and gives the conclusions and recommendations found during this thesis research.
LIST OF FIGURES

1.1 Overview of properties per floor system (span, height, and weight) ........................................ 1
1.2 Two basic concepts of composite slab systems ................................................................. 2

2.1 Roll forming, and folding and press braking [1, p.9-11] ......................................................... 10
2.2 Modification of the stress-strain curve of the steel due to cold-forming .................................. 11
2.3 Moment-curvature diagram if failure due to buckling ............................................................. 12
2.4 The behaviour of steel deck in the construction stage ............................................................. 14
2.5 Elastic- and plastic stress distribution ...................................................................................... 15
2.6 Composite floor slabs using different types of profiled sheets [2, p.792] .................................... 17
2.7 Location of critical sections in a simply supported composite slab [3, p.131] ............................ 17

4.1 Bending moment resistance of the deck ................................................................................... 26
4.2 Loads in the deck during construction ..................................................................................... 27

5.1 Alternative designs for steel-concrete composite floor slabs [4, p.28] ....................................... 29
5.2 Steel beams and composite floor manufactured in one rolling process ................................. 30
5.3 Sandwich panel with composite floor as top skin ................................................................. 30
5.4 Deep deck with steel beams in ribs ....................................................................................... 31
5.5 Upscale of current deep deck: steel-concrete composite floor with ribs ................................... 31
5.6 Alternative deck shapes: inverted U-profiles (left), Z-profiles, and box-profiles (right) ........... 32
5.7 Cold-formed sections as downstand beam: U-decks (left) and CFS (right) ............................... 32
5.8 Steel-concrete composite slab with weight reduction .............................................................. 33
5.9 Alternately trapezium deck for steel-concrete composite slab ............................................... 33
5.10 Steel-concrete composite slab installed with a double cast .................................................. 34

6.1 Two design concepts: small ribs and wide ribs with fill element ........................................... 35
6.2 Requirements for the concrete cover ...................................................................................... 36
6.3 Method to estimate the amount of effective steel ................................................................. 37
6.4 Stacking of current deep decks: Comflor 225 (l) and Comflor 210 (r) ..................................... 38
6.5 Assumptions for the dimensions of a trapezoidal deck .......................................................... 38
6.6 Effective cross-section and design of flange- and web stiffeners ......................................... 39
6.7 Two design concepts: composite slabs (left) and single deck elements (right) ....................... 41
6.8 Installation with double cast: up to centroid (left) and up to top of deck (right) ..................... 42
6.9 Principle of floor systems with weight reducing elements ..................................................... 44
6.10 Casting of concrete with fill element ..................................................................................... 45
6.11 Position of the reinforcement: at the support (left) and in the cross-section (right) ............... 45
6.12 Position of the fill element in the steel-concrete composite slab .......................................... 46
6.13 Impression of the installation process ................................................................................... 46
6.14 Design concept of a steel-concrete composite floor slab with a double cast and fill element .... 47
6.15 Design deck for the floor slab with weight reduction ............................................................. 48

7.1 Tension field method to determine the post-buckling strength of the web of the deck ............ 53
7.2 Cross-section of the embossments used in current deep decks (Comflor 210 and 225) ......... 54
7.3 View and cross-sections of the web ....................................................................................... 54
7.4 Longitudinal- and vertical shear forces .................................................................................. 55
7.5 Vertical- and longitudinal shear forces in the web of the deck near the support .................... 56
7.6 Tension field in web of deck .................................................................................................. 56
7.7 Tension field according to Basler (left) and capacity of the tensile tie (right) ......................... 57
7.8 Stiffener modelled as small buckling column (left) and interaction between local- and overall buckling failure (right) ............................................................... 58

xv
LIST OF TABLES

1.1 Advantages of ASB combined with deep decks (Slimdek) [4, p.35] ........................................ 3
2.1 Properties and advantages of steel-concrete composite floor slab during construction .......... 13
2.2 Section properties of deep decks: Comflor 210 and Comflor 225 [15, p.108] ...................... 16
2.3 Properties and advantages of steel-concrete composite floor slab during service life ............. 20
3.1 Specifications for the design .......................................................... 21
4.1 Required bending stiffness and strength for a range of bigger spans .................................. 27
6.1 Two design concepts: cross-sectional properties and deflections during construction .......... 40
6.2 Variant study for the two designs compared with existing deep decks .............................. 40
7.1 Effect of the translational stiffness $K_{1,2}$ on the Euler buckling resistance $N_{cr}$ .............. 67
7.2 Effect of rotational stiffness $C_{Q,1,2}$ on the buckling resistance $N_{cr}$ ............................... 68
8.1 Characteristic value of the longitudinal shear strength of the Comflor 210 floor slab [16, p.7] .... 73
9.1 Required reinforcement for fire resistance R60 ad R90 ....................................................... 92
9.2 Self-weight of the fill element ......................................................................................... 94
10.1 Section properties, required buckling strength, and required length of the web embossment for different sheet thicknesses $t$ ........................................................................... 115
12.1 Most important specifications for the design ................................................................. 133
12.2 Properties of the JorFlor compared with existing deep decks ........................................... 137
A.1 Construction loads (characteristic values) [17] ............................................................. 141
A.2 Material properties [6, 18] ............................................................................................. 142
B.1 Field of application of the EC4-2 method to determine the fire resistance of unprotected composite slabs .................................................................................................................. 154
B.2 Properties of steel deck design and composite floor slab ............................................... 160
B.3 Verification criteria for the steel deck during the construction stage in SLS and ULS ........ 160
B.4 Verification criteria for the composite slab during service life in SLS and ULS ................. 160
B.5 Variant study of two types of deck designs ...................................................................... 161
B.6 Different variants of design concept 1: small ribs .......................................................... 162
B.7 Different variants of design concept 2: wide ribs with fill element ..................................... 163
B.8 Effective section properties of design with small ribs if no stiffeners are designed ............ 167
B.9 Effective section properties of design with small ribs if only two symmetric flange stiffeners are designed .................................................................................................................. 171
B.10 Effective steel area of design with small ribs with two symmetric flange stiffener and a single web stiffener ........................................................................................................... 174
B.11 Effective steel area of design concept 1 ......................................................................... 175
B.12 Effective steel area of design concept 2 ......................................................................... 175
B.13 Variant study for the two designs compared with existing deep decks ........................... 176
LIST OF NOTATIONS

TERMS AND DEFINITIONS

Construction loads
additional load during the construction stage to take account for working personnel, small equipment and heaps of concrete

Construction stage
design situation during the installation of the steel-concrete composite floor slab, when the deck functions as workfloor, and shuttering for the wet concrete

Design thickness of the deck (t₀)
the thickness of the deck reduced with 20 µm on both sides to take account for the sink layer applied to provide a anti-corrosive layer

Effective cross-section (Aₑ)
the cross-section is reduced, to take account for local buckling and the embossed parts

Embossment
indentations, in the web- or flange of the deck, to ensure steel-concrete composite action

Eurocode
European norms and guidelines for the building practice

Fill element
lightweight elements in the floor to reduce the amount of concrete, and therefore the self-weight of the floor slab

Fire design
design situation of the composite floor slab in case of fire

Floor system
the system of structural- and non-structural elements that together make up the floor, e.g. secondary beams, subfloor, finishing, etc.

Load case
load arrangement with permanent- and variable actions for a particular design situation

Model or structural model
often a simplification of the structure or a part of the structure, used for the purpose of analysis, design and verifications

Permanent actions (gₚ)
or dead loads, are static forces that are relatively constant over time, and include the self-weight of the floor slab

Resistance
capacity of the deck, composite slab, or a member of the slab or deck, to withstand a certain load case without mechanical failure, e.g. bending resistance or buckling resistance

Service life
the design situation that is relevant during the design working life of the composite floor slab

Service limit state (SLS)
state where the loads are un-factored to check service requirements for the floor or a part of the floor, e.g. deflections or vibrations
Shear connector
a connector that resists horizontal shear between the steel deck and the concrete, e.g. dowels, embossments, or re-entrant profiles

Steel-concrete composite floor slab
steel decking and cast in-situ concrete connected together using shear connectors such that a composite floor slab is formed

Steel deck or decking
thin-walled profiled sheeting used to construct a steel-concrete composite floor slab

Stiffeners
in the longitudinal or transverse direction of the span, used to improve the buckling resistance of the deck and applied in forms of bends, folds, angles or indentations

Ultimate limit state (ULS)
state where loads are factored to check different forms of structural failure of the deck or the composite floor slab

Unity check (UC)
used as verification for the strength and stiffness of a structure or a part of the structure, where the UC should be less than 1.0

Unpropped construction
temporary supports or props are not required during the construction of a steel-concrete composite floor slab

Variable actions ($q_k$)
or live- or imposed loads, are temporary loads that can move and be of short duration, e.g. people that use a building

ABBREVIATIONS

c.t.c. centre to centre
n.a. neutral axis
e.n.a. elastic neutral axis
p.n.a. plastic neutral axis
ASB asymmetric steel beam
BC boundary conditions
CF210 Comflor 210
CF225 Comflor 225
CL centre line
DE differential equation
EC Eurocode
EI bending stiffness ($E \cdot I$)
EPS expanded polystyrene
FEM finite element model
MC matching conditions
SLS service limit state
ULS ultimate limit state
UC unity check
SYMBOLS

Latin upper case letters

\( A_c \) cross-section area of concrete
\( A_p \) gross cross-section area of the deck (or \( A_{gr} \))
\( A_{pe} \) effective cross-section area of the deck
\( A_{red} \) cross-section area of the fill element
\( A_s \) cross-section area of reinforcement steel
\( A_{s,red} \) effective cross-section area of a stiffener (or \( A_{eff} \))
\( C_0 \) rotation spring stiffness
\( E \) modules of elasticity
\( E_{cm} \) mean modules of elasticity of concrete
\( E_s \) modules of elasticity of steel
\( F_{bt} \) tension force in reinforcement at anchorage
\( G \) self-weight of the composite floor slab
\( G_c \) weight of the concrete
\( G_{deck} \) weight of a single deck panel
\( G_p \) weight of the deck per square meter floor area
\( G_{ponding} \) weight of the accumulated concrete
\( G_s \) weight of the reinforcement in the steel-concrete composite floor
\( I \) second moment of area
\( I_{c,0} \) time independent second moment of area of the cracked cross-section of the composite slab
\( I_{c,e} \) time dependent second moment of area of the cracked cross-section of the composite slab
\( I_{eff} \) second moment of area of the effective area of the deck
\( I_s \) second moment of area of the reinforcing steel
\( K \) translation spring stiffness
\( L \) span; length
\( L_{red} \) length of the fill element over the span
\( L_x \) length of the fully cast ribs near the supports
\( M_{Ed} \) design bending moment
\( M_{el,Rd} \) elastic bending moment resistance
\( M_{pl,Rd} \) plastic bending moment resistance
\( M_{Rd} \) bending moment resistance
\( N_{b,Rd} \) design buckling resistance
\( N_c \) compression force in the concrete
\( N_{cr} \) Euler buckling resistance (or \( F_{cr} \))
\( N_{Ed} \) design buckling load (or \( F \))
\( N_p \) tension force in the steel deck
\( N_s \)  
tension force in the reinforcement

\( P_{Rd} \)  
design longitudinal shear resistance of a connector

\( Q_d \)  
design value of a concentrated load

\( R_{60} \)  
design fire resistance of 60 minutes

\( R_{90} \)  
design fire resistance of 90 minutes

\( S_z \)  
vertical component of normal- and shear force

\( V_c \)  
volume of concrete

\( V_{Ed} \)  
design vertical shear load

\( V_{l,Ed} \)  
design longitudinal shear force

\( V_{l,Rd} \)  
design longitudinal shear resistance

\( V_{Rd} \)  
design vertical shear resistance

\( W \)  
section modulus

\( W_{eff} \)  
effective section modulus

**Latin lower case letters**

\( b_0 \)  
width of the top flange of the deck

\( b_d \)  
width of a single deck panel

\( b_{eff} \)  
effective width

\( b_{om} \)  
average width of the concrete in a rib

\( b_u \)  
width of the bottom flange of the deck

\( b_w \)  
smallest width of the concrete cross-section

\( c \)  
cement cover

\( d \)  
effective depth

\( e_c \)  
distance from the n.a. of the deck to the top flange

\( e_t \)  
distance from the n.a. of the deck to the bottom flange

\( f_{bd} \)  
design value of the ultimate bond strength

\( f_{cd} \)  
design strength of concrete

\( f_{ctd} \)  
design tension strength of concrete

\( f_e \)  
resistance to vibrations

\( f_y \)  
yield strength

\( f_{yb} \)  
yield strength of the deck

\( g \)  
gravitational acceleration

\( g_k \)  
characteristic value of the permanent loads

\( h \)  
height of the floor slab

\( h_c \)  
height of the concrete above the deck

\( h_p \)  
height of the deck

\( l \)  
liter; length

\( l_0 \)  
system length of an element

\( l_{bd} \)  
required design anchorage length
**List of Notations**

- $l_{cr}$  buckling length of an element
- $n_p$ amount of ribs per meter width
- $q_d$  design value of the uniform distributed load
- $q_k$  characteristic value of the variable loads
- $r$  corner radius
- $s_w$  slant height
- $t$  thickness of the deck
- $t_0$  design thickness of the deck
- $t_i$  fire resistance in minutes
- $u$  unit load
- $v$  angle of the web of the deck (or $\alpha$)
- $w$  deflection
- $w_{addl}$  additional deflection of the composite slab during service life
- $w_{tot}$  total deflection of the composite slab during service life
- $x_c$  height of the concrete compression zone
- $x_{el}$  height of the compression zone
- $z$  internal leverarm

**Greek upper case letters**

- $\Psi_2$  factor for quasi-permanent value of a variable action

**Greek lower case letters**

- $\alpha$  angle of the web of the deck; factor for appropriate buckling curve
- $\gamma_c$  partial factor for concrete
- $\gamma_g$  partial factor for permanent actions
- $\gamma_M$  partial factor for a material property
- $\gamma_q$  partial factor for variable actions
- $\gamma_{Vs}$  partial factor for shear resistance
- $\delta_0$  deflection of the deck during construction
- $\delta_{s,max}$  maximum deck deflection during construction
- $\epsilon$  strains
- $\theta$  temperature
- $\kappa$  reduction factor for buckling
- $\lambda$  non-dimensional slenderness
- $\nu$  Poisson’s ratio in the elastic range
- $\rho$  density of a material
- $\sigma$  stress
- $\tau_u$  longitudinal shear strength of a composite slab
- $\varphi$  rotation
This chapter gives a brief introduction to the subject of this thesis. The main objective is defined and corresponding research questions are formulated. This thesis is structured in four parts, where the first three parts each provide an answer to one of the three research questions. Together, the findings and conclusions of these three parts give an answer to the main objective of this thesis in the last part.

1.1. AN INTRODUCTION TO FLOOR SYSTEMS

A floor system is defined as the system of structural and non-structural elements that together make up the floor, e.g. secondary beams, subfloor, finishing, etc. [19, p.128-152]. The different types of floor systems that dominate the building industry at the moment are classified according to three main groups:

- the steel floor systems (floor systems mainly constructed of steel);
- the concrete floor systems (floor systems mainly constructed of concrete);
- the steel-concrete composite floor systems (floor systems constructed from steel deck profiles where concrete is cast on top; a distinction can be made between low- and high steel deck profiles).

The properties of these different groups are depicted in figure 1.1, where the maximum span, the floor height, and the self-weight are compared. Within this thesis the focus is set on the steel-concrete composite floor system.

![Figure 1.1: Overview of properties per floor system (span, height, and weight)](image)
1.2. THE UNPROPPED FLOOR SYSTEM

The steel-concrete composite floor system is constructed either propped or unpropped. A propped installation requires temporary support when the concrete is poured. These temporary supports are needed until the concrete has hardened and the floor has a self-carrying ability. In figure 1.1 the unpropped span is indicated with a colored bar, the propped span capacity is indicated with an outline, and the span capacity of a low profile decking combined with a composite beam (downstand beam) with a dashed line.

The unpropped floor is favorable during construction, because the installation is less complicated and the building speed increases. At this moment in time the prefab Hollow Core Slabs (HCS) are a popular product in the Dutch building industry. This floor has a big unpropped span capacity and a high load-capacity. However, when weight plays an important role, the building project is complex, or it beholds a renovation project the steel-concrete composite floor may be more beneficial.

At the moment the unpropped span capacity of a steel-concrete composite floor is relatively low. This means that more cross beams and joists are required to install the floor. Progression can be made developing a steel-concrete floor with a bigger unpropped span.

1.3. INTRODUCING THE STEEL-CONCRETE COMPOSITE FLOOR SYSTEM

Mullett [15] identifies two concepts for the composite floor systems:

- Profiled decking / in-situ concrete floor combined with a downstand beam (1.2a);
- Slimdeck: Profiled deep decking / in-situ concrete combined with an asymmetric beam (1.2b).

Stark and Schuurman [20] present an interesting discussion on the application of composite floor systems, using some project examples developed in the Netherlands. Each project tells why the choice for a composite floor system was beneficial. The literature shows that there are different types of profiled steel sheets for composite floor systems available [8]. Low steel profiles with a height of 16 to 100 mm and high steel profiles – better known as deep decks – with a height of 200 mm. Figure 1.2a gives an example of the possible application of low steel profiles. These types of steel profiles are often applied in combination with a downstand beam, providing a large span capacity. Openings in the web of the beam allow that building services can be installed direct underneath the floor minimizing the total floor height of this system.

Figure 1.2b is an example of the application of deep decks. In this situation the decking is combined with an integrated asymmetric beam (ASB). The fire performance is better than with a downstand beam, and building services can be installed underneath the floor without additional measurements [3, 15]. The total floor height of this system is then comparable with low steel profiles on a downstand beam. Table 1.1 gives an overview of the advantages of this system by Brekelmans.

This form of construction have led to a huge increase in steel framed buildings. The range of application is wide, and includes (1) multi-storey buildings (2) renovation projects (3) car parks (4) warehouses (5) storage buildings (6) housing and (7) community service buildings [3, p.122]. The shear connection between the steel and the concrete ensures the composite behavior. In this situation the concrete is efficiently under compression and the steel efficiently under tension. The floor system uses both materials to their advantage. The
1.4. PROBLEM STATEMENT

As presented in the previous text, the steel-concrete composite floor is not a competing floor system on the Dutch building market at this moment in time. The floor system is not yet available with an unpropped span of 7.2 meter. No optimal design for the steel deck is yet available to create the unpropped span during construction.

1.5. RESEARCH METHODOLOGY

For this research an approach was maintained, which is subdivided into four different phases.
PHASE 1 - INTRODUCING THE RESEARCH

The first part provides a literature study on the current state of knowledge regarding the steel-concrete composite floors. This review is used to clearly formulate the design criteria and boundary conditions necessary for the research design. The latter formed the second part of the literature review.

During phase 1 the steel deck and the steel-concrete composite floor slab are examined to gain understanding of the structural behavior. The base material- and the manufacturing process of the steel decking are analysed, because they influence the material properties.

The Eurocode prescribes that testing is required to determine the steel-concrete composite behavior, when actual implementation is desired. In this research the steel-composite behavior is not verified in such manner. However, the steel-concrete composite behavior can be estimated by analogy or simple calculations. The research will be concluded by giving requirements regarding the testing procedures. The design of shear connectors, which are necessary to ensure the composite behavior of the steel-concrete interface are incorporated in the deck design.

PHASE 2 - SPECIFYING THE DESIGN

In part 2 the feasibility of this research is verified and design concepts are introduced.

The required load-carrying capacity of the deck is determined. A comparison with current decks is made, to indicate if an unpropped span of 7.2 meter is theoretically possible.

Additionally, different design concepts with bigger spans are studied and evaluated using the design criteria formulated in phase 1. In different steps with an increasing level of detail the designs are elaborated. Design rules and methods provided by the Eurocode are used to verify the results. The most feasible design is chosen to be fully developed.

Note: the deck deflections during construction are verified taking the construction loads into account. These loads are not obligatory, but are applied in practice for safety reasons. In phase 3 the deflections during construction are verified without taking these construction loads into account.

PHASE 3 - THE PRODUCT

The third part of the thesis is the design phase. During this phase the design of a steel deck for a steel-concrete composite floor system with an unpropped span of 7.2 meter is elaborated. The information found during the second phase is used as a starting point and to demarcate the boundaries.

PHASE 4 - WHAT HAVE WE LEARNED?

In the last phase the conclusions and recommendations found during the research process are given.

1.6. MAIN OBJECTIVE

This master research is focused on finding a possible design for a steel-concrete composite floor that has a bigger span than available on the present day Dutch building market. As stated under “research methodology”, the thesis contains four different phases. For each phase an objective is formulated, and can be found in this paragraph.

Phase 1: The research objective regarding the literature study:

To identify the general technical specifications of the steel-concrete composite floor, which can be used to formulate the design concepts and develop the design meeting the main research objective.

Phase 2: The research objective regarding the design concepts:

To perform a feasibility study based on different design concepts using criteria found in the literature study.

Phase 3: The research objective regarding the design:
1.7. The research questions

To theoretically develop the most feasible concept, and mathematically substantiate whether an unpropped span of 7.2 meter can be accomplished.

Phase 4: Main research objective:

To theoretically analyse whether it is structurally and practically possible to design a steel deck for a steel-concrete composite floor slab that can span 7.2 meter, and can be constructed without the need of temporary supports.

1.7. The research questions

Maintaining the same structure as for the main objectives, the following research questions are formulated.

Phase 1:

What is the current technical and practical knowledge on steel-concrete composite floors, and which criteria, boundary conditions and design requirements can be formulated for the design of a 7.2 meter unpropped steel-concrete composite floor?

Phase 2:

What are the most critical design criteria, and how can they be combined into a feasible concept?

Phase 3:

Is it theoretically and mathematically possible to design a steel deck for a steel-concrete composite floor that can span 7.2 meter without the need of propping?

Phase 4: Main research question:

Is it structurally and practically possible to design a steel deck for a steel-concrete composite floor slab that can span 7.2 meter and be constructed without the need of temporary support?
Part 1

Introducing the Research
This literature review is meant to provide an overview of current knowledge on steel decks and steel-concrete composite floor slabs that is required to set boundary conditions and criteria for the design. The first section of this chapter elaborates on the steel deck and the second on the steel-concrete composite slab. The most important verification criteria are given with reference to the different parts of the Eurocode.

2.1. The Steel Deck

Cold-formed members are produced from coated or uncoated hot-rolled or cold-rolled flat strips or coils. Cold-formed members are produced in two main types: individual structural framing members, and panels and decks. The individual structural framing members are made from cold-formed sections. Panels and decks are made from profiled sheets and linear trays. The stiffness of both cold-formed sections and sheeting is increased with edge and intermediate stiffeners [1].

Advantages cold-formed steel in the building construction:

- compacted packing and shipping;
- economic production of unusual configurations and favourable strength-to-weight ratios;
- panels and decks can be used for floor construction;
- panels and decks are not only able to withstand loads normal to their surface, but also resist force in their own planes.

Other advantages compared with other materials are:

- lightness;
- high strength and stiffness;
- ability to provide long spans, up to 12 m;
- ease of prefabriation and mass production;
- fast and easy installation;
- substantial elimination of delays due to weather;
- more accurate detailing;
- non-shrinking and non-creeping at ambient temperatures;
- formwork unnecessary;
- termite-proof and rot-proof;
- uniform quality;
• economy in transportation and handling;
• non-combustibility;
• recyclable material.

The steel deck is a type of cold-formed steel member.

This section examines different aspects of the steel deck: the material properties, the installation process, the load-carrying capacity, design verifications, and current decking.

2.1.1. MATERIAL PROPERTIES

The steel decking is produced by the cold forming process of thin steel sheets. Three parameters of the base material are important for the design of the steel decking: the steel grade, the width of the coil, and the thickness of the sheet. Different steel grades are available on the market. A steel grade of S350 with a design yield strength of 350 Mpa is common practice, but this grade can easily go up to a steel grade of S550 with a design yield strength of 550 Mpa. The width of the coil comes in a range from 1.0 to 1.25 m, but this can be increased up to 1.4 m. The thickness of the decking is usually between 0.75 and 1.25 mm, but a thickness of 1.5 or 2 mm is possible. The sheets are galvanized with a 275 g/m² sink layer on each side that correspond with a thickness of 20 µm on both sides.

MANUFACTURING PROCESS

In the manufacturing process the thin steel sheets are formed in cold-state (i.e. without applying any heat) into the desired shape. The manufacturing process of thin walled cold-formed steel has influence on the structural behaviour of the deck. First thin steel sheets are rolled out from the coils and cut into the appropriate width. Then two types of cold-forming processes are available that can be used for the manufacturing of the decks [1, p.9].

• roll forming (figure 2.1a);
• folding and press braking (figures 2.1b and 2.1c).

Roll forming is a manufacturing process where a continues steel strip is fed through a series of opposing rolls. Each pair of opposing rolls is called a stage. In each stage the steel plastically is deformed to form the desired shape. The amount of stages depends on the complexity of the desired cross-sectional shape. A complex shape requires more rolling stages. Roll forming is suitable for a large volume of long products.

Folding is used to produce members of a short length with a simple geometry by folding a series of bends. This process is rather simple, but has limited applications. Press braking is used more often. The member is formed by pressing a steel strip of a certain length between shaped dies. Each press results in a bend. The disadvantage of this process is the limitation of the section length that is usually 5 m. This process is suitable for a small volume of short products with a relative simple shape.

![Figure 2.1: Roll forming, and folding and press braking](1, p.9-11)
2.1. The Steel Deck

The manufacturing process of cold-formed members leads to some specific characteristics [21, p.5-6].
- modification of the stress-strain curve of the steel (figure 2.2);
- residual stresses.

The manufacturing process leads to an increase of the yield strength and sometimes of the ultimate strength. This is especially noticed in the corners where the steel is deformed plastically. In the rolling process this effect is also appreciable in the flanges. The yield strength increases due to strain hardening and depends on the type of steel used for cold rolling. The ultimate strength increases due to strain aging and depends on the metallurgical properties of the material. Strain aging is accompanied by a decrease of the ductility.

Residual stresses can also be introduced during the cold-forming process by varying stretching forces. These residual stresses can change the load-bearing resistance of a cold-formed section. Residual stresses are favourable if introduced in parts of the cross-section that are both in compression and susceptible to local buckling.

\[ M_e = W_e f_y \]
\[ M_p = W_p f_y \]

\( W_e \) is the elastic section modulus;
\( W_p \) is the plastic section modulus.

A strength increase of a cross-section while keeping the amount of material constant is possible if also the slenderness increases. The slenderness becomes greater for a bigger profile height where the web and flanges are thinner. Slender structures have the disadvantage of being sensitive to buckling. The slenderness of a cross section is characterised by the cross-section class. The classification is based on the least favorable width-thickness ratio of an element under bending and/or compression.
Cross-section class 1 (curve 1) has a low slenderness and buckling will occur after a certain amount of plastic deformation. Cross-section class 4 (curve 4 in figure 2.3) has a high slenderness and buckling will occur before the elastic moment capacity \( M_e \) is reached. Class 2 and 3 are in-between. It is only allowed to determine the strength of cross-section classes 2, 3 and 4 with the elastic approach, where only the extreme fibers yield.

Steel decks are usually classified as cross-section class 4 and at most class 3. The elastic capacity is reduced for this type of cross-sections and therefore also the ductility. The Eurocode provides an 'effective width method'. Here the reduced elastic capacity is determined with a reduced cross-section taking the effect of local buckling into account.

**Steel grade**

The strength is also influenced by the choice of steel grade. A higher steel grade means a higher yield strength, and both \( M_e \) and \( M_p \) are linear dependent on the yield strength. However, steel with a higher strength is more sensitive for buckling than steel with a low strength. Eurocode 3 incorporates this effect with the factor \( \epsilon \):

\[
\epsilon = \sqrt{\frac{235}{f_y}}
\]

The width-thickness ratio to determine the cross-section class is reduced with this factor \( \epsilon \). This means that a cross-section with a higher steel strength should have a bigger width-thickness ratio to have the same buckling resistance as a cross-section with a low steel strength [22].

The bending stiffness \((EI)\) is not influenced by the steel grade. The modulus of elasticity \(E\) is independent of the steel grade and thus the bending stiffness is also independent of the steel grade. It implies that the application of a higher steel grade in general can lead to larger deflections. This is illustrated with the following. For a single span deck with cross-section class 3 or 4 holds:

\[
M_{Ed} = \frac{1}{8} qL^2 \leq M_e = W_e \cdot f_y \quad \Rightarrow \quad q \leq \frac{8W_e f_y}{L^2}
\]

\[
w = \frac{5}{384} \frac{qL^4}{ET}
\]

From expression 2.1 follows that the choice for a higher steel grade will increase the bending strength \((M_e)\), which allows for a similar cross-section a higher value of the load per unit length \((q)\). However, a higher \(q\) will lead to a bigger deflection \((w)\), because the bending stiffness does not depend on the steel grade. This means that the deflections are often governing if higher steel strengths are applied.
2.1.2. INSTALLATION

Most advantages of the steel-concrete composite floors arise during the installation process that is very efficient. One truckload can transport a significant amount of floor deck surface. At site is limited space required and a package of multiple deck panels (one package covers 100-300 m² floor area) is hoisted in position. Once hoisted in position the panels are placed manual resulting in a quick erection. The panels form a working platform and provide a permanent shuttering for the wet concrete. The individual panels are connected with self-drilling screws or pop nails, increasing the composite behaviour of the floor slab and the overall strength and stiffness. Finally the building services are installed and the floor is finished with a ceiling and perhaps a finishing layer. Note that if the individual deck panels are placed manual the self-weight should not exceed 50 kg. Advantages during construction are summarised in table 2.1.

<table>
<thead>
<tr>
<th>Property:</th>
<th>Advantage:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lightweight stackable elements</td>
<td>• multiple lightweight deck panels in one compact package</td>
</tr>
<tr>
<td></td>
<td>• effective transport and stacking</td>
</tr>
<tr>
<td></td>
<td>• limited space required at site</td>
</tr>
<tr>
<td></td>
<td>• quick and simple installation</td>
</tr>
<tr>
<td></td>
<td>• limited required hoisting capacity (if concrete is pumped)</td>
</tr>
<tr>
<td></td>
<td>• decks panels can be placed manual</td>
</tr>
<tr>
<td></td>
<td>• simple connection system</td>
</tr>
<tr>
<td>Deck as shuttering and reinforcement</td>
<td>• reduction of required the installation time</td>
</tr>
<tr>
<td>Deck as shuttering and work-floor</td>
<td>• safe work-floor once decks are installed</td>
</tr>
<tr>
<td>Deck as structural element</td>
<td>• prevent leaking of cement water</td>
</tr>
<tr>
<td>Thin steel sheets</td>
<td>• deck prevent lateral torsional buckling of the steel beams</td>
</tr>
<tr>
<td></td>
<td>• diaphragm to transfer horizontal loads during construction</td>
</tr>
<tr>
<td></td>
<td>• edges and openings can be adapted at site</td>
</tr>
</tbody>
</table>

Table 2.1: Properties and advantages of steel-concrete composite floor slab during construction

2.1.3. DESIGN VERIFICATIONS OF THE STEEL DECK

There are many types of steel sheetings each designed for different functional requirements and load conditions. The design is usually based on experimental investigations. Analytical design methods are only valid for trapezoidal shaped decking. Developments led to new profile types with intermediate stiffeners, higher strength steel and favourable geometrical properties, all to increase the bending resistance of the decking. These first developments where all based on experience and ’design by testing’, but eventually let to design formula to predict the load bearing resistance of trapezoidal sheeting [21]. If testing is required, for optimization or if analytical methods are insufficient, the following can be used:

- single span beam test: bending- resistance and stiffness;
- intermediate support test or two-span beam test: combined bending and shear or crippling resistance;
- end support test: shear resistance at end support;
- walkability test: resistance to concentrated loads.

If the deck is designed according to calculation the following should be verified:

- bending resistance;
- shear resistance;
- concentrated load resistance (crippling resistance);
- interaction of bending and shear or crippling;
- stiffness of the decking.

In this study the strength and stiffness criteria are verified according to calculation.
LOADS DURING CONSTRUCTION

Actions on the steel deck as shuttering include:

- weight of the fresh concrete and the steel deck;
- construction loads (only included for ultimate limit state verifications);
- ponding effect.

The construction loads are determined according to the Eurocode 1 part 6 [17, p.22-23]. An overview is given in table A.1 of the appendix. The ponding effect represents the load increment that occurs due to the accumulation of the fresh concrete in the center of a steel deck plate that deflects under the weight of the cast concrete. This effect should be taken into account according to Eurocode 4 if \( \delta > \frac{1}{10} \cdot h \). Where \( \delta \) is the deflection of the steel sheeting calculated in serviceability limit state and \( h \) is the total slab depth.

LOAD-CARRYING CAPACITY OF THE DECK

Wright et al. [23] have carried out over 200 tests and studied different parts of the steel-concrete construction systems in different load cases. The construction or wet stage is often the critical loading for the design of a steel-concrete composite slab, where the deck acts as shuttering for the wet concrete.

The load-deflection curve of the steel deck is linear until the steel buckles. This buckling occurs in the compression zone as illustrated in figure 2.4a and leads to non-linear behaviour. However, the initial buckling behaviour is linear elastic until a certain extent. This means that the ultimate strength capacity of the steel decking is greater than the strength at the first buckle. Many steel decks are designed including this behaviour [23].

The verification of the steel sheets as shuttering is according to Eurocode 3, where design rules for cold formed sheeting are given [5, 13]. The strength validation is straightforward, but criteria regarding the required stiffness and allowed deflections are less unambiguous.

DEFLECTION

Limited deflections of the floor during construction are necessary to limit the total deflection of the finished floor and restrain the effect of ponding during construction. The definition for the maximum deflection during construction is given by the Eurocode 4 as \( L/180 \), where \( L \) is the span [12, p.100]. According to this design rule a midspan deflection of 40 mm is allowed for a span of 7.2 m. The Steel Deck Institute (SDI) specifies that the calculated deflections are limited by the smallest value of \( L/180 \) or 19 mm [25, p.8]. Varma and Huber [26] mention that this limitation is designed for decks that span up to 3 to 3.6 meter and not for decks that span up to 9 meter.

EFFECT OF BUCKLING

As mentioned before, steel decks are usually classified as cross-section class 4. The thin-walled steel is susceptible for local buckling. This effect is taken into account using a reduced cross-section, better know as

![Typical buckling failure](image1.png)

![Effective section of the Comflor 225 deck](image2.png)

Figure 2.4: The behaviour of steel deck in the construction stage
the ‘effective cross-section’ \((A_{pe})\). The effective cross-section is determined with the ‘effective width method’ according to Eurocode 3 part 3 and 5.

Longitudinal stiffeners in the deck prevent local bucking and therefore contribute to the overall bending stiffness. There are multiple types of longitudinal stiffeners: folds and bends, folded- and curved grooves, or a bolted angle stiffeners \([5, \ p.10]\). The corners of a cross-section function naturally as longitudinal stiffeners.

Transverse stiffeners ensure composite action between the concrete and the steel, and prevent buckling in the transverse direction of the span. In addition, provide transverse web stiffeners the shear resistance of the deck. Transverse stiffeners are often called embossments. In the study of Bode and Däuwel \([24]\) is shown that the embossments are compressed or elongated when the deck is in bending. Therefore is assumed that the embossed area does not contribute to the overall bending stiffness and this area is neglected when calculating the effective cross-section (figure 2.4b (right) and 2.5).

**Moment resistance**

The elastic- and plastic moment resistance is determined according to the stress distributions illustrated in figure 2.5. Where e.n.a. is the elastic neutral axis and p.n.a. the plastic neutral axis.

![Figure 2.5: Elastic- and plastic stress distribution](image)

The elastic neutral axis or centre of gravity is defined as the axis about which: \( A_{pe,\text{top}} \cdot e_c = A_{pe,\text{bot}} \cdot e_t \). The elastic moment capacity is determined according:

\[
M_{el,Rd} = \frac{I_{eff} f_{yb}}{z}
\]  

(2.2)

Where:

- \( I_{eff} \) is the effective second moment of area of \( A_{pe} \) with respect to the resistance to vertical deflections;
- \( f_{yb} \) is the design yield strength of the deck;
- \( z \) is the distance from the e.n.a. to the extreme fiber.

The elastic moment resistance is used to obtain the bending resistance of the deck for stiffness (SLS) verifications.

The plastic neutral axis for a cross-section corresponds to the axis about which the total area is equally divided: \( A_1 = A_2 = A/2 \). The plastic moment capacity is determined according:

\[
M_{pl,Rd} = \frac{1}{2} A_{pe} f_{yb} (\bar{e}_c + \bar{e}_t)
\]

(2.3)

Where:

- \( A_{pe} \) is the effective area of the cross-section;
- \( \bar{e}_c \) is the distance from the p.n.a. to the centre of the top part of the cross-section;
$\bar{e}_t$ is the distance from the p.n.a. to the centre of the bottom part of the cross-section. The plastic moment resistance is used to obtain the bending strength of the deck for strength (ULS) verifications.

**Design thickness**

The design thickness of the steel sheet $t_0$ is equal to the thickness of the steel sheet $t$ reduced with the sink layer that is applied to provide a anti-corrosive layer. As mentioned before, the sink layer is equal to 20 µm on both sides.

$t_0 = t - 0.04$ [mm]

### 2.1.4. CURRENT DECKING

At this moment have deep decks a span capacity up to 5.5 m without the use of temporary supports. The Comflor 210 (CF210) and Comflor 225 (CF225) are two of these decks with a trapezoidal decking profile and a deck height of 210 and 225 mm, respectively. The composite action is ensured with the use of embossments along the web and top flange, and in case of the CF225 also with re-entrant profile on the top flange (figure 2.4b). The geometry of the CF225 is similar to the CF210, but the bending resistance is bigger. This is largely due to the re-entrant profile on the top flange [15].

The properties of current deep decks used for steel-concrete composite flooring are given in table 2.2.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>CF210</th>
<th>CF225</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Steel deck:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Width deck element</td>
<td>mm</td>
<td>600</td>
<td>600</td>
</tr>
<tr>
<td>Height deck element</td>
<td>mm</td>
<td>210</td>
<td>225</td>
</tr>
<tr>
<td>Thickness sheet</td>
<td>mm</td>
<td>1.25</td>
<td>1.25</td>
</tr>
<tr>
<td>Amount of steel</td>
<td>mm$^2$/m</td>
<td>2017</td>
<td>2278</td>
</tr>
<tr>
<td>Section modulus</td>
<td>cm$^3$/m</td>
<td>66.3</td>
<td>75.0</td>
</tr>
<tr>
<td>Second moment of area</td>
<td>cm$^4$/m</td>
<td>855</td>
<td>950</td>
</tr>
<tr>
<td>Weight$^1$</td>
<td>kg</td>
<td>70</td>
<td>79</td>
</tr>
<tr>
<td>Unpropped span</td>
<td>m</td>
<td>5.45</td>
<td>5.6</td>
</tr>
<tr>
<td>Steel grade</td>
<td>Mpa</td>
<td>350</td>
<td>350</td>
</tr>
<tr>
<td><strong>Composite slab$^{2,3}$:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Height</td>
<td>mm</td>
<td>280</td>
<td>285</td>
</tr>
<tr>
<td>Weight$^2$</td>
<td>kg/m$^2$</td>
<td>279</td>
<td>318</td>
</tr>
<tr>
<td>Fire resistance</td>
<td>min</td>
<td>60</td>
<td>60</td>
</tr>
</tbody>
</table>

$^1$ weight deck element for a length of 7.2 m

$^2$ imposed loads: 4.0 kN/m$^2$ or 7.0 kN

$^3$ normal weight concrete C20/25

Table 2.2: Section properties of deep decks: Comflor 210 and Comflor 225 [15, p.108]

### 2.2. THE STEEL-CONCRETE COMPOSITE SLAB

A steel-concrete composite floor slabs is made by casting concrete on top of a steel deck. The expression ‘steel-concrete composite floor slab’ implies that there is a system to connect the steel deck and the concrete slab. This means that if the steel deck and the concrete need to behave as one composite slab a mechanical interlock is required.

Three types of shear connectors are used to ensure composite behaviour between the steel deck and concrete:
2.2. **THE STEEL-CONCRETE COMPOSITE SLAB**

- dowels, similar to the principle of a downstand beam;
- dovetail, used to ensure composite behaviour in a re-entrant profile;
- embossments, provide shear resistance in most of the current decks.

Some examples are illustrated in figure 2.6. Here the embossments in the profiled steel sheet are one of the most efficient and applied systems to achieve sufficient interlocking between the steel deck and the concrete slab [27].

![Figure 2.6: Composite floor slabs using different types of profiled sheets](image)

There are three failure modes for steel-concrete composite slabs:

1. flexural failure (or bending failure);
2. longitudinal shear failure;
3. vertical shear failure.

These failure modes are illustrated in figure 2.7. In most cases the longitudinal shear failure is governing and therefore the design of the mechanical interlocking is important for the overall performance of the steel-concrete composite slab [12, 23].

This section elaborates on the structural behaviour of the steel-concrete composite slab. Besides that, attention is paid to the developments of the steel deck to function as shear connector. Finally, an overview is given of the required design verifications for the steel-concrete composite slab.

### 2.2.1. **MATERIAL PROPERTIES**

Material properties for reinforcing steel and concrete are found in table A.2 of the appendix. It is supposed that normal weight concrete (NWC) with a cylinder strength of 20 Mpa (C20/25) is used. The Eurocode prescribes that the concrete self-weight may be increased with 1 kN/m$^3$ for unhardened concrete and 1 kN/m$^3$ for a normal percentage of reinforcing steel [28, p.32].

#### LIGHTWEIGHT CONCRETE

The use of normal weight concrete (NWC) is common practice with steel-concrete composite floor slabs. However, it is possible to cast light weight concrete (LWC) if weight is an issue. Disadvantage of LWC are the higher costs, the availability and the difficulties that arise during construction.

![Figure 2.7: Location of critical sections in a simply supported composite slab](image)
2.2.2. **LONGITUDINAL SHEAR RESISTANCE**

Porter and Ekberg developed in 1976 a method to rate the shear bond characteristics of the embossment by two empirical parameters $m$ and $k$. Over 350 experiments were carried out to develop a design procedure for the determination of the ultimate strength of composite slabs. Simply supported one-way steel-concrete composite slabs were tested in a four point bending test. A linear regression relationship between $V_{us}/bd$ and $pd/L_s f'_c$ was established to determine the two constants, where the slope ($m$) represents the mechanical interlocking between the steel and concrete, and the intercept ($k$) the friction between the steel and the concrete [2, 3]. Once $m$ and $k$ are determined for a steel deck these factors may be used to predict the ultimate shear capacity ($V_u$) for other slab configurations with the same decking: different spans, slab thicknesses and concrete strengths.

At the moment two methods are available in the Eurocode 4 to determine the longitudinal shear resistance: the $m$-$k$ method and the partial connection method [12]. Both methods require full scale testing. The advantage of the partial connection method is that additional parameters, such as end anchorage and reinforcement may be included. This method, however, is only applicable to slabs with a ductile behaviour. Eurocode 4 defines the behaviour of a slab as ductile if the failure load exceeds the load causing first recorded end slip by more than 10%.

The success of the composite action is determined by the amount of horizontal shear forces that can be transferred between the steel deck and the concrete slab. The steel decking acts as tensile reinforcement if this transfer capacity is sufficient. Bending in the steel-concrete composite slab leads, besides to horizontal shearing forces, also to vertical separation between the steel and concrete [2]. This means that the steel decking has to be designed to not only transfer horizontal shear forces, but also resists vertical separation. The resistance to vertical separation is ensured by suitable shape of the trapezoidal profile and by the embossments.

**EMBOSSMENTS**

The longitudinal shear resistance of the steel-concrete interface is improved by providing the steel decking with embossments. These embossments are pressed in the web and flange and are usually made in shapes of circular spot, chevron or bar at different angles. Not only the shape and position of the embossment, but also the steel sheet thickness influence the bond strength. Due to the complexity, the effect of the embossments and steel sheet thickness on the bond strength of the composite slab is experimentally studied in a ‘push’ test.

The effect of different design measurements to improve the shear resistance is studied by Makalainen and Sun (1998, 1999), Jolly and Zubair (1987), Crisinel and Schumaker (2000), and Wright and Essawy (1996) [29, p.13-14]. Here an overview is given of the findings.

- vertical embossments are most effective in shear resistance;
- the chance for the concrete to slip over the deck is increased by discontinuities of embossment shape;
- increasing the amount embossments that requires a decreasing in size did not improved the shear resistance;
- the increased depth of embossments is the most effective to improve the shear resistance, but there is a maximum depth since tear can occur during production;
- the embossment is most effective if located in the middle of the web. Embossments in the corners of the web do not improve the shear resistance that much and are difficult to construct. Those in the compression flange increase the effect of buckling since these embossments act as initial deformations;
2.2. The Steel-Concrete Composite Slab

- increasing the length of the embossments leads to an increase of the shear resistance, but this effect is only significant up to a certain length;
- the concrete that enters the holes of the penetrated embossments leads to big improvements of the shear resistance;
- increased deck thickness improves the shear resistance;
- re-entrant profiles improve the shear resistance with 63-88%, where the relation between the improvement and the area of concrete under the re-entrant is linear. Unembossed re-entrant profiles still provide 50% of the shear strength compared to an embossed deck.

End Restraints

End anchorages at the support of composite slabs also contribute to the longitudinal shear resistance. An example are welded-studs on a steel-concrete composite beam that anchor the deck at the support. Seven test on simply supported one-span composite slabs with different end restraints where carried out by Chen [30]. From this study is clear that slabs with end anchorage have a higher shear bond strength than slabs without end anchorage.

2.2.3. Design Verifications of the Steel-Concrete Composite Slab

As mentioned before, the construction stage is often the critical loading for the design of a steel-concrete composite slab. However, the steel deck as well as the steel-concrete composite floor slab need to perform adequate in all load situations. The loading on the floor is defined for three different situations.

1. load resistance of the steel deck during construction (discussed in the previous section);
2. load resistance of the composite slab during service life;
3. load resistance of the composite slab during fire.

The steel-concrete composite slab should have sufficient strength and serviceability (or stiffness) during service life and in case of fire. Here is elaborated on different design verifications of the steel-concrete floor slab.

Composite behaviour

In this study the steel-concrete composite behaviour is not verified conform the Eurocode. The Eurocode prescribes that testing is required to determine the steel-concrete composite behaviour (m-k and/or $\tau_u$ values) of a new deck profile. m-k and/or $\tau_u$ values are required to verify the strength and serviceability criteria during the service life. However, the steel-concrete composite behaviour can be estimated by analogy or simple calculations and recommendation can be given for the required testing procedures. The design of shear connectors, that is necessary to ensure the composite behaviour of the steel-concrete interface, should be incorporated in the deck design.

Deflection

According to the national annex of Eurocode 0 the maximum total deflection $w_{tot}$ of a floor is limited to $L/250$ and the maximum additional deflection $w_{add}$ of a floor is limited to $L/500$. Therefore the maximum deflections of the composite floor during service life are limited by the following.

- $w_{add} < L/500$, for the additional deflections;
- $w_{tot} < L/250$, for the total deflections.

Note that the deflections during construction for unpropped construction may be excluded when determining the total deflections. However, according to Eurocode 0 should attention be paid to the appearance of the construction work. This can be the case if no ceiling is installed and large deflection are visible to the user of a building. The additional deflections need to be within the given criteria to prevent damage of non-structural members and finishing.
Vibrations

The floor vibrations are verified with the eigenfrequencies of the floor. The eigenfrequency or natural frequency is the frequency where a system, in this case the floor, tends to oscillate. The required eigenfrequency is given in the national annex to Eurocode 0 and depends on the function of a building [31, p.35-36 and 56]. The criteria for vibrations aim to prevent resonance of the floor that is experienced as unpleasant by the users of a building. Dwelling and office buildings require a eigenfrequency of at least 3 Hz and gyms or dancing halls at least 5 Hz. The eigenfrequency is determined with a quasi-permanent load-combination according to Eurocode 0 article 6.16b for a short-term loading situation. Verification for vibrations are not required for loads over 5 kN/m².

Fire resistance

The floor design should have sufficient fire resistance. This is verified according to Eurocode 0 [31, p.36-37] and Eurocode 4 part 2 [12]. Steel-concrete composite floors have to be verified for three criteria regarding the fire resistance.

- resistance or strength (R);
- thermal insulation (I);
- structural integrity (E).

The required mechanical resistance of the floor slab for fire depends on the function of the building, the height of the building and the permanent fire load. Commercial buildings with a height of more than 5 m require a fire resistance of 90 minutes. A reduction is allowed for fire loads less than 500 MJ/m². This is often the case for office buildings, where a fire resistance of 60 minutes is sufficient [32, p.22-23]. Current steel-concrete composite floors designed for room temperature have a fire resistance of at least 30 minutes. This can go up to 60 minutes if standard reinforcement is applied (to prevent shrinkage).

Advantages of the steel-concrete composite slab are summarised in table 2.3.

<table>
<thead>
<tr>
<th>Property:</th>
<th>Advantage:</th>
</tr>
</thead>
<tbody>
<tr>
<td>General:</td>
<td></td>
</tr>
<tr>
<td>Construction height</td>
<td>• minimal, when combined with a asymmetric steel beam</td>
</tr>
<tr>
<td>Light weight construction</td>
<td>• low self-weight compared to other concrete floor systems</td>
</tr>
<tr>
<td></td>
<td>• reduce costs for the foundation</td>
</tr>
<tr>
<td></td>
<td>• suitable for complex projects, renovation and high-rise</td>
</tr>
<tr>
<td></td>
<td>• sustainable and cost efficient due to efficient use of materials</td>
</tr>
<tr>
<td>Concrete floor slab:</td>
<td></td>
</tr>
<tr>
<td>Monolith floor construction</td>
<td>• diaphragm to transfer horizontal loads</td>
</tr>
<tr>
<td></td>
<td>• prevention of lateral torsional buckling for steel beams</td>
</tr>
<tr>
<td>Not sensitive for cracking</td>
<td>• shrinkage is limited and no creep if floors are unpropped</td>
</tr>
<tr>
<td>Steel decking:</td>
<td></td>
</tr>
<tr>
<td>Deck as reinforcement</td>
<td>• less material costs</td>
</tr>
<tr>
<td>Installing the reinforcement</td>
<td>• less labour costs</td>
</tr>
<tr>
<td>Quick installation</td>
<td>• save on interest and rental costs</td>
</tr>
<tr>
<td>Fire resistance:</td>
<td></td>
</tr>
<tr>
<td>Minimal 60 minutes fire resistance</td>
<td>• good fire resistance</td>
</tr>
<tr>
<td>Finishing:</td>
<td></td>
</tr>
<tr>
<td>Trapezoidal bottom</td>
<td>• ceiling not necessary</td>
</tr>
<tr>
<td></td>
<td>• integrated system to assemble a ceiling and other services</td>
</tr>
<tr>
<td></td>
<td>• space for services between ribs</td>
</tr>
<tr>
<td></td>
<td>• effective thermal mass</td>
</tr>
</tbody>
</table>

Table 2.3: Properties and advantages of steel-concrete composite floor slab during service life
In this chapter the findings from the literature review are formulated in specifications for the design. The scope of this research is limited by the manufacturing process and the material properties of the steel deck, as discussed in section 2.1.1. Other specifications aim to maintain the existing advantages of the steel deck and steel-concrete composite floor slabs, as summarised in table 2.1 and 2.3. Design rules obtained from the Eurocode are used to verify the steel deck as well as the steel-concrete composite floor for strength and serviceability in all load stages. Together with the main objective of this study the specifications for the design are given in table 3.1. These specifications are used as a starting point for the design and form a basis for evaluation.

<table>
<thead>
<tr>
<th>Steel deck</th>
<th>Composite slab</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Boundary conditions/assumptions:</strong></td>
<td></td>
</tr>
<tr>
<td>• maximum thickness steel sheet ≤ 1.5 mm</td>
<td>• normal weight concrete C20/25</td>
</tr>
<tr>
<td>• yield strength steel 350 Mpa</td>
<td></td>
</tr>
<tr>
<td>• maximum coil width ≤ 1400 mm</td>
<td></td>
</tr>
<tr>
<td><strong>Dimensions:</strong></td>
<td></td>
</tr>
<tr>
<td>• lightweight deck elements &lt; 50 kg</td>
<td>• aim at ‘minimum height’ floor slab &lt; 300 mm</td>
</tr>
<tr>
<td>• length deck elements 7.2 m</td>
<td>• aim at ‘lightweight’ floor slab &lt; 300 kg/m²</td>
</tr>
<tr>
<td>• aim at thickness steel sheet 1.0 up to 1.25 mm</td>
<td></td>
</tr>
<tr>
<td>• aim at ‘minimum amount’ of steel $A_p/G_p$</td>
<td></td>
</tr>
<tr>
<td><strong>Criteria:</strong></td>
<td></td>
</tr>
<tr>
<td>• deflection during construction: $\delta_0 &lt; L/180$</td>
<td>• total deflection: $w_{tot} &lt; L/250$ ($\delta_0$ not incl.)</td>
</tr>
<tr>
<td>• deck as: work-floor, shuttering, and reinforcement</td>
<td>• additional deflection: $w_{addl} &lt; L/500$</td>
</tr>
<tr>
<td>• construction unpropped</td>
<td>• adequate performance of floor vibrations: $f_p &gt;$3-5 Hz</td>
</tr>
<tr>
<td>• elements are stackable</td>
<td>• aim at minimal material use: sustainability</td>
</tr>
<tr>
<td>• deck should establish composite behaviour</td>
<td>• fire resistance ≥ 90 minutes</td>
</tr>
<tr>
<td>• simple connections: pop nails or self-drilling screws</td>
<td>• building services preferably integrated in floor slab</td>
</tr>
<tr>
<td>• aim at minimal actions to install floor</td>
<td>• aim at flat bottom slab</td>
</tr>
<tr>
<td>• prevent LTB steel beam/provide diaphragm action</td>
<td>• monolith floor that provides diaphragm action</td>
</tr>
</tbody>
</table>

Table 3.1: Specifications for the design
Part 2

Specifying the Design
In this preliminary analysis the load-carrying and deflection characteristics of the steel deck during construction are studied. Here is assumed that the deck carries its own self-weight, the weight of the wet concrete, and the construction loads. Furthermore an indication is given of the extra load-carrying capacity required for bigger spans to indicate the feasibility of this study.

4.1. **Strength and Stiffness**

In this section the load-carrying and deflection characteristics are studied to determine the effect of ponding. Besides that, two expressions to obtain the minimum strength and stiffness of the deck are given.

4.1.1. **Euler-Bernoulli Beam**

The structural behaviour of the deck is analysed by assuming a certain width of the deck as a beam. This 'beam' is described with the Euler-bernoulli bending beam theory to obtain the load-carrying and deflection characteristics of the deck.

For a simply supported beam the maximum deflection $w_{\text{max}}$ and the maximum bending moment $M_{\text{max}}$ are found at half the span ($L/2$).

\[
\begin{align*}
  w_{\text{max}} &= w \left( \frac{L}{2} \right) = \frac{5}{384} \frac{qL^4}{EI_{\text{eff}}} \\
  M_{\text{max}} &= M \left( \frac{L}{2} \right) = \frac{1}{8} qL^2
\end{align*}
\]

A derivation of these expressions is found in appendix A.3.

4.1.2. **Bending Stiffness**

Deflections are calculated in Service Limit State (SLS) where only the linear elastic capacity is taken into account. The required stiffness ($EI_{\text{eff}}$) can be determined if the maximum deflection is known.

\[
\begin{align*}
  w_{\text{max}} &\leq \frac{5}{384} \frac{qL^4}{EI_{\text{eff}}} \quad \Rightarrow \quad EI_{\text{eff}} &\geq \frac{5}{384} \frac{qL^4}{w_{\text{max}}} \\
  \delta_{s,\text{max}} &= L/180. \text{ Substitution of } \delta_{s,\text{max}} = w_{\text{max}} = L/180 \text{ in expression above gives:} \\
  EI_{\text{eff}} &\geq \frac{75}{32} qL^3
\end{align*}
\]

The maximum deflection is defined as: $\delta_{s,\text{max}} = L/180$. Substitution of $\delta_{s,\text{max}} = w_{\text{max}} = L/180$ in expression above gives:

\[
EI_{\text{eff}} \geq \frac{75}{32} qL^3
\]
4. PRELIMINARY ANALYSIS

4.1.3. EFFECT OF PONDING

The effect of ponding occurs during the construction of the composite slab when the deck acts as shuttering. The wet concrete will accumulate due to the deflection of the deck under the weight of the wet concrete. Eurocode 4 describes that the effect of ponding should be taken into account if the maximum deflection \( w_{\text{max}} \) is bigger than 10% of the total slab depth. The effect of ponding is than taken into account by increasing the nominal concrete thickness by \( 0.7 \cdot w_{\text{max}} \) [12, p.98].

Expression A.1 in the appendix describes the deflection of the deck with span \( L \) over \( x \) for a uniform distributed load \( q \). The amount of wet concrete that will accumulate is approximated with this deflection curve. The amount of accumulated concrete is determined by integrating this expression over the span length \( L \).

\[
\text{Area} = \int_0^L w(x) \, dx = \frac{1}{E I_{\text{eff}}} \left[ \frac{1}{120} q x^5 - \frac{1}{48} q L x^4 + \frac{1}{48} q L^3 x^2 \right]_0^L = \frac{1}{120} \cdot \frac{q L^5}{E I_{\text{eff}}} \quad (\text{mm}^2)
\]

This is translated into a load increment. The area is equally divided over the span length to simplify the calculation. The ponding load for a square meter of floor area is obtained.

\[
h_{\text{ponding}} = \frac{\text{Area}}{L} = \frac{q L^4}{120 E I_{\text{eff}}} \quad (\text{mm over the floor area})
\]

\[
q_{\text{ponding}} = \left( \frac{h_{\text{ponding}}}{1000} \cdot \rho_c \cdot g \right) / 1000 \quad (\text{kN/m}^2)
\]

The required bending stiffness is now obtained with expression 4.2 taking the effect of ponding into account with expression 4.3. Note that multiple iterations are required to provide a safe estimation of the ponding effect.

This approach gives similar results as the design rule given in Eurocode 4, where \( h_{\text{ponding}} \) is calculated with \( 0.7 \cdot w_{\text{max}} \). This is elaborated on in appendix A.4.

4.1.4. BENDING STRENGTH

The bending strength or bending moment resistance \( (M_{Rd}) \) is calculated in Ultimate Limit State (ULS) taking only the linear elastic capacity into account as shown in figure 4.1 (no plastic deformation capacity). This is a conservative, but safe assumption.

The bending moment resistance depends on the effective cross-sectional properties of the steel deck and is influenced by the effective section modulus \( (W_{\text{eff}}) \), the yield strength of the base material \( f_{yb} \) and a material factor \( (\gamma_{M0}) \).

\[
M_{\text{el,Rd}} = W_{\text{eff}} \cdot \frac{f_{yb}}{\gamma_{M0}}
\]

Combined with expression 4.1 it means that the section modulus for a single span deck should fulfill the following requirement:

\[
M_{Rd} \geq M_{Ed} = M_{\text{max}}
\]

\[
\Rightarrow W_{\text{eff}} \geq \frac{1}{8} q L^2 \gamma_{M0} / f_{yb}
\]

Figure 4.1: Bending moment resistance of the deck
4.2. REQUIRED STRENGTH AND STIFFNESS

In this section an indication of the required strength as well as the stiffness is given for the steel deck of a steel-concrete composite floor slab constructed without temporary supports. The strength is expressed in $W$ and the stiffness is expressed in $I$. The loads during construction are estimated with an existing steel-concrete composite slab; the Comflor 225 with a slab height of 285 mm [8].

- $G_s = 0.18 \text{kN/m}^2$ (self-weight of the deck);
- $V_c = 125 \text{l/m}^2$ (volume of the wet concrete);
- $\rho_c = 26 \text{kN/m}^3$ (density of the wet reinforced concrete);
- $G_c = \frac{\rho_c V_c}{1000} = 26 \cdot \frac{125}{1000} = 3.25 \text{kN/m}^2$ (concrete self-weight).

The loads during construction are calculated according to table A.1 of the appendix.

In SLS: $q_{Ed} = 1 \cdot 0.75 + 1 \cdot 3.43 = 4.18 \text{kN/m}^2$ (426 kg/m$^2$)
In ULS: $q_{Ed} = 1.5 \cdot 0.75 + 1.2 \cdot 3.43 = 5.25 \text{kN/m}^2$ (536 kg/m$^2$)

The loads in SLS and ULS during construction are illustrated in figure 4.2.

\[
\begin{array}{c|c|c}
\hline
\text{Span (m)} & \text{Second moment of area $I$ (cm}^4/\text{m}) & \text{Section modulus $W$ (cm}^3/\text{m}) \\
\hline
7.2 & 1745 & 60\% & 77.6 & -12\% \\
8.1 & 2485 & 128\% & 98.2 & 12\% \\
10.8 & 5891 & 441\% & 174.6 & 99\% \\
14.4 & 13963 & 1182\% & 310.3 & 253\% \\
\hline
\end{array}
\]

W calculated with $f_{yb}$ is 350 MPa

The required bending stiffness and bending strength are determined with expressions 4.2 and 4.5. The effect of ponding is taken into account. In table 4.1 is the required strength ($W$) and stiffness ($I$) is determined for the steel decking during construction for a range of bigger spans.

The gray values in table 4.1 give a comparison of the required cross-sectional properties ($I$ and $W$ required) and the properties of the Comflor 225 deck. The bending stiffness of the current Comflor 225 deck should increase 60% to span 7.2 m without the need for temporary supports. A very high increase of the bending stiffness is required for the spans over 10 m. Note that the bending strength of the Comflor 225 is already sufficient. However, the Comflor 225 deck does not meet other criteria given in chapter 3. This deck profile is not suitable, because the deflections during construction and the self-weight of a single deck panel are too big.

From this table is clear that bigger spans require an increase of the bending stiffness and strength. The required stiffness increases faster for bigger spans than the required strength. This also can be seen in expression 4.2, where the parameter $L$ is found to the power four and in expression 4.5, where the parameter $L$ is found to the power two.

![Figure 4.2: Loads in the deck during construction](image)

Table 4.1: Required bending stiffness and strength for a range of bigger spans

<table>
<thead>
<tr>
<th>Span (m)</th>
<th>Second moment of area $I$ (cm$^4$/m)</th>
<th>Section modulus $W$ (cm$^3$/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.2</td>
<td>1745</td>
<td>60% 77.6  -12%</td>
</tr>
<tr>
<td>8.1</td>
<td>2485</td>
<td>128% 98.2  12%</td>
</tr>
<tr>
<td>10.8</td>
<td>5891</td>
<td>441% 174.6  99%</td>
</tr>
<tr>
<td>14.4</td>
<td>13963</td>
<td>1182% 310.3  253%</td>
</tr>
</tbody>
</table>

W calculated with $f_{yb}$ is 350 MPa

ponding effect is taken into account

values in gray: comparison with Comflor 225: $I = 1089.0 \text{cm}^4$ and $W = 88.0 \text{cm}^3$ [8]
4.3. **Conclusion**

It may be concluded that the stiffness of the deck is governing during construction and not the strength. Therefore the bending stiffness is the leading design criteria for a steel deck loaded during construction.

The Comflor 225 has at the moment the biggest unpropped span capacity and can span up to 5.5 m without the need for temporary supports. If this deck system would span 7.2 m unpropped the bending stiffness ($E I_{\text{eff}}$) should increase with 60%. The bending strength of the Comflor 225 is already sufficient to span 7.2 m unpropped. However, this deck profile is not suitable, because the deflections during construction and the self-weight of a single deck panel are too big.

A deck design for an unpropped span of 7.2 m is feasible. However, spans bigger than 7.2 m are very demanding.
Brekelmanns has developed some concepts for alternative steel-concrete composite floor systems for bigger spans manufactured with profiled steel sheets that have not been applied in practice [4, p.28]. These concepts are illustrated in figures 5.1a, 5.1c and 5.1b. This chapter focuses on the development of concept deck designs for bigger span steel-concrete composite slabs. The ideas of Brekelmanns and other design concepts are described as well as discussed. Also, their feasibility is assessed.

(a) Steel beams and composite floor manufactured in one process  
(b) Sandwich panel with composite floor as top skin  
(c) Deep deck with steel beams in ribs

Figure 5.1: Alternative designs for steel-concrete composite floor slabs [4, p.28]

In the previous chapter is shown that the deflections of the deck are a leading design criteria. For this reason, alternative deck designs are presented that provide more bending stiffness and limit the deflection during construction. In addition, different criteria are used to evaluate other aspects of alternative design concepts without the necessity of any calculations. These criteria are obtained from table 3.1.

- composite behaviour: the deck should be able to transfer sufficient shear forces to the concrete;
- resistance to fire: 'good' fire resistance of the individual elements and total floor slab;
- transport: stackability of the deck elements;
- installation: lightweight deck elements, deck as working platform and shuttering;
- economy: 'minimum' amount of steel in the deck.

Besides that, the steel deck should be designed to prevent local buckling of the thin steel in the deck (susceptibility of the deck for buckling).
5.1. **DEVELOPMENT OF DIFFERENT DESIGN CONCEPTS**

In this section different design concepts are described and illustrated. Their feasibility is assessed based on the criteria mentioned above.

**5.1.1. CONCEPT 1**

In concept 1 steel beams and the decking for the steel-concrete composite floor are manufactured in one rolling process (figures 5.1a and 5.2).

- low profiled sheets are bend in the same rolling process;
- bend borders become thin-walled cold-formed steel beams;
- hollow space could be filled with thermal insulation.

The governing design criteria:

- transport: panels cannot be stacked efficient;
- support on thin-walled steel beams is critical;
- resistance to fire: thin-walled sections are not protected.

![Figure 5.2: Steel beams and composite floor manufactured in one rolling process](image)

**5.1.2. CONCEPT 2**

Concept 2 is a sandwich panel with composite floor as top skin (figures 5.1b and 5.3).

- standard sandwich panel, but now with a profiled steel sheet as top skin;
- similar to OP-deck: concrete rib floor cast on sandwich panel that functions as lost shuttering;
- application: with insulation suitable for inside-outside situation.

The governing design criteria:

- transport/installation: stacking and installation of sandwich elements is less efficient than deep decks;
- economy: relative high costs due to insulation and the amount of steel.

![Figure 5.3: Sandwich panel with composite floor as top skin](image)
5.1.3. Concept 3

A deep deck with steel beams in the ribs (figures 5.1c and 5.4).

- combination of two existing sections: deep decks placed on cold-formed sections (beams);
- beams required to provide more bending stiffness during construction (not for extra composite action).

Governing design criteria:

- installation: positioning and connecting the elements at site requires more effort;
- economy: large amount of steel is used.

Figure 5.4: Deep deck with steel beams in ribs

5.1.4. Concept 4

This concept is the upscale of current deep decks: a steel-concrete composite floor with ribs (figure 5.5).

- upscale of the current deep deck design to provide more bending stiffness during construction;
- thicker steel sheet, more profile height and/or more ribs per meter width;
- structural behaviour is comparable to current composite floors slabs with deep decks;
- sufficient composite behaviour and good fire resistance;
- two deck designs: trapezium or inverted trapezium.

Governing design criteria:

- loads during construction versus the bending stiffness of the steel deck (upscale will also lead to an increase in the amount of concrete and therefore a higher self-weight).

Figure 5.5: Upscale of current deep deck: steel-concrete composite floor with ribs
5.1.5. **Concept 5**

Alternative deck shapes: inverted U-profiles, Z-profiles, and box-profiles (figure 5.6).

- low self-weight: only a thin concrete layer on top.

Governing design criteria:

- composite behaviour: difficult to provide shear connection;
- resistance to fire: thin-walled sections are not protected;
- buckling: unsupported thin-walled steel is susceptible for buckling.

![Figure 5.6: Alternative deck shapes: inverted U-profiles (left), Z-profiles, and box-profiles (right)](image)

5.1.6. **Concept 6**

Cold-formed sections (CFS) as downstand beam with a steel-concrete composite floor on top (figure 5.7).

- U-decks: comparable to the IDES floor system (IDES no composite action);
- CFS: comparable to Sadef steel frame.

Governing design criteria:

- composite behaviour: difficult to provide shear connection between beams and the concrete;
- resistance to fire: thin-walled sections are not protected;
- buckling: unsupported thin-walled sections are susceptible for buckling;
- economy: large amount of steel is used (low profile decking in the transverse direction of the span).

![Figure 5.7: Cold-formed sections as downstand beam: U-decks (left) and CFS (right)](image)
5.1.7. Concept 7

Concept 7 is a steel-concrete composite slab with weight reduction (figure 5.8).

- weight reduction in the ribs of deep decks;
- wider ribs with lightweight fill element: block (e.g. EPS) or a plastic tube;
- floor behaves as a steel-concrete composite slab with weight reduction;
- three deck designs: trapezium, inverted trapezium, and alternating trapezium.

Governing design criteria:

- loads during construction versus the bending stiffness of the steel deck.

![Figure 5.8: Steel-concrete composite slab with weight reduction](image)

5.1.8. Concept 8

This concept is an alternating trapezium deck for steel-concrete composite slab (figure 5.9).

- deep decks are positioned alternately to provide more bending stiffness during construction;
- good alternative if the amount of effective steel cannot be provided within a limited floor height.

Governing design criteria:

- economy: large amount of steel is used.

![Figure 5.9: Alternately trapezium deck for steel-concrete composite slab](image)
5.1.9. **Concept 9**

The steel-concrete composite slab is installed with a double cast (figure 5.10).

- **Deep decks are installed with a double cast:** first cast is only the ribs, second cast is the final casting.

Governing design criteria:

- **Installation:** double cast requires an extra action at site;
- **Composite behaviour:** between the second cast and the steel-concrete composite ribs.

![Figure 5.10: Steel-concrete composite slab installed with a double cast](image)

5.2. **Conclusion**

Most design concepts are rejected, because these designs do not meet the criteria for resistance to fire and buckling. The thin-walled cold-formed steel in these designs is not protected or supported by concrete. In addition, some design concepts are not suitable due to the demanding installation process or the large amount of steel that is used.

The design concepts feasible for further development are design alternatives for the current deep decks, where concrete is cast in ribs. The concrete in the ribs is of importance to provide good fire resistance, sufficient steel-concrete composite behaviour and to support the thin-walled steel.

Large deflections during construction are limited by an increased load-carrying capacity and/or a weight reduction. More load-carrying capacity is provided with a thicker steel sheet, a higher deck profile and/or more ribs per meter deck width. Weight reduction is provided with a lightweight fill element.
STEEL-CONCRETE COMPOSITE FLOOR WITH RIBS

In the previous chapter is concluded that the shape of the deck should be trapezoidal with a reinforcement bar embedded in a concrete rib. An unpropped span of 7.2 m is achieved by a deck that has more load-carrying capacity and/or a reduction of the self-weight of the floor.

In this chapter two design concepts are introduced and studied: a deck design with small ribs, and a deck design with wide ribs and a lightweight fill element in these wide ribs (figure 6.1). Firstly, some of the made assumptions are given for the design of the deck and steel-concrete composite floor. In 4 steps a number of designs are verified in an increasing level of detail. After that the design and installation process is discussed. Finally, one design is selected for further development.

6.1. DESIGN ASSUMPTIONS

The design assumptions are used for the design and verification of the steel deck and steel-concrete composite floor slab.

**General**: the deck and composite slab are designed and verified as single span.

**Weight reduction**: the most suitable type of weight reduction is not yet investigated. Here is assumed that the fill element has no contribution to the cross-sectional resistance. The weight of the fill element is taken into account as: \( \rho_{EPS} = 45 \text{ kg/m}^3 \).

**Local transverse force**: the thin steel webs are in general not able to resist the local transverse force at the support. However, current deep decks use cleats to take up these high local forces. These cleats are not designed in this chapter and therefore this criteria is not verified.

Figure 6.1: Two design concepts: small ribs and wide ribs with fill element
Figure 6.2: Requirements for the concrete cover

**Cover:** a cover of 30 mm is taken into account to ensure sufficient bondage between the reinforcement in the bottom of the ribs and the concrete. A cover of minimal 20 mm, between the webs of the deck and the fill element, is taken into account to ensure sufficient longitudinal shear resistance. Both are illustrated in figure 6.2.

**Fire resistance:** the fire resistance of a composite slab can be determined with the method described in NEN-EN 1994-1-2. The scope of the method is defined in table B.1 and figure B.3 of the appendix. The method is not applicable for deep deck composite slabs and also not for the two design concepts, because the width of the top flange ($l_3$) and the height of the profile ($h_2$) are exceeding the scope.

From practice is known that the fire resistance of current deep decks is ‘good’. The reinforcement bar in the bottom of the ribs is required to ensure a minimal fire resistance of 30 minutes. With this bar a fire resistance of 60 minutes is easily provided. However, the temperature of the reinforcement bar heat up relatively quick for severe fire loads of 90 and 120 minutes. The maximum span capacity of the floor is than determined by the fire resistance [8].

The behaviour during fire, of the two design concepts that are studied in this chapter, is comparable to the current deep decks. Therefore is assumed that the fire resistance is sufficient.

**Longitudinal shear resistance:** longitudinal shear resistance is based on test values of the Comflor 210. This is a conservative assumption, because the new design has a longer shear span, more ribs per meter width, and in some cases a thicker steel sheet.

**Vertical shear:** concentrated loads in general are the governing load case for vertical shear. The method given in the Eurocode to calculate the effective width for a point load is not applicable for deep decks: $h_p / h \leq 0.6$ (NEN-EN 1994-1-1). Here is a safe approach used by taking only the capacity of one rib into account.

### 6.2. Deck Design

In this section the two design concepts for a steel-concrete composite floor with ribs are studied.

1. deck design with small ribs (figure 6.1a);
2. deck design with wide ribs and a fill element in the wide ribs (figure 6.1b).

The two concepts are designed and verified in four steps increasing the detail level of the calculations.

- **Step 1:** rough deck design and verification of the deck and composite slab;
- **Step 2:** trapezoidal deck design and verification of the deflection during construction;
- **Step 3:** design of stiffeners and embossments, verification of the effective cross-section and deflection during construction;
- **Step 4:** redesign and verification of the deck and composite slab during construction and service life.

#### 6.2.1. Step 1: Rough Deck Design

Different design variants are studied while changing some important parameters: steel thickness $t$, profile height $h_p$, width of the top flange $b_0$, width of the bottom flange $b_u$, and panel width $b_d$ (figure 6.1). In this
6.2. **DECK DESIGN**

(a) Small ribs

(b) Wide ribs and fill element in the wide ribs

Figure 6.3: Method to estimate the amount of effective steel

rough design are embossments and stiffeners not yet designed. Therefore is assumed that 60 mm of steel is effective around the corners of the deck, as shown in figure 6.3, and local buckling is not account for.

The design variants are verified during construction and service life. An calculation example, where the deck is verified during construction and the composite slab during service life, is given for the design with small ribs in appendix B.1. The calculation procedure is similar for other design variants.

In table B.5 of the appendix are different design variants of the two design concepts compared. The comparison is made for 'minimum floor height', \( h_p \) is 210 mm, and different steel sheet thicknesses, \( t \) is 1.0, 1.25 or 1.5 mm. The deck deflection during construction is the governing criteria for all design variants. The shear resistance of the steel-concrete composite slab is also a governing design criteria for the design with small ribs.

From this table is observed that a bigger c.t.c. distance between the ribs \( b_d \) is possible for a thicker steel sheet \( t \). Besides that, the self-weight of the steel-composite floor slab is high compared to current decks.

6.2.2. **STEP 2: TRAPEZOIDAL DECK DESIGN**

The rectangular deck profiles from the previous section cannot be stacked. Therefore the decks are designed in a trapezoidal shape. This trapezoidal shape has influence on the amount of effective steel in the cross-section of the deck and the amount of concrete in the composite slab. In this step the designs from the previous section are designed as trapezoidal shaped elements.

Firstly, some considerations are given for the design and verification of trapezoidal deck profiles. Secondly, the results of the trapezoidal deck profile designs are compared and redesigned if necessary. Finally, is concluded which design variants are selected for further development.

A trapezoidal deck shape is stackable, which leads to an effective transportation and installation of steel-concrete composite floor slabs. Re-entrant shaped shear connectors have a negative effect on the stacking of the individual deck elements. The difference is illustrated in figure 6.4, where the Comflor 225 is clearly less suitable to stack efficient. An angle between 70° and 80° is assumed sufficient to stack the individual deck panels. Therefore \( \alpha = 80^\circ \) is chosen to ensure that the deck panels can be stacked, but the amount of concrete in the ribs is kept to a minimum.
The minimum cover is determined according to NEN-EN 1992-1-1 [6, p.49-50]:

\[ c_{\text{nom}} = c_{\text{min}} + \Delta c_{\text{dev}} \]

Where:
- \( c_{\text{nom}} \) is the nominal concrete cover in mm;
- \( c_{\text{min}} \) is the minimum cover;
- \( \Delta c_{\text{dev}} \) is the allowance in the design for deviation;

The minimum cover for a bar is taken equal to the bar diameter. The national annex of NEN-EN 1992-1-1 gives that \( \Delta c_{\text{dev}} \) is 5 mm. Therefore is \( c_{\text{nom}} = \phi + 5 \) mm, where \( \phi \) is the bar diameter in mm. Thus for a tension reinforcement bar of 20 mm is the a minimum cover of 25 mm required.

The dimensions are illustrated in figure 6.5. Here the labeling of the deck parameters and the minimum width of the bottom flange is illustrated. For \( \phi 16 \) mm bottom reinforcement holds that \( c_{\text{nom}} = 16 + 5 = 21 \) mm. Therefore is assumed that the bottom flange \( b_u \) should have a minimum width of 40 mm.

In table B.6 and B.7 of the appendix are different variants for the two design concepts studied. The deck designs are only verified for the maximum deflection during construction \( \delta_0 \). The bending stiffness of the designs from step 1 prove to be insufficient and therefore a improvement of the bending stiffness is required. Element width \( b_d \), profile height \( h_p \), and steel sheet thickness \( t \) are varied for the different trapezoidal designs with a slant angle \( \alpha \) of 80°.

The following designs have sufficient bending stiffness to limit the deflections during constructions and are suitable for further development.
6.2. DECK DESIGN

- design concept 1: design variant $t = 1.5$ (table B.6)
- design concept 2: design variant $t = 1.25$ mm and $h_p = 220$ mm (table B.7)

These variants are most suitable when aiming at ‘minimum floor height’ requirements and a minimum amount of steel.

6.2.3. STEP 3: DESIGN OF STIFFENERS AND EMMOSSMENTS

In this step the two selected decks, from step 2, are designed with stiffeners and embossments. In addition, the amount of effective steel is determined according to the ‘effective width method’ obtained from Eurocode 3.

For the two design concepts different flange stiffeners are suitable. In figure 6.6a two symmetric flange stiffeners are applied in design with small ribs. In figure 6.6b an edge stiffener is applied in the design with wide rib. In both design concepts are the embossments located in the web.

Figure 6.6: Effective cross-section and design of flange- and web stiffeners

The effective width method is used to verify three situations:

- no stiffeners;
- only flange stiffeners;
- flange and web stiffeners.

The calculation procedure and results are presented in appendix B.3, where a calculation example for design concept 1 is given. The calculation procedure for the other design is similar. In tables B.11 and B.12 are the results presented, where the total amount of steel $A_p$, total amount of effective steel $A_{pe}$, the effective second moment of area $I_{eff}$, and the deflection during construction $\delta_0$ are given. From these tables is clear that flange and web stiffeners are required to increase the effective cross-section $A_{pe}$ and the bending stiffness $I_{eff}$. Besides that, these tables show that the assumptions for the effective cross-section from step 1 were in the right direction, but are not accurate enough to design with.

Table 6.1 gives an overview of the dimensions of the deck, the cross-sectional properties of the deck, and the deflection during construction for the two designs with flange and web stiffeners. From this table is clear that design with small ribs has insufficient bending resistance to limit the deflections during construction. The decks are redesigned in step 4. The amount of effective steel is now determined with the ‘effective width method’ and stiffeners and embossments are designed for.

6.2.4. STEP 4: REDESIGN OF THE DECKS

In this section the different deck designs are redesigned. The decks are verified during construction and the composite slabs are verified during service life. Stiffeners are required to ensure sufficient cross-sectional resistance of the deck during construction. Embossments are required to ensure sufficient composite behaviour between the steel and the concrete. The design variants are designed to limit the deflections during construction $\delta_0$. 
Table 6.1: Two design concepts: cross-sectional properties and deflections during construction

An overview of the variant study is given in table B.13. In this table the different variants are evaluated according to the Unity Checks (UC) and the design specifications (chapter 3). A summary of the findings are given in table 6.2.

Table 6.2: Variant study for the two designs compared with existing deep decks

All the variants are designed having sufficient bending stiffness during construction. The results in table B.13 show that decks with a steel sheet thickness of \( t = 1 \) mm fail during construction in ULS. The decks with a steel sheet thickness of \( t = 1.5 \) mm result in rather heavy deck elements (for a length of 7.2 m) that cannot be manhandled at site. Therefore are the designs with a thickness of \( t = 1.25 \) mm most favourable.

In the design specification is stated that the floor design should aim at minimum self-weight and that the deck panels should be manhandled. The steel-concrete composite slab of the deck design with small ribs are heavy and therefore is this design less suitable for further development. This design is heavy due to the large
amount of concrete $V_c$ (L/m²) that is used in the design as shown in figure 6.7a. The deck of the design concept with wide ribs and fill elements meets all design criteria and therefore is suitable for further development (illustrated in figure 6.7b).

![Design Concepts](image)

(a) design concept 1: small ribs  
(b) design concept 2: wide ribs with fill element

Figure 6.7: Two design concepts: composite slabs (left) and single deck elements (right)

### 6.2.5. Conclusion

In this section the two design concepts for a steel-concrete composite floor with ribs are studied: deck design with small ribs and a deck design with wide ribs and a fill element in the wide ribs. The two design concepts are designed and verified in four steps increasing the detail level of the calculations.

In step 1 different design variants are studied while changing some important parameters. In this rough design are embossments and stiffeners not yet designed. The design variants are verified during construction and service life. Here is observed that a bigger c.t.c. distance between the ribs $b_d$ is possible for a thicker steel sheet $t$.

In the step 2 the decks are designed in a trapezoidal shape. This trapezoidal shape has influence on the amount of effective steel in the cross-section of the deck and the amount of concrete used in the composite slab. The deck designs are only verified for the maximum deflection during construction $\delta_0$. The trapezoidal shape leads in general to an increase of the amount of concrete and a decrease in the amount of effective steel.

In step 3 the decks, from step 2, are designed with stiffeners and embossments. In addition, the amount of effective steel is determined according to the ‘effective width method’ obtained from Eurocode 3. From this study is obtained that the design of stiffeners and embossments is required to provide a sufficient amount of effective steel.

In step 4 the different deck designs are redesigned. Stiffeners are required to ensure sufficient cross-sectional resistance of the deck during construction. Embossments are required to ensure sufficient composite behaviour between the steel and the concrete. It is found that the designs with a sheet thickness of $t = 1.25$ mm are most favourable. The deck of the design concept with small ribs are heavy and cannot be man-handled. Therefore this design is less suitable for further development. The deck of the design concept with wide ribs and fill elements meet all design criteria and therefore is suitable for further development.

### 6.3. Installation with a Double Cast

In the previous section is found that the deck design with small ribs do not meet the requirements, because the decks are heavy and cannot be man-handled. However, a double cast, where the concrete is cast in two stages, can provide extra stiffness during the installation of the floor allowing for a lighter deck profile. This section studies the effect of a double cast on the deflection of a deck with small ribs and a steel thickness of 1.25 mm.

#### 6.3.1. Double Cast

As shown in figure 6.8 are in the first cast only the ribs or a part of the ribs cast. When the concrete in the ribs is hardened it will provide extra bending stiffness for the second cast. The installation of the floor slab has two load stages and the finished floor behaves as current steel-concrete floor slabs. Two situations are considered.
first cast up to centroid of the deck;

first cast up to top of the deck.

The floor slab is finished with a second cast. The deflections of the deck with a single cast are used as reference value: $\delta_0 = 44.2$ mm (ponding included).

**LOAD STAGES**

During the first cast carries the decking its own weight, the wet concrete with tension reinforcement, and the construction loads. During the second cast carries the decking with the concrete ribs its own weight, the wet concrete with top reinforcement, and the construction loads.

**BENDING STIFFNESS**

The second moment of area of the steel deck with concrete ribs is determined. Some assumptions are made:

- short-term loading: $E_{cm} = 30000$ N/mm² for C20/25 (no creep and shrinkage);
- contribution of the concrete calculated with factor: $n = \frac{E_s}{E_{cm}} = \frac{210000}{30000} = 7$;
- the cross-section is cracked and the concrete in tension is neglected.

**TRIAL SECTION**

A deck profile from the previous section with small ribs is reevaluated. The weight of a single deck panel is 42 kg. Other properties of the deck are given in appendix C.

**6.3.2. FIRST CAST UP TO CENTROID OF DECK**

The first cast is up to the height of the centroid of the deck: $h_f = 105$ mm.

Deflection of the deck during the first cast is 9.1 mm. The construction loads are not taken into account during this first cast, because the deflections during the second cast will be governing.

The bending stiffness of the partially steel-concrete composite slab is calculated to determine the deflection during the second cast. The deflection during the second cast is 36.1 mm.

The total deflection during construction (incl. ponding):

$$\delta_0 = \delta_1 + \delta_2 = 9.1 + 36.1 = 45.2$$ mm

The calculation is found in appendix C.1.
6.3.3. **First cast up to height of deck**

The first cast is up to the height of the top of the deck: \( h_f = h_p = 220 \text{ mm} \).

Deflection of the deck during the first cast is 21.6 mm. The construction loads are not taken into account during this first cast, because the deflections during the second cast will be governing.

The bending stiffness of the partially steel-concrete composite slab is calculated to determine the deflection during the second cast. The deflection during the second cast is 17.2 mm.

The total deflection during construction (incl. ponding):

\[
\delta_0 = \delta_1 + \delta_2 = 21.6 + 17.2 = 38.8 \text{ mm}
\]

The calculation is found in appendix C.2.

6.3.4. **Conclusion**

The deflections during construction for an installation with a double cast are calculated for two situations: first cast up to the centroid of the deck and first cast up to the top of the deck. The slab is finished with a second cast. The deflection during construction when the slab is cast in one action is used as reference value: \( \delta_0 = 44.2 \text{ mm (ponding and construction loads included)} \).

When the first cast is up to the centroid of the deck the total deflections during the construction are \( \delta_0 = 45.2 \text{ mm} \). When the first cast is up to the top of the deck the total deflections during the construction are \( \delta_0 = 38.8 \text{ mm} \).

The advantage of a double cast is the extra stiffness of the partially steel-concrete composite floor after the first cast. This first cast is only effective if the concrete in the ribs behave as small concrete beams. Therefore the first cast should be approximately up to the profile height. The trial section, with a steel thickness of 1.25 mm, provides sufficient bending resistance to limit the deflection during construction if the concrete is casted in two stages.

The goal of this study is to design a steel-concrete composite floor with a large unpropped span capacity. An advantage of an unpropped construction is that no temporary supports are required during construction resulting in a high building speed. An installation process with a double cast is very time consuming and does not coincide with the goal of this study. This installation process too complicated and time consuming for common practice.

6.4. **Installation with a fill element**

In this section the installation of the design concept with wide ribs and a fill element is studied. The installation is different from current steel-concrete composite floors due to the use of this fill element. Firstly, other floors that use fill elements for weight reduction are discussed. Secondly, is elaborated on the casting of the concrete and the sequence for installation. Finally, is concluded if the design of the deck with wide ribs and a fill element is possible.

6.4.1. **Fill element**

**Other floors with weight reduction**

Weight reduction in concrete floors is not new. These floor systems are better known as hollow floor systems and design rules and guidelines are covered with help of Eurocode 2. Hollow core slabs have prefabricated hollow cores that reduce weight. Other floors systems with weight reduction, the BubbleDeck, the Airdeck and the OP-deck, have lightweight elements. These examples are illustrated in figure 6.9.

The BubbleDeck floor is a prefabricated concrete slab with lattice girders that is finished at site with a concrete layer. The self-weight is reduced with 35% due to air filled plastic spheres that are enclosed in the concrete between the reinforcement. The prefabricated concrete slabs need temporary support during the construction of the floor. The BubbleDeck floor is available in different slab heights and can span between 6 and 14
m. A floor with an height of 280 mm weighs 4.6 kN/m$^2$ and can span between 7 and 8 m. The spheres in this floor have a diameter of 225 mm [33].

The Airdeck floor is a prefabricated concrete slab with lattice girders and air filled plastic boxes that is finished at site with a concrete layer. These air filled plastic boxes are pressed 10 mm into the prefabricated concrete. The self-weight is reduced with 25-30% due to these boxes. The prefabricated concrete slabs need temporary support during the construction of the floor. A floor with a height of 280 mm can span up to 7.5 m. Here have the air filled boxes height of 180 mm [34].

The OP-deck is build-up from a sandwich panel with PIR insulation, lightweight EPS elements and in-situ cast concrete. The cast concrete forms a T-rib floor, where the ribs have a c.t.c. distance of 333 mm. In the OP-deck no composite behaviour is present between the steel top skin of the sandwich panel and the concrete. The sandwich panel is used as temporary shuttering and has no structural function once the concrete is hardened. However, it provides insulation at the bottom of the floor. Therefore the OP-deck is suitable for specific situations where floor insulation is required: parking roofs, cantilevers, insulated roofs or ground floors, etc. During construction no temporary supports are required if the span is limited to 5.5 m [35].

**Fill element**

The Bubbledeck and Airdeck are two floor systems that carry loads in two directions. The shape of the small elements (plastic balls and boxes) create concrete ribs in two directions similar to a cassette floor. The OP-deck floor system carries loads in one direction. The long EPS blocks create concrete ribs in one direction similar to a T-rib floor. The shape of the weight reducing element should fit the type of load carrying system.

The steel-concrete composite floor designed in this section carries the loads mainly in one direction parallel to the direction of the ribs. The weight reducing elements should therefore be orientated in this direction. This is comparable to the EPS elements used in the OP-deck. Another element that might be suitable is a plastic tube.

### 6.4.2. Concrete part

**Casting of concrete**

The distance between the lightweight element and the deck ($b_c$) should have a minimum width such that the concrete can be placed and compacted. The minimum distance of $b_c$ is based on the minimum bar spacing.

$$b_c = \min(k_1 \sigma; d_g + k_2; 20 \text{ mm}) = \min(1 \cdot 16; 32 + 5; 20) = 37 \text{ mm}$$

Where $k_1 = 1$ and $k_2 = 5$ [36, p.19]. The maximum aggregate size $d_g$ is assumed as 32 mm. Therefore should $b_c$ at least be 37 mm (figure 6.10a). In figure 6.10b is shown that the assumed weight reduction of section 6.2 is overestimated.

Besides that, it is expected that the designs in figure 6.10b requires a difficult installation process with at least three casting stages. First the bottom part of the rib with the tension reinforcement is cast, then the fill element is placed and the ribs are cast, finally the top reinforcement is installed and the top slab is cast.

(a) Bubbledeck  (b) Airdeck  (c) OP-deck

Figure 6.9: Principle of floor systems with weight reducing elements
6.4. INSTALLATION WITH A FILL ELEMENT

(a) Space for fill element
(b) Result of $h_c$ on possible weight reduction

Figure 6.10: Casting of concrete with fill element

Figure 6.11: Position of the reinforcement: at the support (left) and in the cross-section (right)

COVER FOR REINFORCEMENT

The height of the concrete layer on top of the steel deck $h_c$ largely depends on the required reinforcement in the concrete top slab (figure 6.11). In the previous section is assumed that $h_c = 60$ mm. In case of a reinforcement mesh of ø8 and extra tension reinforcement at hogging moment region ø20:

$$c_{nom} = c_{min} + \Delta_{dev} = 20 + 5 = 25 \text{ mm}$$
$$h_{c,min} = \phi8 + \phi20 + c_{nom} = 8 + 20 + 25 = 53 \text{ mm}$$

Where $c_{nom}$ is the cover between the top surface of the slab and the top reinforcement. Therefore $h_c = 60$ mm is valid. The width of the bottom flange is determined by the required cover for the tensile reinforcement. In case of a tensile reinforcement bar of ø16.

$$c_{nom} = c_{min} = 16 \text{ mm}$$
$$b_{u,min} = \phi16 + 2c_{nom} = 16 + 2 \cdot 16 = 48 \text{ mm}$$

Here is no allowance taken for deviation ($\Delta_{dev}$), because the tensile bar can be placed very accurate in the rib. The width of the bottom flange should at least be 48 mm.

6.4.3. DOUBLE CAST WITH FILL ELEMENT

Figure 6.10 illustrates that there is only little space for a fill element in the ribs if this element is fully surrounded by concrete. The boundaries are determined by the required cover for the tension reinforcement, the required width to cast the concrete along the lightweight element, and the required cover for the top reinforcement.

In the section 6.2 the deck is designed assuming a weight reduction of 37%. However, it is only possible to reduce up to 16% with an EPS element or 10% with a tube (figure 6.12). Therefore the required weight reduction is not possible within this design concept.
A different design approach, where the fill element is not surrounded by concrete, but placed on top of the first cast and the slab is finished with a second cast, is illustrated in figure 6.13. Here is sufficient weight reduction possible and this design concept is more practical regarding the installation process. Besides that, the first cast is only in the bottom part of the ribs, which allows for a prefabricated variant.

The concrete of the first cast is located in the tension zone and will not contribute to a higher bending stiffness during the second cast (in contrary to the double cast of the previous section).

This design could provide an answer on the main question. However, the finished floor will not behave as current steel-concrete composite floor slabs and there are critical aspects that require attention before any further conclusions can be drawn.

6.4.4. CONCLUSION

The installation of the design concept with wide ribs and a fill element studied. It is found that a EPS element or plastic tube is suitable to reduce weight in the ribs of the deck. However, due to the casting of concrete is less weight reduction possible that initially expected. Besides that is the installation process not practical. The design with a fill element in the ribs surrounded by concrete is therefore not possible.

A different design approach, where the fill element is not surrounded by concrete, but are placed on top of the first cast and the slab is finished with a second cast, is a good design solution. Here is sufficient weight reduction possible and this design is more practical regarding the installation process. However, the finished floor will not behave as current steel-concrete composite slab and there are critical aspects that require attention before any further conclusions can be drawn.

6.5. DECK DESIGN FOR A DOUBLE CAST WITH FILL ELEMENT

This section elaborates on the deck design with a double cast and fill element. The design concept of this floor is illustrated and the critical design aspects are indicated. Besides that, a trail section is presented used for further studies.
6.5. DECK DESIGN FOR A DOUBLE CAST WITH FILL ELEMENT

6.5.1. DESIGN CONCEPT

The design concept in cross-section is illustrated in figure 6.14. The different elements have the following function in the design:

- concrete top flange and bottom flange of the steel deck → resist the bending moment;
- the webs of the deck → resist vertical- and longitudinal shear forces;  
- steel-concrete interface → resist longitudinal shear forces;
- transverse stiffeners in the web → improve shear buckling resistance of the thin-walled steel webs;
- longitudinal stiffeners in the web → improve bending resistance;
- tension reinforcement in bottom of ribs → take up tension forces during fire.

In this study is assumed that the fill element has no structural function. Besides that, the reinforcement bar in the bottom of the ribs is only considered to determine the resistance of the floor during the situation of fire and not during service life (room temperature).

6.5.2. CRITICAL ASPECTS

The structural behaviour of this floor slab is different from current steel-concrete composite slabs, because the webs of the deck are not in contact with the concrete (figure 6.14). Three critical aspects need further investigation to determine if this design is feasible.

- thin-walled steel web of the deck: should transfer vertical shear forces and longitudinal shear forces, and is susceptible to buckling (chapter 7);
- steel-concrete interface: steel top flange (and a part of the web) should transfer longitudinal shear forces to the concrete (chapter 8);
during fire: steel deck is expected to heat up quick and to become inefficient, the reinforcement in the bottom part of the ribs should take up tension forces (chapter 9).

The following chapters elaborate on these critical aspects.

6.5.3. DECK DESIGN

Here the cross-sectional properties and loads are given for the design concept with a double cast and fill element used for further studies.

**TRAIL SECTION**

The dimensions of the deck profile are illustrated in figure 6.15.

**Dimensions of the deck profile**

- Thickness of the steel sheet: \( t = 1.25 \text{ mm} \)
- Height of the deck: \( h_p = 220 \text{ mm} \)
- Breadth of a single deck panel: \( b_d = 288 \text{ mm} \)

**Properties of the cross-section**

- Gross area: \( A_p = 3159 \text{ mm}^2/\text{m} \)
- Effective area: \( A_{pe} = 2334 \text{ mm}^2/\text{m} \)
- Second moment of area: \( I_{eff} = 1957 \cdot 10^4 \text{ mm}^4/\text{m} \)

**Properties of the floor slab with weight reduction**

- Height of the concrete top slab: \( h_c = 60 \text{ mm} \)
- Height of the floor slab: \( h = 280 \text{ mm} \)
- Self-weight of the floor slab: \( G = 300 \text{ kg/m}^2 \)
- Extra weight due to the effect of ponding: \( G_{ponding} = 70 \text{ kg/m}^2 \)

**LOADS**

Loads during service life are based on a slab self-weight of approximately 370 kg/m\(^2\) (ponding is included).

**Permanent actions**

- Composite slab (ponding included): \( g_k = 3.6 \text{ kN/m}^2 \)

**Variable actions**

- Imposed floor load (uniform): \( q_k = 4.0 \text{ kN/m}^2 \)
- Imposed floor load (concentrated): \( Q_k^{1) } = 7.0 \text{ kN} \)

1) *Point load* \( Q_k \) is not governing for \( M \) and \( V \). Only if the point load is positioned close to the support and the load spread between the ribs is assumed to be minimal. This load case is not considered in this study.

In serviceability limit state (SLS) the design load is \( q_d = 1.0 \cdot 3.6 + 1.0 \cdot 4 = 7.6 \text{ kN/m}^2 \) and in ultimate limit state (ULS) the design load is \( q_d = 1.2 \cdot 3.6 + 1.5 \cdot 4 = 10.4 \text{ kN/m}^2 \).
The moment \((M_{Ed})\) and shear force \((V_{Ed})\) per meter width in ULS:

\[
M_{Ed} = \frac{1}{8} q_d L^2 = \frac{1}{8} \cdot 9.3 \cdot 7.2^2 = 67.4 \text{ kNm/m}
\]

\[
V_{Ed} = \frac{1}{2} q_d L = \frac{1}{2} \cdot 10.4 \cdot 7.2 = 37.5 \text{ kN/m}
\]

**Installation**

The design with a double cast and fill element could be fabricated and installed in two ways: in-situ or pre-fabricated. The in-situ installation is more flexible, but requires a double cast at site. A deck profile that would suit both is preferable. In chapter 10 the deck profile is redesigned and verified during construction. In chapter 11 the structural behaviour of the steel-concrete composite slab is discussed and the strength and stiffness criteria verified according to the Eurocode.

**6.6. Conclusion**

In this chapter two design concepts are studied: a deck design with small ribs, and a deck design with wide ribs and a fill element in these wide ribs.

It is found that the deck design with small ribs do not meet the requirements, because the decks are heavy and cannot be man-handled. A double cast, where the concrete is cast in two stages, can provide extra stiffness during the installation of the floor allowing for a lighter deck profile. However, this installation process too complicated and time consuming for common practice.

The deck of the design with wide ribs and fill elements in the concrete ribs meets all criteria. However, the installation process of a fill element in the ribs that is surrounded by concrete is not practical and sufficient weight reduction is not possible.

A different design approach, where the fill element is placed on top of the first cast and the slab is finished with a second cast, is promising. The finished floor slab will not behave as current steel-concrete concrete slabs and three different aspects need to be studied to determine if the design is feasible: the shear resistance of the deck, the steel-concrete composite connection and the design during fire. In the following chapters is elaborated on this design.

- chapter 7: design of the deck to resist shear
- chapter 8: design of the shear connector
- chapter 9: design of the floor slab during fire
- chapter 10: design of the steel deck
- chapter 11: design of the steel-concrete composite floor slab
PART 3

THE PRODUCT
DESIGN OF THE WEB TO RESIST SHEAR

In the design of the floor slab the fill element is assumed to have no structural function. For this reason, the web of the deck should have the capacity to transfer vertical- and longitudinal shear forces in the floor. This chapter elaborates on the design of the web to resist shear. The tension field method is used to determine the post-buckling strength of the web in a truss model. In this model the compression struts are formed by the transverse stiffeners (web embossments) and the tensile ties are formed by the tension fields in the web. Firstly, the tension field method is elaborated on as model. Secondly, the capacity of the tensile tie and the compression strut is calculated. Finally, a conclusion is made if the web can be designed to resist shear.

7.1. TENSION FIELD METHOD

Thin-walled stiffened webs are also used in plate girders, where two flanges are welded to a thin-walled web to form the plate girder. It is known that the thin-walled webs in plate girders are susceptible to shear buckle. This behaviour occurs often near the supports, where the shear loads are most severe. Studies prove that the post-buckling strength of the thin-walled plate girders is much greater than the strength prior to buckling. This additional strength is the result of a tension field that is formed in the web. A new load-carrying mechanism occurs that is very similar to a truss beam: the chords are formed by the flanges, the ties are formed by the tension field and the struts are formed by transverse stiffeners. These transverse stiffeners are welded to the web and help to prevent buckling of the compression strut. Basler (1965) was the first to study this behaviour and presented a method to calculate the post-buckling strength of plate girders.

7.1.1. MODEL

A design approach is used in which the thin-walled webs of the deck behave similar to the webs in plate girders and the theory of Basler is applicable (figure 7.1).

![Shear buckling of thin-walled web](image1)

![Tension fields](image2)

![Truss analogy](image3)

Figure 7.1: Tension field method to determine the post-buckling strength of the web of the deck
Here is assumed that:

- the thin-walled parts of the web that are not stiffened will shear buckle at a low shear force;
- the embossments in the web behave as transverse stiffeners;
- a tension field is formed between the transverse stiffeners.

The tension field method for plate girders given in Eurocode 3 part 5 is applicable for $1.0 \leq \frac{a}{d} \leq 3.0$, where $a$ is the spacing between the transverse stiffeners and $d$ is the height of the transverse stiffener [13].

The embossments in current decks have a width of 17 mm, depth of 2 mm and a c.t.c spacing of 38 mm. The embossments are designed to prevent buckling of the web and provide steel-concrete composite behaviour (figure 7.2). The small c.t.c. distance of the embossments is required to ensure sufficient steel-concrete composite behaviour.

![Figure 7.2: Cross-section of the embossments used in current deep decks (Comflor 210 and 225)](image)

The embossments in this design are not in contact with the concrete and are therefore not used to provide the steel-concrete connection. The c.t.c. distance is therefore designed to allow a tension field to be formed. The spacing of the stiffeners $a$ is at least equal to the height of the web $d$ as illustrated in figure 7.3.

![Figure 7.3: View and cross-sections of the web](image)

**7.1.2. LOADS**

Shear forces in the web are illustrated in figure 7.4. The distribution of the vertical shear forces over the span is illustrated in figure 7.4a.

In figure 7.5 the vertical- and longitudinal shear forces are determined in truss model for the web of the deck. For simplicity the load $q$ is equally divided in point loads $F$ that are uniformly distributed over the floor span. The compression flange of the truss is formed by the top of the deck and the concrete top slab. The tension flange of the truss is formed by the bottom of the deck. Both are not verified in this chapter.
7.1. TENSION FIELD METHOD

(a) Shear forces during service life: in a single web over the length of the span (left) and diagram that illustrates these forces (right)

(b) Shear forces in the thin-walled web

Figure 7.4: Longitudinal- and vertical shear forces

From figure 7.5 is obtained that the maximum vertical shear force $V_{Ed}$ at the support is 5.4 kN/web and the maximum tension force $N_{Ed}$ in the tension field is 7.2 kN/web.

No weight reduction is applied near the support and the ribs are completely filled with concrete. Weight reduction is applied at a distance of 600 mm from the supports. The concrete in the ribs is parted here and the web should take up the shear forces. The stiffness of the fully cast cross-section is much larger than the stiffness of cross-section with fill element. The expectation is that the full cross-section near the support will transfer the shear forces (appendix H.1). Therefore, the maximum design shear force that should be resisted by the web of the deck is (figure 7.4a):

$$V_{Ed} = 4908 \text{ N/web}$$

7.1.3. CONCLUSION

According to the tension field method it is possible to determine the load-carrying capacity of the web with a truss model. In this model the embossments (transverse stiffeners) are the compression struts and the tension fields are the tensile ties. The capacity of struts and ties should be calculated to determine if the truss model has sufficient capacity to resist the shear forces.
7.2. **Tension field and capacity of the tensile tie**

The capacity of the tension field is determined to obtain the design resistance of the tensile tie. The formation of the tension field in the web of the deck is determined according to the theory of Basler as illustrated in figure 7.6.

Most conservative is to assume that the tension field is formed under an angle of $22.5^\circ$: \( \phi = \frac{\theta}{2} = \frac{45^\circ}{2} = 22.5^\circ \), where \( \theta = \arctan\left(\frac{d}{a}\right) \) (figure 7.7). The width of the tension field is found as:

\[
g = \frac{d}{2} \cdot \cos(\theta) = \frac{220}{2} \cdot \cos(22.5^\circ) = 101 \text{ mm}
\]

The capacity of the tension field is:

\[
N_{tie} = g f_{yb} / \gamma_M = 101 \cdot 1.21 \cdot 350 / 1 \cdot 10^{-3} = 42.7 \text{ kN}
\]

The tensile tie is able to transfer a vertical force of 19.1 kN and a horizontal force of 38.1 kN.

The capacity of the tension field is more than sufficient to transfer the shear forces. Note that the width of the tension field formed under a load of 5 kN is much smaller than illustrated in figure 7.7.

It seems that the tension field has sufficient capacity. However, it is too early to draw conclusions. It is unclear how the actual tension field is formed. The formation of the tension field is influenced by the interaction...
between the shear buckling of the web and the bending stiffness and position of the transverse stiffeners. Besides that, the tension field could influence the behaviour of the embossment as transverse stiffener. This is elaborated on in section 7.4.

7.3. TRANSVERSE STIFFENER AND CAPACITY OF THE COMPRESSION STRUT

The capacity of the web at position of the transverse stiffener is determined to obtain the design resistance of the compression strut. The web of the deck, supported in one direction by the concrete and the fill element, can buckle in the outward plane.

A smaller transverse stiffener (embossement in the web) will provide more bending stiffness of the deck, because the web embossments act as a harmonica if the deck is subjected to bending and will not contribute to the bending stiffness of the deck. Therefore, the aim of this section is to design the minimum height of the transverse stiffener that provides sufficient buckling strength \( N_{b,Rd} \) to resist the shear forces.

First the buckling model is described and the cross-sectional properties are calculated. Then the Euler-buckling force \( N_{cr} \) is obtained by solving the differential equation (DE) for this buckling problem. After that the design buckling strength \( N_{b,Rd} \) is calculated, where imperfections are taken into account with the net-section properties, the Euler-buckling force and the appropriate buckling curve.

7.3.1. MODEL

The compression strut is modelled as a small buckling column (left figure 7.8). The column can fail due to local- or overall buckling. An impression of the interaction between these two failure modes is illustrated in figure 7.8 (right).

The cross-section of the web is divided in a stiffened- and an unstiffened part, respectively \( E_1I_1 \) and \( E_2I_2 \), shown in figure 7.9 (right). It is expected that the bending stiffness of the unstiffened part is much lower than the stiffened part. The bending stiffness of the stiffened- and unstiffened part are taken into account in the buckling model.

The system length of the buckling column is equal to the height of the web, because the whole web can buckle, \( l_0 = h_p = 220 \, \text{mm} \). Besides that, the buckling column is assumed to be pin-joined at both ends. The buckling length is therefore equal to the system length, \( l_{cr} = l = 220 \, \text{mm} \). The effect of the longitudinal stiffeners and top- and bottom flange on the buckling length is elaborated on in section 7.3.7.

7.3.2. SECTION PROPERTIES

The stiffeners have a width of 17 mm, are designed to be 3 mm in depth, and have a c.t.c. distance of \( a = d = 220 \, \text{mm} \). The cross-sections dimensions are given for the stiffened (1) and unstiffened parts (2) in figure 7.10.
7. Design of the web to resist shear

Figure 7.8: Stiffener modelled as small buckling column (left) and interaction between local- and overall buckling failure (right)

Figure 7.9: View of web (left), cross-section of web, and buckling model (right)
7.3. TRANSVERSE STIFFENER AND CAPACITY OF THE COMPRESSION STRUT

The area- and second moment of area of gross section of the stiffener are:

\[ A_{s,1} = 273.4 \text{ mm}^2 \]
\[ I_{s,1} = 220.1 \text{ mm}^4 \]

The effective cross-section is obtained according to Eurocode 3 part 3 and 5 to take local buckling into account [5, 13].

\[ A_{eff,1} = 160.8 \text{ mm}^2 \]

A detailed calculation is given in appendix D.1.

The area- and second moment of area of the gross section of the unstiffened part are:

\[ A_{s,2} = d \cdot t = 220 \cdot 1.21 = 266.2 \text{ mm}^2 \]
\[ I_{s,2} = \frac{1}{12} b t^3 = \frac{1}{12} 220 \cdot 1.21^3 = 32.4 \text{ mm}^4 \]

It is expected that the compression force from the transverse stiffener is introduced under an angle of 45° in the unstiffened part of the web (figure 7.11). It is safe to assume that the width and thickness of the unstiffened parts are equal the effective width and thickness of the stiffener (appendix D.1).

The width of the unstiffened part is:

\[ b_{om} = b_t + b_{1,e2} + b_{2,e1} \leq \text{ c.i.c. distance between the transverse stiffeners} \]
\[ b_{om} = 17 + 55.3 + 55.3 = 127.3 \leq 220 \text{ mm} \]

The effective thickness of the unstiffened part is:

\[ t = t_{red} = 1.20 \text{ mm} \]

The effective width of the unstiffened part is obtained with the expressions for an internal plane element.
under uniform compression, where $\sigma_k = 4$, $\Psi = 1$, $\bar{b} = 127.3$, and $t = 1.20$ [13, p.18-21].

\[
\lambda_p = \frac{\bar{b}/t}{28.4\sqrt{\lambda_{cr}}} = \frac{127.3/1.20}{28.4\sqrt{235/350\sqrt{4}}} = 0.998
\]

\[
\rho = \frac{\lambda_p - 0.055(3 + \Psi)}{\lambda_p^2} = \frac{0.998 - 0.055(3 + 1)}{0.998^2} = 0.781
\]

\[
b_{eff} = \rho \cdot \bar{b} = 0.781 \cdot 127.3 = 99.6 \text{ mm}
\]

\[
A_{eff,2} = b_{eff} \cdot t_{red} = 99.6 \cdot 1.20 = 120.0 \text{ mm}^2
\]

### 7.3.3. Differential Equation

The Euler-buckling force $N_{cr}$ is obtained by solving the differential equation (DE) for this buckling problem with the right boundary and matching conditions. Parameters of the problem are (figure 7.12):

- bending stiffnesses: $EI_1$ and $EI_2$ (Nmm$^2$);
- height of the web or length of the buckling column: $L$ (mm);
- length of the embossed parts: $l$ (mm);
- buckling load: $N_{cr}$ (N).
7.3. TRANSVERSE STIFFENER AND CAPACITY OF THE COMPRESSION STRUT

Derivation of the differential equation

The differential equation (DE) follows from the equilibrium of a small part of the bar with an initial deflection. The equilibrium of the internal forces provide a 4th order DE in displacement \( w \). The buckling equation is:

\[
EIw'''' + \frac{F}{EI}w'' = 0
\]

(7.1)

Here is assumed that the normal force is constant over the length and that no uniform force acts on the bar, respectively \( q_x = 0 \) and \( q_z = 0 \). Where \( EI \) is the bending stiffness of the bar and \( F \) the buckling force. The derivation of the DE is found in appendix D.2.

Solving the differential equation

The 4th order homogeneous DE is rewritten:

\[
w'''' + \alpha^2 w'' = 0, \quad \text{with} \quad \alpha^2 = \frac{F}{EI}
\]

Substitution of the solution \( w(x) = e^{tx} \) gives the characteristic equation:

\[
t^4 + \alpha^2 t^2 = 0
\]

The solution of this equation has four roots:

\[
t_{1,2} = \pm \alpha i \quad \text{and} \quad t_{3,4} = 0
\]

General solution for the buckling equation:

\[
w(x) = C_1 + C_2 x + C_3 \sin(\alpha x) + C_4 \cos(\alpha x)
\]

(7.2)

The constants are solved with the boundary- and matching conditions. Each edge provides two boundary conditions (BC) and each field provides four matching conditions (MC). The BC and MC are expressed in \( w \) with the following relations for \( \varphi \), \( M \), and \( S_z \).

\[
\begin{align*}
\varphi &= -w' = -C_2 - C_3 \alpha \cos(\alpha x) + C_4 \alpha \sin(\alpha x) \\
M &= -EIw'' = \alpha^2 EIC_3 \sin(\alpha x) + \alpha^2 EIC_4 \cos(\alpha x)
\end{align*}
\]

(7.3)

Moment equilibrium provides:

\[
S_z = M' + S_x w' = -EIw'''' - \alpha^2 EIw'
\]

\[
= -\alpha^3 EIC_3 \cos(\alpha x) + \alpha^3 EIC_4 \sin(\alpha x) + \alpha^2 EIC_2 + \alpha^3 EIC_3 \cos(\alpha x) - \alpha^2 EIC_4 \sin(\alpha x)
\]

\[
= \alpha^2 EIC_2 = FC_2
\]

(7.4)
**Solution for a bar with a different $EI$**

In this buckling problem the bar is divided in three parts and thus three DE’s are required to solve this problem (figure 7.12).

\[
\begin{align*}
  w_1(x) &= C_1 + C_2 x + C_3 \sin(ax) + C_4 \cos(ax) & \text{for} & \quad 0 < x < l_1 \text{ with } EI_2 \\
  w_2(x) &= C_5 + C_6 x + C_7 \sin(ax) + C_8 \cos(ax) & \text{for} & \quad l_1 < x < l_2 \text{ with } EI_1 \\
  w_3(x) &= C_9 + C_{10} x + C_{11} \sin(ax) + C_{12} \cos(ax) & \text{for} & \quad l_2 < x < L \text{ with } EI_2
\end{align*}
\]

Here are due to symmetry of the embossment in the web:

- $l_1 = \frac{L - l}{2}$
- $l_2 = l_1 + l$

**Boundary- and matching conditions:**

- $x = 0: \quad w_1 = 0 \text{ and } M_1 = 0$
- $x = l_1: \quad w_1 - w_2 = 0, \quad \varphi_1 - \varphi_2 = 0, \quad M_1 - M_2 = 0 \text{ and } S_{z,1} - S_{z,2} = 0$
- $x = l_2: \quad w_2 - w_3 = 0, \quad \varphi_2 - \varphi_3 = 0, \quad M_2 - M_3 = 0 \text{ and } S_{z,2} - S_{z,3} = 0$
- $x = L: \quad w_3 = 0 \text{ and } M_3 = 0$

**Determine the buckling load**

The buckling problem is now described with:

- 12 equations: 4 from BC and 8 from MC (homogeneous equations);
- 12 unknown integration constants ($C_1, C_2, \ldots, C_{12}$);
- parameters: bending stiffness $EI_1$ and $EI_2$, and lengths $L$ and $l$ (figure 7.12);
- one variable: buckling load $F$.

The 12 equations are written in Matrix notation:

\[
A\tilde{x} = 0
\]

Where:

- $A$ is the matrix with a all coefficients (contains $F$);
- $\tilde{x}$ the vector with the integration constants ($C_1, C_2, \ldots, C_{12}$);
- $0$ the null vector.

This system of homogeneous equations has only a non-trivial solution if the determinant of $A$ is equal to zero, $\det(A) = 0$. This is possible for a certain value of $F$. The integration constants in $\tilde{x}$ remain unsolved. The buckling load $N_{cr}$ is found for the lowest value of $F$, where $F \neq 0$. $\det(A)$ is solved in Maple, due to the size of this matrix. The Maple sheet is found in appendix D.3.

### 7.3.4. Euler buckling load

The buckling load $N_{cr}$ is found with the solution of the buckling equation. The buckling model for the design of the web is illustrated in figure 7.13. The following parameters are used to obtain the buckling load.

- $EI_1 = 2.1 \cdot 10^5 \cdot 220.1 = 46221000 \text{ Nmm}^2$;
- $EI_2 = 2.1 \cdot 10^5 \cdot 32.4 = 6804000 \text{ Nmm}^2$;
- $l = 170 \text{ mm}$;
- $L = 220 \text{ mm}$.
The gross-section properties are used to determine the bending stiffness of the cross-sections. Overall buckling is considered and therefore it is allowed to use the gross sectional properties. Note that Eurocode 3 part 3 uses the effective cross-section to include the effects of local buckling.

This system of equations is solved in Maple according to the method above. The plot in figure 7.13 shows the function of the determinant $A$ for $F$. The buckling load $N_{cr}$ is the lowest value of $F$ for which the function is 0. The buckling load for this design is:

$$N_{cr} = 8323 \text{ N}$$

The solution is verified for an lower and upper boundary. The bending stiffness of the bar is here assumed to be independent of $x$ and therefore the Euler-buckling formula applies [37, p.127 and 146].

$$F_{cr} = \frac{\pi^2 EI}{l_{cr}^2}$$

The lower boundary is found with $EI = EI_2 = 6804000 \text{ Nmm}^2$ and $l_{cr} = L = 220 \text{ mm}$:

$$F_{cr} = \frac{\pi^2 EI}{l_{cr}^2} = \frac{\pi^2 6804000}{220^2} = 1387 \text{ N}$$

The upper boundary is found with $EI = EI_1 = 46221000 \text{ Nmm}^2$ and $l_{cr} = L = 220 \text{ mm}$:

$$F_{cr} = \frac{\pi^2 EI}{l_{cr}^2} = \frac{\pi^2 46221000}{220^2} = 9425 \text{ N}$$

In figure 7.14 the same values are found by solving the DE. The buckling load found for a bar with a varying bending stiffness over $x$ is within the lower- and upper boundaries that are given above. Therefore the outcome of the solution found in Maple is valid.

### 7.3.5. Design Buckling Load

The design buckling load $N_{b,Rd}$ is determined by taking the imperfections into account using the net-section properties, the Euler-buckling force and the appropriate buckling curve ($a_0$, $a$, $b$, $c$, or $d$). Eurocode 3 part 1 provides the expressions to determine the design buckling load [5, p.56-57].
The reduction factor $\kappa$ to determine $N_{b,Rd}$ is calculated with the non-dimension slenderness of the column and the appropriate buckling curve.

$$\kappa = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}}$$

$$N_{b,Rd} = \kappa A_{eff} f_y / \gamma_M$$

The non-dimensional slenderness $\bar{\lambda}$ for class 4 cross-sections:

$$\bar{\lambda} = \sqrt{\frac{A_{eff} f_y b}{N_{cr}}}$$

Here is $A_{eff} = A_{s,red}$. The appropriate buckling curve for the cross-sections of the web is buckling curve $c$ (figure 7.15). This curve is also described with the following empirical formula to describe the relation between $\kappa$ and $\bar{\lambda}$. Factor $a$ is 0.49 for buckling curve $c$.

$$\Phi = 0.5 \left[ 1 + a(\bar{\lambda} - 0.2) + \bar{\lambda}^2 \right]$$

The design buckling load $N_{b,Rd}$ depends on the effective cross-sectional area $A_{eff}$ and the buckling load $N_{cr}$ as shown in figure 7.16. The design buckling load should meet the following requirement: $N_{b,Rd} > N_{Ed} = 4908$ N.

$A_{eff}$ is not constant over the cross-section. $A_{eff}$ is determined for the stiffened part $A_{eff,1}$ and the unstiffened part $A_{eff,2}$ of the web. A safe design assumption is to use the minimum value of $A_{eff,1,2}$ over the whole
7.3. Transverse Stiffener and Capacity of the Compression Strut

Figure 7.16: Diagram of relation between $N_{cr}$ and $A_{eff}$ for $N_{b,Rd} = 4908$ N

The length of the web.

$$A_{eff} = \min (A_{eff,1}; A_{eff,2}) = \min (160.8; 120.0) = 120.0 \text{ mm}^2$$

The required Euler buckling resistance $N_{cr}$ is obtained for $N_{Ed} = 4908$ N and $A_{eff} = 120.0 \text{ mm}^2$. The required Euler buckling resistance to provide sufficient design buckling resistance is:

$$N_{cr} \geq 5375 \text{ N}$$

7.3.6. Design of the Web

The height of the embossment is designed for $N_{cr} \geq 5375$ N to prevent failure due to overall buckling. The minimum height of the embossment is 131 mm. This provides a Euler buckling resistance of $N_{cr} = 5400$ N. Chosen is to apply a web embossment of 150 mm that provides a Euler buckling resistance of $N_{cr} = 6844$ N and a design buckling resistance of $N_{b,Rd} = 5655$ N.

Unity check for overall buckling:

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{4908}{5655} = 0.86$$

The design resistance for local buckling is:

$$N_d = A_{s,red} f_{yb} = 120 \cdot 350 \cdot 10^{-3} = 42 \text{ kN} \gg N_{Ed} = 4.9 \text{ kN}$$

Note that local buckling is not governing. The design of the web is illustrated in figure 7.17.

Figure 7.17: Design of the web with an embossment of 150 mm
7.3.7. **TRANSLATIONAL- AND ROTATIONAL STIFFNESS**

Web buckling is affected by the longitudinal stiffeners in the web and the top- and bottom flange. The longitudinal stiffeners continues over the length of the span and form a link between the transverse stiffeners. This means that each transverse stiffener is supported by its neighboring stiffeners. This effect is modelled by introducing a translational stiffness $K$ in the model at position of the longitudinal stiffener. The web is fixed between the top and bottom flange of the deck. Both provide a rotational stiffness if the web tends to buckle outward. This effect is modelled by introducing a rotational stiffness $C_{\theta}$ in the model at the top- and bottom of the web.

The buckling model is shown in figure 7.18. This section elaborates on the effect of $K$ and $C_{\theta}$ on the Euler buckling resistance $N_{cr}$.

![Figure 7.18: Cross-section of the web (left), deformation of the web, and buckling model (right)](image)

**TRANSLATION STIFFNESS**

Eurocode 3 part 3 gives an expression to determine the translational stiffness $K$ (figure 7.19). The deflection $\delta$ under a unit load $u = 1$ is:

$$\delta = \frac{ub_1^2b_2^2}{3(b_1 + b_2)} \cdot \frac{12(1 - \nu^2)}{E\ell^3} = \frac{1 \cdot 25^2 \cdot 195^2}{3(25 + 195)} \cdot \frac{12(1 - 0.3^2)}{2.1 \cdot 10^5 \cdot 1.21^3} = 1.057 \text{ mm}$$

Where $\nu = 0.3$ the Poisson’s ratio in the elastic range. Here is assumed that $C_{\theta,1} = 0$ and $C_{\theta,2} = 0$. The translational stiffness is:

$$K_{1,2} = \frac{u}{\delta} = \frac{1}{1.057} = 0.946 \text{ N/mm}$$

Due to symmetry of the web holds that $K_1 = K_2$. Stiffness $K_{1,2}$ is modelled by changing the matching conditions for the vertical force equilibrium in the buckling model (figure 7.20). The vertical force equilibrium at

![Figure 7.19: Actual system (left) and equivalent system (right) [5, p.26-27]](image)
The Euler buckling resistance is determined with Maple where the value $K$ is adjusted (also see appendix D.3). In table 7.1 the Euler buckling resistance is given for three values of $K$.

The effect of $K = 0.946$ on the Euler buckling resistance is negligible. However, $K$ is here obtained without considering that the transverse web stiffener is also supported by the neighboring stiffeners. An infinitely stiff translational stiffness would increase the buckling resistance $N_{cr}$ of the web significantly. Therefore it is expected that the support of the neighboring stiffeners can contribute to increase the buckling resistance.

<table>
<thead>
<tr>
<th>$K_{1,2}$ (N/mm)</th>
<th>$N_{cr}$ (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>6844.0</td>
</tr>
<tr>
<td>0.946</td>
<td>6870.5</td>
</tr>
<tr>
<td>$\infty$</td>
<td>28519.7</td>
</tr>
</tbody>
</table>

Table 7.1: Effect of the translational stiffness $K_{1,2}$ on the Euler buckling resistance $N_{cr}$

**Rotational stiffness**

A simple mechanical model is used to determine the rotational stiffness $C_{\theta}$ assuming that the angle between the web and the flanges remains fixed (figure 7.21). The rotational stiffness depends on the width $l$ and the bending stiffness $EI$ of the flange. Given is that:

- effective width: $b = 220$ mm;
- top flange: $l_1 = 60$ mm;
- bottom flange: $l_2 = 120$ mm.

Note that the effective width of the flange is equal to the c.t.c. distance of the transverse stiffeners in the web.

$I = \frac{1}{12} b l^3 = \frac{1}{12} 220 \cdot 60^3 = 32.5 \text{ mm}^4$

$EI = 2.1 \cdot 10^5 \cdot 32.5 = 6820510 \text{ Nmm}^2$

The rotational stiffness is:

$C_{\theta} = \frac{T}{\theta}$

$C_{\theta,1} = \frac{T}{\frac{T_{fl}}{\frac{EI}{3}}} = \frac{1}{\frac{400}{3\cdot 6820510}} = 94.729 \text{ Nmm/rad}$

$C_{\theta,2} = \frac{T}{\frac{T_{fl}}{\frac{EI}{3}}} = \frac{1}{\frac{1290}{3\cdot 6820510}} = 11.841 \text{ Nmm/rad}$
The rotational stiffness $C_{\theta,1,2}$ is modelled by changing the boundary conditions of the buckling model.

\[
M_1 = -C_{\theta,1}\varphi_1 \\
M_2 = -C_{\theta,2}\varphi_2
\]

The Euler buckling resistance is determined with Maple where the value $C_{\theta,1}$ and $C_{\theta,2}$ are adjusted (see also appendix D.3). The buckling resistance is given in table 7.2 for three values of $C_{\theta,1,2}$.

The effect of the rotational stiffness for $C_{\theta,1} = 94.7$ and $C_{\theta,1} = 11.8$ on the Euler buckling resistance is negligible. However, here is not taken into account that the top- and bottom flange are restrained by concrete. An infinitely stiff rotational stiffness would increase the buckling resistance $N_{cr}$ of the web significantly. It is expected that the top- and bottom flange, if restrained by the concrete, can contribute to increase the buckling resistance.

<table>
<thead>
<tr>
<th>$C_{\theta,1}$ (Nmm/rad)</th>
<th>$C_{\theta,2}$ (Nmm/rad)</th>
<th>$N_{cr}$ (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>6844.0</td>
</tr>
<tr>
<td>94.7</td>
<td>11.8</td>
<td>6844.0</td>
</tr>
<tr>
<td>∞</td>
<td>∞</td>
<td>27401.2</td>
</tr>
</tbody>
</table>

Table 7.2: Effect of rotational stiffness $C_{\theta,1,2}$ on the buckling resistance $N_{cr}$

### 7.3.8. CONCLUSION

In this section the minimum required height of the transverse stiffener is determined to provide buckling resistance. A buckling model takes the bending stiffness of the stiffened- and unstiffened part into account. The Euler-buckling force is obtained by solving the differential equation for this buckling problem. The gross-section properties are used to determine the bending stiffness. The design buckling strength is obtained according to Eurocode 3, where imperfections are taken into account with the net-section properties, the Euler-buckling force and the appropriate buckling curve.

The minimum height of the embossment to prevent overall buckling is 131 mm. Local buckling will not occur. Here is chosen to apply a web embossment of 150 mm.

Web buckling is affected by the longitudinal stiffeners in the web and the top- and bottom flange. Conservative assumptions are used to include both in the buckling model. Their effect on the Euler buckling resistance is negligible. However, it is expected that their effect is underestimated due to the assumptions. This should be proven in practice.

The bending stiffness and the amount of effective steel have considerable influence on the design buckling strength. Both could influence the formation of the tension field. The following section therefore takes a closer look at the formation of the tension field and the interaction with the transverse stiffeners.

### 7.4. FORMATION OF THE TENSION FIELD

The shear resistance of the deck is designed with the post-buckling capacity of the webs. Here is assumed that the webs of the deck will buckle at a low shear force and that a tension field will form between the transverse
stiffeners. This approach is used in Eurocode 3 to design the post-buckling strength of plate girders. In the design of plate girders the bending stiffness and spacing of the transverse stiffeners is designed, such that the tension field will form in between these stiffeners.

Figure 7.22a illustrates the ideal situation where the compression struts are formed by the transverse stiffeners (web embossments) and the tensile ties by the tension fields in the web. These struts and ties form together a truss model used to determine the post-buckling load-carrying capacity of the deck. In the sections 7.2 and 7.3 is calculated that the transverse stiffener and the tension fields provide sufficient resistance. However, it is important to determine if and how the actual tension field is formed in the web, before conclusions are drawn.

![Figure 7.22a: Ideal formation of the tension field](image)

(a) Ideal formation of the tension field

![Figure 7.22b: Behaviour of the stiffener influenced by tension field](image)

(b) Behaviour of the stiffener influenced by tension field

![Figure 7.22c: Elongation of the stiffener](image)

(c) Elongation of the stiffener

The formation of the tension field is influenced by the interaction between the shear buckling of the web, and the stiffness and position of the transverse stiffeners. The tension field, on the other hand, has influence on the behaviour of the embossment as transverse stiffener, because the embossment can elongate and weaken under tension. Two situations are illustrated in figure 7.22b, where the tension field has influence on the behaviour of the stiffener. Here is expected that the stiffener will elongate as shown in figure 7.22c. Elongation of the transverse stiffener will reduce the compression strength and eventually leads to shear failure of the deck.

It is recommended to obtain the shear capacity of the web in a FEM model. This model should determine the interaction between the formation of the tension field and the capacity of the transverse stiffeners. This model should be validated with tests.

### 7.5. **CONCLUSION**

In this chapter the web is designed to resist shear. The maximum shear force on a single web during service life in ultimate limit state is $V_{Ed} = 5 \text{ kN/web}$. 

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The tension field method of Basler, to determine the post-buckling behaviour of plate girders, is used for the design of the thin-walled stiffened webs of the deck. According to the tension field method it is possible to determine the load-carrying capacity of the web with a truss model. In this model the embossments (transverse stiffeners) form the compression struts and the tension fields form the tensile ties. Here is calculated that the tensile ties are able to transfer a vertical force of 19.1 kN and a horizontal force of 38.1 kN, and the transverse stiffeners are able to resist a compression force of 5.6 kN. Therefore is concluded that the truss model has sufficient resistance for the occurring the shear forces.

The transverse web stiffeners under compression are designed to prevent local- and overall buckling. Here the aim is to design the minimum required height of the stiffener, because the bending stiffness of the deck is negatively influenced if the transverse stiffeners are designed over the whole height of the web. A buckling model takes the bending stiffness of the stiffened- and unstiffened part of the web into account. It is found that the minimum height of the transverse stiffener is 131 mm. Here is chosen to apply a transverse stiffener of 150 mm.

Buckling of the transverse stiffeners is affected by the longitudinal stiffeners in the web, and the top- and bottom flange. In the buckling model their contribution to the buckling resistance is negligible. However, it is expected that their effect is underestimated. This should be proven in practice.

Besides that, it is not proven that the theory of Basler is applicable and that the post-buckling resistance of the web can be modelled as a truss. It is important to determine if and how the actual tension field is formed in the web, before conclusions are drawn.

The formation of the tension field is influenced by the interaction between the shear buckling of the web, and the stiffness and position of the transverse stiffeners. The tension field, on the other hand, has influence on the behaviour of the embossment as transverse stiffener. It is recommended to obtain the shear capacity of the web in a FEM model. This model should determine the interaction between the formation of the tension field and the capacity of the transverse stiffeners. This model should be validated with tests.
In this chapter the shear connection of the floor is designed. Firstly, a design is presented for the shear connectors and design assumptions for calculations are given. Secondly, the amount of connectors is calculated for a partial shear connection to determine if the design is feasible. Finally, improvements and recommendations are given for the design of the shear connectors.

8.1. SHEAR CONNECTORS

The shear connection at the steel-concrete interface has two purposes:
- transfer a longitudinal shear forces to take up the bending moment;
- prevent separation of the steel and the concrete at the steel-concrete interface.

8.1.1. DESIGN OF THE SHEAR CONNECTOR

There are three types of shear connectors to transfer longitudinal shear forces:
- (a) dowels on the top flange: used in steel-concrete composite downstand beam;
- (b) embossments in the deck: used in steel-concrete composite floors;
- (c) dove-tail or re-entrant profiles in the ribs: used in steel-concrete composite floors.

The shear forces need to be transferred between the concrete in the top slab and the adjacent part of the steel deck. The possible locations of the different types of shear connectors on the deck design are indicated in figure 8.1. The effectiveness of the shear connection is different for each type.

Current deck panels are connected with self-drilling self-tapping screws and a shear clip. This connection is made in the bottom of the ribs, where the individual panels overlap. The clips are positioned at a maximum c.t.c. distance of 350 mm and it is known that the clips provide a significant amount of shear resistance [16].

Figure 8.1: Possible position for shear connectors: dowels (a), embossments (b), and dove-tail profile (c)
If this connection is made on the top flange of the deck a shear connection could be formed that is similar to the dowels in a steel-concrete composite downstand beam.

The embossments are most effective along the vertical parts of the deck. The shear resistance of the embossments is determined by the type-, geometry-, and the number of embossments. The spacing of the vertical embossments in current deep decks is 38 mm. In the previous chapter the embossments in the web is designed at a distance of 220 mm to allow for the tension field action. Besides that, only a very small part of the web is in contact with the concrete (figure 8.1). Therefore is expected that the embossments in the web have a negligible influence on the amount of longitudinal shear resistance.

Re-entrant profiles provide composite behaviour by a frictional interlock, allowing a tension bar being formed under a concrete compression arch. These profiles are only applicable on the horizontal parts of the deck, where the concrete and the steel tend to separate. The Eurocode does not provide a calculation method for this type of connector [12]. In practice, this type of shear connector is only used for low profile decks or in combination with another type of shear connector.

Therefore, is concluded that the dowels on the top flange of the deck are the most suitable type of connector. The dowel on the top flange of the deck should:

- connect the single deck panels;
- establish a steel-concrete connection and transfer sufficient longitudinal shear forces: $V_{l,Ed} \geq V_{l,Ed}$;
- prevent separation of the steel and the concrete at the interface.

This section presents design assumptions to determine if dowels on the top flange are suitable shear connectors for the design.

**8.1.2. LOADS**

The shear connection should carry a load equal to that acting in the concrete or the steel between the critical section and the support. The critical section for a simple span is at the centre line (CL) of the floor (in the middle), where the bending moment is maximum (figure 8.2). The maximum bending moment per rib is:

$$M_{\text{max,Ed}} = \frac{1}{8} q_d L^2 = \frac{1}{8} \cdot 7.2^2 = 19.5 \text{ kNm/rib}$$

The maximum longitudinal shear force is:

$$V_{l,Ed} = \min(N_c ; N_p), \text{ at position of the maximum occurring moment}$$

Here is $N_c$ the compression force in the concrete top flange and $N_p$ the tension force in the deck at the critical cross-section. $V_{l,Ed}$ should be transferred over half a span by the shear connectors.
8.1. Shear Connectors

The amount of shear connectors to resist the maximum bending moment \( M_{\text{max,Ed}} \) is:

\[
n = \frac{V_{l,Ed}}{P_{Rd}}
\]

Where \( P_{Rd} \) is the design shear resistance of a single dowel.

8.1.3. Design Shear Resistance

Current decks are connected with self-drilling self-tapping screws and a shear clip (figures 8.3a and 8.3b). These clips are positioned at a maximum c.t.c. distance of 350 mm and it is known that the clips provide a significant amount of shear resistance. In that way the behaviour of the shear clips is similar to the dowels on a steel-concrete composite beam.

In studies the longitudinal shear resistance of a Comflor 210 deep deck, with and without, these clips is determined [16, p.7]. Table 8.1 shows that the contribution of the shear connectors is substantial.

<table>
<thead>
<tr>
<th>Thickness of the deck ( t ) (mm)</th>
<th>Without shear clips ( \tau_{u,Rk} ) (kN/m²)</th>
<th>With shear clips ( \tau_{u,Rk} ) (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.00</td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>1.25</td>
<td>25</td>
<td>37.5</td>
</tr>
</tbody>
</table>

Table 8.1: Characteristic value of the longitudinal shear strength of the Comflor 210 floor slab [16, p.7]

The design value of longitudinal shear strength of a composite slab:

\[
\tau_{u,Rd} = \frac{\tau_{u,Rk}}{\gamma_V}
\]

Where \( \gamma_V = 1.25 \) is the partial factor for the design shear resistance of a composite slab. A sheet thickness of 1.25 mm is considered. The contribution of the shear clips is determined as the difference between the resistance with and without the shear clips.

\[
\tau_{u,Rd} = \frac{37.5 - 25}{1.25} = \frac{12.5}{1.25} = 10 \text{ kN/m}^2
\]

The shear clips in the Comflor 210 are positioned in the bottom of each rib, that have a c.t.c. distance of 600 mm, with a maximum spacing of 350 mm. The amount of shear clips in one square meter is therefore:

\[
n = \frac{1000}{600} \cdot \frac{1000}{350} = 4.76 \text{ shear clips/m}^2
\]

The design longitudinal shear resistance of a single shear clip is determined assuming that the longitudinal shear resistance is uniformly distributed.

\[
P_{Rd} = \frac{\tau_{u,Rd}}{n} = \frac{10}{4.76} = 2.1 \text{ kN}
\]

8.1.4. Rigid or Flexible

The difference between rigid and flexible connectors is illustrated in figure 8.4. Flexible connectors have the advantage that the longitudinal shear is distributed more or less uniformly over half the span. The distribution of the longitudinal shear is not uniform when rigid connectors are applied. The longitudinal shear forces near the supports is much higher than the longitudinal shear forces near the centre of the span. Eurocode 4 recommends to consider connectors as absolutely rigid non-ductile if the load-slip behaviour of the connector is unknown. This approach provides a safe design for the shear connectors over the span. The distribution
of the longitudinal shear force for rigid connectors is similar to the distribution of the vertical shear forces (figure 8.5a). Note that the area of the diagram in figure 8.5a is equal to the total longitudinal shear force $V_{l,Ed}$ that needs to be transferred over half the span.

The load-slip behaviour of the connectors is not know and therefore is the behaviour assumed to be absolutely rigid. The position of the connectors to ensure the elastic force distribution should be according to equal surfaces in the triangle, such that each connector is loaded the same. This means that the spacing between the shear clips is smaller near the supports. The concept of dowels as shear connectors on the top flange of the deck is illustrated in figure 8.5b for half a floor span.

8.1.5. CONCLUSION

In this section assumptions are given for the design of dowels on the top flange of the deck as shear connectors. The shear connector should: connect the single deck panels, establish a steel-concrete connection and transfer sufficient longitudinal shear forces, and prevent separation of the steel and the concrete at the interface.

Current decks panels are connected with shear clips. In studies the longitudinal shear resistance, with and without, these clips is determined. The test results provide a value for the design shear resistance of a single
8.1. **Shear Connectors**

**Figure 8.4**: Distribution of longitudinal shear over half span for rigid- and flexible connectors

(a) Longitudinal shear force that needs to be transferred by the steel-concrete interface

(b) Design of shear connectors over half the span

**Figure 8.5**: Force distribution and design of connectors
shear clip, \( P_{Rd} = 2.1 \text{ kN} \). This value is used as a safe, but conservative design value for the shear resistance of a dowel on the top flange.

The behaviour of the connectors is assumed rigid. The distribution of the longitudinal shear forces therefore similar to the distribution of the vertical shear forces. The spacing between the connectors needs to be designed according to this distribution, such that each connector is loaded the same.

The most economic design is achieved for a partial shear connection. Besides that, the shear clips are not designed to prevent separation of the steel and the concrete. The design of the dowels should prevent separation of the steel and the concrete. The following sections will elaborate on this.

8.2. **PARTIAL SHEAR CONNECTION**

Here it is assumed that the shear clips behave as rigid connectors. This implies:

- no slip between the steel and the concrete → elastic calculation;
- distribution of the longitudinal shear force = distribution of the vertical shear forces.

Figure 8.6 illustrates the distribution of the longitudinal shear forces \( V_{l,Ed} \) and the elastic moment resistance.

The design criteria for the longitudinal shear resistance \( V_{l,Rd} \) is:

\[
V_{l,Rd} \geq V_{l,Ed}
\]

The longitudinal shear forces are, \( V_{l,Ed} = \min(N_c \ ; \ N_p) \), at position of the maximum occurring moment. \( V_{l,Ed} \) should be transferred over half a span under the force distribution illustrated in figure 8.6 (triangular shaped).

The amount of shear connectors to resist the maximum moment \( M_{Ed} \) is:

\[
n = \frac{\text{area of longitudinal shear diagram}}{\text{design strength of connector}} = \frac{V_{l,Ed}}{P_{Rd}}
\]

The position of the shear connectors, to ensure the elastic force distribution, should be according to equal surfaces in the triangle. This means that the amount of connectors gets more dense near the supports.

8.2.1. **INTERNAL STRESS DISTRIBUTION**

The internal stress distribution is determined at the critical cross-section, where the maximum moment occurs. The stress distribution at this position is used to obtain the minimum required longitudinal shear resistance to provide a partial shear connection.
8.2. PARTIAL SHEAR CONNECTION

BENDING STIFFNESS

The bending stiffness is calculated to determine the elastic distribution of forces in the cross-section.

Assumptions:

- only the concrete in compression is taken into account;
- fictitious steel cross-section with: \( n = \frac{E_a}{E_{cm}} = \frac{2.1 \times 10^5}{30000} = 14 \);
- the effective width \( b_{eff} \) is equal to the c.t.c. distance of the ribs.

The dimensions of the cross-section are given in figure 8.7. Properties of the deck:

- \( A_{pe} = 1565 \text{ mm}^2/\text{m} = 448 \text{ mm}^2/\text{rib} \);
- \( I_{eff} = 1773 \text{ mm}^4/\text{m} = 507 \text{ mm}^4/\text{rib} \);
- \( d_p = e_c + h_c = 126 + 60 = 186 \text{ mm} \).

![Figure 8.7: Cross-section of the floor with dimensions used to determined the bending stiffness](image)

POSITION OF THE NEUTRAL AXIS

If only the concrete in the top flange is taken into account:

\[
x_{el} = \frac{A_{pe} d_p + \left( \frac{b_{eff} h_c}{n} \right) h_c}{A_{pe} + \left( \frac{b_{eff} h_c}{n} \right)} = 448 \cdot 186 + \frac{286 \cdot 60}{14 - \frac{2}{2}} = 71.7 \text{ mm}
\]

If also the concrete in the ribs is taken into account:

\[
x_{el} = \frac{A_{pe} d_p + \left( \frac{b_{eff} h_c}{n} \right) h_c + \frac{h_{om} h_x}{n} \left( h_c + \frac{h_x}{2} \right)}{A_{pe} + \frac{1}{n} \left( b_{eff} h_c + b_{om} h_x \right)} = \frac{448 \cdot 186 + \frac{286 \cdot 60}{14} + \frac{220 \cdot 35}{14} \left( 60 + \frac{35}{2} \right)}{448 + \frac{1}{14} \left( 286 \cdot 60 + 220 \cdot 35 \right)} = 77.5 \text{ mm}
\]

The concrete in tension is neglected. Iteration of the calculation above with \( h_{x,n} = x_{el,n} - h_c \) gives for \( n \to \infty \): 
\( x_{el} = 68.9 \text{ mm} \). The concrete in the ribs has only a small contribution. For simplicity is only the concrete of the top flange taken into account.

SECOND MOMENT OF AREA

The second moment of area of the cross-section is:

\[
I_c = \frac{b_{eff} h_c^3}{12n} + \frac{b_{eff} h_c}{n} \left( x_{el} - \frac{h_c}{2} \right)^2 + I_{eff} + A_{pe} (d_p - x_{el})^2
\]

\[
= \frac{286 \cdot 60^3}{12 \cdot 14} + \frac{286 \cdot 60}{14} \left( 71.7 - \frac{60}{2} \right)^2 + 507 \cdot 10^4 + 448 (186 - 71.7)^2
\]

\[
= 1342.2 \cdot 10^4 \text{ mm}^4/\text{rib} = 4693 \cdot 10^4 \text{ mm}^4/\text{m width}
\]
**Section Modulus**

The distances from the n.a. to the extreme fibres are:

\[ z_{\text{top}} = x_{\text{el}} = 71.7 \text{ mm} \]
\[ z_{\text{bot}} = h - x_{\text{el}} = 280 - 71.7 = 208.3 \text{ mm} \]

The section modulus for the top part of the cross-section:

\[ W_{\text{top}} = \frac{I_c}{z_{\text{top}}} = \frac{4693 \cdot 10^4}{71.7} = 654533 \text{ mm}^3/\text{m width} \]

The section modulus for the bottom part of the cross-section:

\[ W_{\text{bot}} = \frac{I_c}{z_{\text{bot}}} = \frac{4693 \cdot 10^4}{208.3} = 225300 \text{ mm}^3/\text{m width} \]

**Stresses in the outer fibres**

The maximum moment \( M_{\text{Ed}} \) at mid-span:

\[ M = M_{\text{Ed}} = \frac{1}{8} q_d L^2 = \frac{1}{8} \cdot 10.4 \cdot 7200^2 = 67.5 \cdot 10^6 \text{ Nmm/m width} \]

The maximum stresses in the outer fibres:

\[ \sigma_{\text{max, top}} = \frac{M \cdot z_{\text{top}}}{I_c} = \frac{67.5 \cdot 10^6 \cdot 71.7}{4693 \cdot 10^4} = 103.1 \text{ N/mm}^2 \]
\[ \sigma_{\text{max, bot}} = \frac{M \cdot z_{\text{bot}}}{I_c} = \frac{67.5 \cdot 10^6 \cdot 208.3}{4693 \cdot 10^4} = 299.6 \text{ N/mm}^2 \]

Note that the stresses are given for the fictitious cross-section (figure 8.8). The given stresses \( \sigma_{\text{max, top, bot}} \) are within the elastic range and there is no yielding in the outer fibres: \( \sigma_s \leq f_{yb, d} = 350 \text{ N/mm}^2 \).

![Figure 8.8: Strain- and stress distribution in the critical cross-section and stress distribution in the fictitious cross-section (right)](image)

**8.2.2. Longitudinal Shear Forces**

As shown above is the neutral axis located in the steel and therefore holds that, \( N_c < N_p \). The amount of longitudinal shear forces that needs to be transferred at the steel-concrete interface is: \( V_{l,Ed} = N_c \).
Strains in the critical cross-section:

\[ \epsilon_s = \frac{\sigma_{\text{max,bot}}}{E_a} = \frac{299.6}{210000} = 1.427\% \]

\[ \epsilon_{c1} = \frac{\epsilon_s}{z_{\text{bot}}} z_{\text{top}} = \frac{1.427\%}{71.7} = 0.491\% \]

\[ \epsilon_{c2} = \frac{\epsilon_s}{z_{\text{bot}}} (z_{\text{top}} - h_c) = \frac{1.427\%}{208.3} (71.7 - 60) = 0.080\% \]

Here are \( \epsilon_{c1} \) the concrete strains at the top fibre and \( \epsilon_{c2} \) the concrete strains at the steel-concrete interface.

Concrete stresses in the critical cross-section:

\[ \sigma_{c1} = \epsilon_{c1} E_c = 0.491 \cdot 10^{-3} \cdot 7619 = 3.74 \text{ N/mm}^2 \]

\[ \sigma_{c2} = \epsilon_{c2} E_c = 0.080 \cdot 10^{-3} \cdot 7619 = 0.61 \text{ N/mm}^2 \]

The behaviour of the concrete is within the linear elastic range of the material and therefore is the modulus of elasticity (left figure 8.9):

\[ E_c = \frac{f_{cd}}{\epsilon_{c3}} = \frac{13.33}{1.75\%} = 7619 \text{ N/mm}^2 \]

The force in the concrete is determined according to the stress diagram in figure 8.9 (right).

\[ N_{c1} = \frac{1}{2} b_{\text{eff}} h_c (\sigma_{c1} - \sigma_{c2}) = \frac{1}{2} \cdot 286 \cdot 60 (3.74 - 0.61) \cdot 10^{-3} = 26.9 \text{ kN/rib} \]

\[ N_{c2} = b_{\text{eff}} h_c \sigma_{c2} = 286 \cdot 60 \cdot 0.61 \cdot 10^{-3} = 10.5 \text{ kN/rib} \]

Total force in the concrete:

\[ N_c = N_{c1} + N_{c2} = 26.9 + 10.5 = 37.4 \text{ kN/rib} \]

The longitudinal shear force that needs to be transferred to provide a partial shear connection:

\[ V_{l,Ed} = N_c = 37.4 \text{ kN/rib} \]

\[ v_{\text{max}} = \frac{V_{l,Ed}}{L} / \frac{4 \cdot V_{l,Ed}}{L} = \frac{4 \cdot 37.4}{7.2} = 20.8 \text{ kN/m} \]

Figure 8.9: Stress-strain diagram for concrete [6] (left) and stress-distribution in the concrete at mid-span for \( M_{Ed} \) (right)
8.2.3. **DESIGN OF CONNECTORS OVER THE SPAN**

In the previous section a safe design value for the longitudinal shear resistance of a single connector is determined, \( P_{Rd} = 2.1 \text{kN} \). The required required amount of connectors is therefore:

\[
n = \frac{V_{1,Ed}}{P_{Rd}} = \frac{37.4}{2.1} = 17.8 \rightarrow 18 \text{ connectors per half span}
\]

As discussed in section 8.1 is assumed that the connectors behave rigidly. Their position over the floor span is designed to ensure that each connector is loaded the same providing a linear elastic longitudinal stress distribution. The design load per connector is:

\[
P_{Ed} = \frac{V_{1,Ed}}{n} = \frac{37.4 \cdot 10^3}{18} = 2077.8 \text{ N}
\]

Each connector is loaded the same if designed according to equal surfaces in the longitudinal shear diagram.

The longitudinal shear diagram is written as function over \( x \):

\[
f(x) = -\frac{v_{max}}{L/2} \cdot x + v_{max}, \quad \text{for } 0 \leq x \leq L/2
\]

Here is \( v_{max} = 20.8 \text{ N/mm} \) and \( L = 7200 \text{ mm} \). The load for a connector is determined with the integral of \( f(x) \) over a certain length \( x \) over the span (figure 8.10).

\[
N_i = \int_{n_{i-1}}^{n_i} f(x) \, dx
\]

\[
x_i = n_{i-1} + \frac{n_i - n_{i-1}}{2}
\]

Where:

- \( N_i \) is the longitudinal shear load on connector \( i \) in \( \text{N} \);
- \( n_{i-1} \) is the left boundary of connector \( i \) in \( \text{mm} \);
- \( n_i \) is the right boundary for connector \( i \) in \( \text{mm} \);
- \( x_i \) is the position of the connector from the support to the CL in \( \text{mm} \);
- \( i \) number of the connector: \( i = 1..n \), where \( n \) is the number of connectors for the half-span.

The position of the connector on the half-span \( x_i \) is obtained by solving the integral given that each connector carries the same load, \( N_i = P_{Ed} \) for \( i = 1..n \). There are 18 connectors and therefore 18 integrals need to be solved to determine the position of each connector over the half-span. The left boundary of the first connector is 0. This gives:

\[
N_1 = \int_0^{n_1} f(x) \, dx = \int_0^{n_1} \left( -\frac{v_{max}}{L/2} \cdot x + v_{max} \right) \, dx = \left[ -\frac{v_{max}}{L/2} \cdot x^2 + v_{max} \cdot x \right]_0^{n_1}
\]

\[
= \left( \frac{v_{max}}{L/2} \cdot n_1^2 + v_{max} \cdot n_1 \right) - \left( \frac{v_{max}}{L/2} \cdot 0^2 + v_{max} \cdot 0 \right) = \frac{v_{max}}{L/2} (n_1^2 - 0^2) + v_{max}(n_1 - 0) = P_{Ed} = 2077.8
\]

The solution of this equation gives that: \( n_1 = 101.4 \text{ mm} \). The position of the connector on the span is:

\[
x_1 = 0 + \frac{101.4 - 0}{2} = 50.7 \text{ mm}
\]

With \( n_1 = 101.4 \text{ mm} \) is the position of the second connector calculated using the same approach.

\[
N_2 = \int_{n_1}^{n_2} f(x) \, dx = \int_{101.4}^{n_2} f(x) \, dx = P_{Ed} = 2077.8
\]

Solving this gives that: \( n_2 = 205.9 \text{ mm} \). The position of the connector on the span is:

\[
x_1 = 101.4 + \frac{205.9 - 101.4}{2} = 153.7 \text{ mm}
\]

The position of each connector is calculated with this method. Figure 8.11 shows the design of the shear connectors over half-span. The position of the each connector is measured in mm from the support to the centre line (CL). The average spacing of the connectors is 200 mm, the minimum spacing at the support is 103 mm, and the maximum spacing in the middle of the span is 848.6 mm.
8.2.4. Conclusion

The amount of connectors is calculated for a partial shear connection that is designed to resist the maximum bending moment. The position of the connectors over the floor span are designed to ensure that each connector is loaded the same providing a linear elastic longitudinal stress distribution. It is found that 18 connectors per half-span are required to transfer sufficient longitudinal shear forces. This amount of dowels on the top flange is feasible.

8.3. Moment Capacity

The steel-concrete composite floor has to resist a maximum bending moment $M_{Ed}$ of 67.5 kNm/m. 18 connectors per half-span are required to resist this bending moment providing a partial shear connection. In this section the amount of shear connectors is determined to resist the elastic- and plastic moment providing a partial- and full shear connection, respectively.

8.3.1. Elastic Moment Resistance

The elastic moment resistance is:

$$M_{el,Rd} = W_{el} f_{yh,d} = 225300 \cdot 350 \cdot 10^{-6} = 78.9 \text{ kNm/m width}$$

The n.a. is located in the steel and therefore is $V_{l,Ed} = N_c$. The total force in the concrete is determined with the same approach as in section 8.2. The longitudinal shear force that needs to be transferred over half the

Figure 8.10: Position of the shear connectors according to equal surfaces

Figure 8.11: Designs of the position of the shear connectors over the half-span
span is:

\[ V_{l,Ed} = N_c = 43.6 \text{kN/rib} \]

The required amount of connectors to resist the elastic moment is:

\[ n = \frac{V_{l,Ed}}{P_{Rd}} = \frac{43.6}{2.1} = 20.8 \rightarrow 21 \text{ connectors per half-span} \]

### 8.3.2. Plastic moment resistance

The maximum concrete compression force and steel tension force are:

\[ N_c = 0.85bh_c f_{cd} = 0.85 \times 1000 \times 60 \times 13.33 \times 10^{-6} = 679 \text{kN/m} \]

\[ N_p = A_{pe} f_{yb,d} = 1565 \times 350 \times 10^{-6} = 547 \text{kN/m} \]

The neutral axis is in the concrete, because \( N_p < N_c \). The height of the compression zone is:

\[ x_u = \frac{A_{pe} f_{yb,d}}{0.85 f_{cd} b} = \frac{1565 \times 350}{0.85 \times 13.33 \times 1000} = 32.2 \text{ mm} \]

The effective depth is:

\[ d_p = h_c + \frac{h_w}{2} = 60 + \frac{220}{2} = 170 \text{ mm} \]

The internal leverarm for the plastic moment is:

\[ z = d_p - \frac{x_u}{2} = 170 - \frac{32.2}{2} = 153.9 \text{ mm} \]

Here is assumed that the position of the plastic neutral axis of the deck is at half the height of the deck. The plastic moment resistance:

\[ M_{pl,Rd} = N_p z = A_{pe} f_{yb,d} z = 1565 \times 350 \times 153.9 \times 10^{-6} = 84.3 \text{kNm/m} \]

The longitudinal shear force is:

\[ V_{l,Ed} = N_p = 547 \text{kN/m} = 156 \text{kN/m} \]

The required amount of connectors to resist the plastic moment is:

\[ n = \frac{V_{l,Ed}}{P_{Rd}} = \frac{156}{2.1} = 74.5 \rightarrow 75 \text{ connectors per half span} \]

### 8.3.3. Conclusion

In the previous section is found that a minimum amount of 18 connectors are required per half-span to resist the maximum bending moment of 67.5 kNm/m. In this section the amount of shear connectors per half-span are determined to resist the elastic- and plastic moment. More shear connectors provide more bending moment resistance and can be used to increase the capacity of the steel-concrete composite floor slab.

It is found that 21 connectors per half-span are required to provide the elastic moment resistance of 78.9 kNm/m. 3 extra connectors per half span provide for 11.4 kNm/m extra resistance. For a full shear connection 75 connectors per half span are required to provide the plastic bending moment resistance of 84.3 kNm/m. Thus 57 extra connectors per half span provide for 16.8 kNm/m resistance.

The amount of shear connectors that is required for full shear connection is not feasible. Under this plastic stress distribution are the forces in the steel and concrete increased, but the internal leverarm is smaller. Therefore a lot of extra shear connectors are required that provide little extra bending moment resistance. If extra capacity is required it is recommended to install 3 extra connectors per half-span.

Note that the full plastic moment capacity of the composite floor is larger than required, because the construction stage is governing for the deck dimensions.
8.4. ALTERNATIVE SHEAR CONNECTIONS

Two other types of connectors could be designed in addition to the dowels: dove-tails and embossments, both mentioned in section 8.1.

A dove-tail is considered as an unreliable longitudinal shear connection. However, the advantage of this connection is prevention of vertical separation of the steel and the concrete. An impression of a dove-tail profile on the top flange is given in figure 8.12. The disadvantage of this design is shown in the detail. The amount of effective steel in the compression zone will reduce and the stackability of the deck panels is less effective.

Embossments in the web can be designed if the ribs are partially cast (figure 8.12). These embossments could provide extra longitudinal shear resistance. The disadvantage is that the amount of effective steel in the compression zone will reduce.

The drawback of a design with multiple types of shear connections is the possible difference of stiffness. This difference could lead to early failure. For example, dowels are designed with embossments, where the longitudinal shear connection of the dowels is stiffer. First the shear connection of the dowels will fail and after that the longitudinal shear forces are only resisted by the embossments. These have less capacity and will immediately fail. This implies that when designing a deck with two types of connectors the capacity of the connectors cannot be simply added.

A second drawback of the design with dove-tails and embossments that both have a negative effect on the amount of effective steel.

8.5. DESIGN IMPROVEMENTS FOR THE CONNECTORS

In the previous sections is shown that the dowels as shear connectors provide sufficient steel-concrete composite behaviour and connect the individual panels during construction. Test results for shear clips, used in current deep decks, provide a safe design value for the shear resistance of a single connector. However, these clips are not designed to prevent vertical separation of the steel and the concrete. In this section the shear connector on the top flange of the deck is designed to prevent vertical separation.

Figure 8.13 shows two designs for connectors that would prevent vertical separation. The first design is similar to the current shear clip, but installed upside down. Besides that, it is suggested to bend the upturned part back to provide optimal anchorage. The second design is almost identical to the first, but rotated 90 degrees. It is therefore more comparable with the design of shear connectors used for steel-concrete composite beams, where the connector works as a shovel in the concrete. The dimensions of both designs are based on the current shear clip. It is expected that both designs provide at least a shear resistance of 2.1 kN.

The behaviour of the shear connectors could be calculated with a FEM model. It is very difficult to model the actual steel-concrete interaction and the behaviour of the screw. Therefore is recommended to perform tests. The capacity of dowels in steel-concrete beams is determined with ‘push tests’. This small scale test is suitable to determine the longitudinal shear resistance and the load-slip behaviour for the connectors.
There are four existing types of ‘push test’ to determine the longitudinal shear bond characteristics of profiled sheeting: Daniel’s, Patrick’s, Porter’s, and Stark’s push test (figure 8.14). Burnet and Oehlers [7] recommend to use Stark’s push test, because the shear connector behaviour is modelled the most realistic and the test piece is manufactured easiest.
8.6. **CONCLUSION**

The steel-concrete shear connection is established with dowels on the top flange of the deck. Test results for current decks provide a value for the design longitudinal shear resistance of a single dowel and the behaviour is assumed to be rigid.

The minimum amount of shear connectors for the steel-concrete composite floor slab is calculated to resist the maximum bending moment $M_{Ed}$ of 67.5 kNm/m. It is found that 18 connectors per half-span should be installed. The position of the connectors over the floor span is designed to ensure that each connector is loaded the same, providing a linear elastic longitudinal stress distribution.

The amount of shear connectors for a full shear connection is not feasible. If extra moment capacity is required is recommended to install 3 extra connectors per half-span, providing a maximum bending moment resistance of 78.9 kNm per meter floor width. The dowels as shear connectors can provide sufficient steel-concrete composite behaviour and connect the individual deck panels during construction. However, the current design does not prevent vertical separation of the steel and the concrete.

Two designs for shear connectors that would prevent vertical separation are proposed. The shear behaviour of the connector is difficult to predict with a FEM model. Therefore is recommended to perform a 'push tests', in accordance with Eurocode 4, to determine the longitudinal shear resistance and the load-slip behaviour per connector. The authors recommend to use Stark’s push test, because the shear connector behaviour is modelled the most realistic and the test piece is manufactured easiest.
In this chapter the effect of fire on the floor slab will be dealt with. The floor slab is designed to prevent failure as a result of fire. Verifications for bending, shear and buckling are performed. Besides that, attention is paid to the design of the anchorage of the tension reinforcement at the support.

Figure 9.1 shows two fire situations for the floor.

- fire load from below (1);
- fire load from above (2).

This chapter considers the first situation, because this is the most severe fire situation for the floor slab. The floor slab is designed to provide a mechanical fire resistance of 60 (R60) and 90 (R90) minutes.

### 9.1. Floor Slab during Fire

During fire it is expected that the steel deck will heat up quickly and lose strength already at a low fire load (figure 9.2). In the fire design of the floor slab is therefore assumed that the deck is not able to transfer forces.
9.1.1. Load-carrying model

If the deck loses its strength a new load-carrying principle will develop. In the load-carrying model during fire, shown in figure 9.3, the reinforcement in the bottom of the ribs acts as a tensile tie and the concrete as a compression arch. The concrete top flange is under compression, whereas the reinforcement in the bottom part of the ribs will take up the tension forces.

The assumptions for the behaviour of the floor during fire are illustrated in figure 9.4.

- deck will lose strength → ignore deck during fire;
- fill element will burn or gas → ignore fill element during fire;
- load-capacity of steel and concrete reduces at elevated temperatures;
- concrete top slab is heated from one side and the tension bar from all sides.

In the design concept the ribs near the supports are fully cast (figure 9.3).

- to anchor the tension reinforcement;
- guide the compression force towards the support;
- provide more shear resistance near the support.

The tension reinforcement in the bottom of the ribs is protected by some concrete and is therefore able to take up a tensile force. Important is proper anchorage of the tension reinforcement at the support.

9.1.2. Loads

During fire all load- and partial factors are equal to 1.0 and allows the Eurocode to reduce the variable load with factor $\Psi_2=0.3$ (figure 9.5). The design loads during fire are therefore:

$$ q_{f_i,d} = \gamma_k R_k + \gamma_q \Psi_2 q_k = 1.0 \cdot 3.6 + 1.0 \cdot 0.3 \cdot 4.0 = 4.8 \text{ kN/m}^2 = 1.4 \text{ kN/m per rib} $$

The maximum shear force and bending moment during fire are:

$$ V_{f_i,Ed} = \frac{1}{2} q_{f_i,d} L = \frac{1}{2} \cdot 1.4 \cdot 7.2 = 5.0 \text{ kN per rib} $$

$$ M_{f_i,Ed} = \frac{1}{8} q_{f_i,d} L^2 = \frac{1}{8} \cdot 1.4 \cdot 7.2^2 = 9.0 \text{ kNm per rib} $$
9.1. Floor Slab During Fire

(a) Effect of fire on the floor slab

(b) Concrete top slab with fire load at one side: design with some concrete in the ribs (left) and design without concrete in the ribs (right)

(c) Tension reinforcement: fire load at four sides

Figure 9.4: Fire load on the concrete top slab and the tension reinforcement

Figure 9.5: Loads and partial factors during fire
9.1.3. Failure modes
The design of the floor slab is verified during fire:

- the moment capacity of the compression arch with tensile tie;
- the shear capacity of the concrete top slab.

Besides failure due to bending or shear could this floor fail due to instability. The different failure modes for instability are illustrated in figure 9.6. The cross-section of the ribs near the supports have a high bending- and torsional stiffness, because the ribs are fully cast. Therefore only one instability problem is addressed:

- buckling of the concrete slab under compression.

The following section first elaborates on the influence of the temperature increase on the material strength.

![Figure 9.6: Different failure modes of the floor slab due to instability](image)

9.2. Reduced material properties
Material properties of the steel- and concrete are influenced during fire, where the strength reduces at certain elevated temperatures. The load-carrying capacity of the concrete top slab and the tension reinforcement is determined with temperature profiles given in the appendix of Eurocode 2 part 2.

During fire material-factors are equal to 1: $f_{sd} = 500 \text{ N/mm}^2$ and $f_{cd} = 20 \text{ N/mm}^2$.

9.2.1. Concrete slab
The reduced concrete cross-section is based on the 500\degree isotherms. The concrete in the cross-section with temperatures less than 500\degree is fully taken into account, including the present reinforcement bars [14, p.76]. The isotherms for a concrete flat slab are illustrated in figure E.2 of the appendix. The concrete that is affected due to a R60 and R90 fire load is illustrated in figure 9.7. The reduced concrete cross-sections are obtained with these values:

- amount of concrete affected after a fire load of 60 minutes: $f_{R60} = 22 \text{ mm}$;
- amount of concrete affected after a fire load of 90 minutes: $f_{R90} = 29 \text{ mm}$.

9.2.2. Tension reinforcement
The tension reinforcement is protected by a concrete beam of $h \times b = 120 \times 60 \text{ mm}$. Temperature profiles for a concrete beam $h \times b = 300 \times 160 \text{ mm}$ are given in Eurocode 2. The temperature of the reinforcement is estimated at the centre of the bar using the temperature profiles found in figure E.3 of the appendix. It is expected that the temperature of the reinforcing bar is 500\degree for R60 and 650\degree for R90.
9.3. Moment Capacity

Eurocode 2 gives reduction factors $\Psi$ for structural steel at elevated temperatures given in table E.1 of the appendix. The maximum reduction factor depends on ratio between the load at fire limit state and the tensile resistance under normal fire conditions.

$$\frac{P}{P_u} = \frac{\text{Load at fire limit state}}{\text{Tensile resistance under normal fire conditions}} = \frac{N_s}{A_s \cdot f_{sd}} \tag{9.1}$$

Note that the dimensions of the concrete beam in the bottom of the ribs are chosen to limit the temperature increase of the reinforcement bar for a minimum weight increase. Larger dimensions provide more fire resistance, but also result in a heavier floor slab.

\section*{9.3. Moment Capacity}

The load-carrying principle during fire, as discussed in section 9.1, is illustrated in figure 9.8. This section verifies the compression force in the concrete and tension force in the reinforcement to determine if the moment resistance is adequate.

The design compression- and tension force are obtained with the internal leverarm and the occurring maximum bending moment. Here is:

- the internal leverarm is estimated at: $z = 210$ mm;
- design compression- or tension force: $M_{fi,Ed} = Nz \geq M_{fi,Ed} \rightarrow N = M_{fi,Ed} / z = 9/0.211 = 43$ kN/rib.

The maximum compression force in the top flange is obtained for the most severe fire load, R90.

$$N_c = 0.85 b h_c f_{cd} = 0.85 \cdot 288 \cdot 31 \cdot 13.33 \cdot 10^{-3} = 101.2 \text{ kN/rib} > N_{Ed} = 43 \text{ kN/rib}$$

The concrete top flange has sufficient resistance.

With expression 9.1, for the maximum reduction factor, the temperature of the reinforcing bar, and the tension force in the reinforcing bar during fire ($N_s = 43$ kN), the required area of reinforcement is determined. The following relation is used.

$$\frac{N_s}{A_s f_{sd}} \leq \Psi \Rightarrow A_s \geq \frac{N_s}{\Psi f_{sd}}$$

$$q_{fl,Ed} = 1.4 \text{ kN/rib}$$

\section*{Figure 9.8: Load-carrying principle (left) and resisting moment (right) during fire
An overview of the required reinforcement is given in table 9.1. Apply Ø16 mm in each rib for a fire resistance of 60 minutes and Ø20 mm for a fire resistance of 90 minutes.

<table>
<thead>
<tr>
<th>Steel temperature</th>
<th>Strength reduction</th>
<th>Required steel area</th>
<th>Chose bar</th>
<th>Provided steel area</th>
</tr>
</thead>
<tbody>
<tr>
<td>R 60</td>
<td>500</td>
<td>0.63</td>
<td>137</td>
<td>16</td>
</tr>
<tr>
<td>R 90</td>
<td>650</td>
<td>0.33</td>
<td>261</td>
<td>20</td>
</tr>
</tbody>
</table>

Table 9.1: Required reinforcement for fire resistance R60 ad R90

9.4. SHEAR CAPACITY

The shear resistance is determined taking only the concrete cross-section into account. The shear resistance should be verified for two cross-sections (figure 9.9).

1. the fully cast cross-section near the support;
2. the cross-section with fill element over the length of the span.

![Figure 9.9: Fully cast cross-section (left) and cross-section with fill element (right)](image)

The shear loads are illustrated in figure 9.10 for a single rib. The shear load at the support is 5 kN/rib (maximum shear load for cross-section 1), and the shear load at a distance of 600 mm from the support is 4.2 kN/rib (maximum shear load for cross-section 2).

Firstly, the resistance of the cross-section with fill element (2) is verified. This cross-section is expected to be critical. Given is:

- reduced height of the cross-section: \( h = h_c - f_{R90} = 60 - 29 = 31 \text{ mm} \);
- width of the concrete cross-section = c.t.c. distance of the ribs: \( b_w = 286 \text{ mm} \);
- effective depth: \( d = c_d + \sigma/2 = 25 + 8/2 = 29 \text{ mm} \).

Eurocode 2 gives an expression to calculate the minimum shear capacity of a concrete cross-section.

\[
V_{Rd,\text{min}} = v_{\text{min}} b_w d = 0.035 k^2 \cdot f_{ck}^{\frac{3}{2}} \cdot b_w d
\]

\[
k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{29}} = 3.62 < 2.0
\]

\[
V_{Rd,\text{min}} = 0.035 \cdot 2^{\frac{3}{2}} \cdot 20^{\frac{1}{2}} \cdot 286 \cdot 29 \cdot 10^{-3} = 3.6 \text{ kN/rib}
\]

The minimum shear capacity is not sufficient. Therefore the contribution of the reinforcing steel is included using the following equation [6, p.84-87].

\[
V_{Rd,c} = C_{Rd,c} k \left[ 100 f_{ck} \left( \frac{k_1}{\sigma_{cp}} \right) \right] b_w d \geq v_{\text{min}} b_w d
\]
The very slender concrete top flange is under compression and therefore sensitive for buckling. During fire, when the deck and fill element reduce in strength, the concrete top flange will deflect under its own weight. This deflection combined with the compression force results in an extra bending moment and therefore extra deflection. The extra deflection results in an extra load, and so on. This effect is called the 2\textsuperscript{nd} order effect and is approximated with a magnification factor \( n/(n - 1) \). The buckling of the concrete slab is an instability problem and could cause early failure of the floor during fire.

There are two design solutions for this buckling problem. The first is to prevent buckling by supporting the concrete top flange over its length. This is achieved by applying a non-combustible lightweight fill element. The second option is to design the concrete top flange such that it has sufficient bending stiffness to prevent failure due to buckling. This is achieved by:

- increasing the amount of concrete \( h_c \) and/or the concrete class;
- applying a double mesh, thicker bars and/or a finer mesh.

The most favourable option depends on the design of both variants, where self-weight and cost are decisive. This section elaborates on both solutions.
9.5.1. **Non-combustible fill element**

A non-combustible fill element will support the concrete top flange during fire and prevents failure due to buckling. The Eurocode EN 13501-1 classifies the combustibility of insulation materials; class A1 and A2 are non-combustible and class B burns with great difficulty [38]. Materials of these classes are suitable to support the concrete top flange during fire.

Plastic foam fill elements are easy to install, provide a working platform and formwork for the concrete, are moisture resistant and have a low density of 15–45 kg/m$^3$. There are different types of plastic foams: EPS, XPS and PUR. However, these plastic foam are very combustible and not suitable to support the top flange during fire.

Another plastic foam is PIR, which is comparable to PUR, but has a better fire resistance. PIR is classified as class B and could be suitable to apply as non-combustible fill element.

Resol foam is a different type of plastic foam that has a good fire resistance. This product is better known as Kooltherm [39]. Kooltherm panels have a low density of approximately 40 kg/m$^3$ and are classified in fire class B. The disadvantage is its sensitivity for water, and transport in a closed vehicle and dry storage at site is recommended. The cast of wet concrete on this material will not cause problems, because the concrete dries relatively quickly due to the fast chemical reaction between water and cement.

Mineral wools in general are non-combustible and classified in fire class A1, A2 and B. There are different types of mineral wools:

- glass wool blankets: are compressed easily and will deflect when the top flange is cast;
- rock wool panels: resist compression, but are sensitive for water (density 45-150 kg/m$^3$);
- foam glass panels: resist compression and are moisture resistant (density 100-165 kg/m$^3$).

The disadvantage of rock wool plates are the transport and installation requirements that are similar to the Koolterm [40]. Foam glass panels have a higher density, but are not sensitive for water.

PIR, Koolterm, rock wool, and foam glass are suitable to prevent buckling of the concrete slab during fire. A comparison is made between the weight of a foam glass fill element and a EPS fill element. Table 9.2 shows that both provide good weight reduction. Here the foam glass is compared, because the density is the highest of the different non-combustible insulation materials.

<table>
<thead>
<tr>
<th>Ribs</th>
<th>density (kg/m$^3$)</th>
<th>self-weight$^{(1)}$ (kN/m$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>fully cast</td>
<td>2400</td>
<td>2.64</td>
</tr>
<tr>
<td>EPS filled</td>
<td>45</td>
<td>0.05</td>
</tr>
<tr>
<td>foam glass filled</td>
<td>165</td>
<td>0.18</td>
</tr>
</tbody>
</table>

$^{(1)}$ Volume of the fill element per rib is 0.192 m$^3$ (based on an area of 200 x 160 mm over a length of 6 m)

Table 9.2: Self-weight of the fill element

9.5.2. **Design of the concrete top flange**

In this section the concrete top flange is designed such that it has sufficient bending stiffness to prevent failure due to buckling.

**Buckling problem**

This buckling problem is similar to a slender column loaded in compression and bending. The full cross-section is expected to be very stiff compared to the reduced cross-section. Therefore, the concrete top flange is modelled as a column under compression and bending clamped at both ends. The buckling length for this situation is known to be half the column length (figure 9.11).

The following is used:
9.5. Buckling of the concrete slab

Figure 9.11: Buckling modes of the concrete part that is under compression

- loads: \( N_c = 43 \text{ kN} \) and \( q_{f1,Ed} = 1.4 \text{ kN/m} \);
- geometry: \( l = 6000 \text{ mm} \) and \( l_{cr} = 3000 \text{ mm} \).

Here is \( l \) the system length and \( l_{cr} \) the buckling length. The buckling force \( F_{cr} \) is determined with the Euler buckling equation [37, p.146].

\[
F_{cr} = \frac{\pi^2 EI}{l_{cr}^2} \tag{9.2}
\]

To solve this, one needs to determine the bending stiffness of the concrete top flange. This is obtained with a MN\(\times\)-diagram, because the bending stiffness of the concrete cross-section depends on the occurring moment, normal force and curvature. The maximum bending moment resistance is reached at rupture, where the bending stiffness \( EI_{Rd} \) is the lowest. Besides that, the bending resistance will differ for the hogging- and the sagging bending region. A safe design assumption is therefore to determine \( F_{cr} \) assuming the minimum bending stiffness over the whole length of the concrete top slab.

\[
EI = \min(EI_{Rd}^-; EI_{Rd}^+) \tag{9.3}
\]

The bending stiffness is used to calculate the 1\(^{\text{st}}\) order moments and deflections. The 1\(^{\text{st}}\) order moments and deflections are used to determine the magnification factor \( n/(n-1) \) to calculate the 2\(^{\text{nd}}\) order moments. \( n/(n-1) \) is approximated with [37, p.323]:

\[
n = \frac{F_{cr}}{F} \tag{9.3}
\]

Here is \( F \) the load on the system.

The design of the concrete top flange is verified for the hogging- and sagging bending region: \( M_{f1,Rd}^- \geq M_{f1,Ed}^- \) and \( M_{f1,Rd}^+ \geq M_{f1,Ed}^+ \).

Cross-sectional properties

For both regions, sagging and hogging, the cross-section of the concrete compression flange is simplified to determine the cross-sectional properties. Figure 9.12a shows both cross-sections. Here indicates \( x \) the amount of concrete in the ribs in mm.

The amount of applied reinforcement (top \( A_{s1} \) and bottom \( A_{s2} \)) and the amount of concrete in the ribs \( x \) are important design parameters that influence the moment resistance and bending stiffness. Assumed is that:
9. Situation during fire

(a) Reduced cross-section due to fire exposure

(b) Labelling of the simplified cross-sections

Figure 9.12: Simplified cross-sections: for sagging region (left) and for hogging region (right)

- reduction of the cross-section due to fire: \( f_{R90} = 29 \, \text{mm} \);
- top reinforcement is positioned at 29 mm from the top \( (c_d + \phi/2 = 25 + 8 = 29) \);
- bottom reinforcement is positioned on the top flange of the steel at 60 mm from the top \( (h_c = 60) \);
- steel temperature \( \theta < 400 \, ^\circ\text{C} \) → reduction factor for steel \( \Psi(\theta) = 1 \).

The following material properties are used:

\[
\begin{align*}
f_{cd} &= \frac{f_{ck}}{\gamma_c} = \frac{30}{1} = 30 \, \text{N/mm}^2 \\
E_c &= \frac{f_{cd}}{\epsilon_{cd}} = \frac{30}{1.75\%} = 17142 \, \text{N/mm}^2 \\
E_s &= 200000 \, \text{N/mm}^2 \quad \text{(B500 reinforcing steel)}
\end{align*}
\]

The different moduli of elasticity are taken into account with factor \( n \).

\[
n = \frac{E_s}{E_c} - 1 = \frac{200000}{17142} - 1 = 10.7
\]

Note that this factor should be reduced with 1, because the concrete is already taken into account.

The cross-sections are designed to provide sufficient bending resistance.

**Sagging moment**

Design of the cross-section for the sagging region:

- height of the concrete in the ribs: \( x = 35 \, \text{mm} \);
- top reinforcement: standard mesh \( \phi 8 - 200 \rightarrow A_{s1} = 72 \, \text{mm}^2 \);
- bottom reinforcement: \( 3\phi 12 \rightarrow A_{s2} = 339 \, \text{mm}^2 \).

Dimensions of the simplified reduced cross-section:

\[
\begin{align*}
h_1 &= 38 \, \text{mm} \\
h_2 &= 35 \, \text{mm}
\end{align*}
\]
9.5. Buckling of the Concrete Slab

$h = 73$ mm;
$b_1 = 286$ mm;
$b_2 = 170$ mm;
$d_1 = 29$ mm;
$d_2 = 60$ mm.

The dimensions are illustrated in figure 9.12b. Here are $d_1$ and $d_2$ the effective depths of the top- and bottom reinforcement, respectively.

Hogging Moment

Design of the cross-section for the hogging region:
- height of the concrete in the ribs: $x = 70$ mm;
- hogging moment reinforcement: standard mesh $\varnothing 8 - 200$ and $3\varnothing 12 \rightarrow A_{s1} = 72 + 339 = 411$ mm$^2$;
- bottom reinforcement: $3\varnothing 12 \rightarrow A_{s2} = 339$ mm$^2$.

Dimensions of the simplified reduced cross-section:
$h = 108$ mm;
$b = 170$ mm;
$d_1 = 79$ mm;
$d_2 = 48$ mm.

The dimensions are illustrated in figure 9.12b.

Bending Strength and Stiffness

The lowest bending stiffness is found for the maximum bending moment. The maximum bending moment is at rupture, when the concrete fails in compression. The stains, stresses and internal forces at this moment are illustrated in figure F.3d of the appendix. The stress-strain relations are found under the following assumptions:
- $\epsilon_c = \epsilon_{cu3} = 3.5\%$;
- $\epsilon_s \cdot E_s < \epsilon_{ud} = 0.9\epsilon_{uk} = 0.9 \cdot 25 = 22.5\%$.

Yielding of the steel occurs at:
$$\epsilon_{sy} = \frac{f_{sd}}{E_s} = \frac{500}{200000} = 2.5\%$$

With these assumptions the maximum bending moment $M_{Rd}$, the related curvature $\kappa_{Rd}$, and the bending stiffness $EI_{Rd}$ for the sagging- and hogging moment are calculated in appendix F.2. The moment resistance of the sagging region is $M^+_{Rd} = 7.81$ kNm and the moment resistance of the hogging region is $M^-_{Rd} = 10.85$ kNm. A safe design value for the bending stiffness of the concrete top flange is:
$$EI = EI_{Rd} = \min\left\{EI^+_{Rd}; EI^-_{Rd}\right\} = \min(85.0; 158.0) = 85.0 \text{ kNm}^2$$

Loads and Deflections

The moments in the concrete top flange due to the uniform load are:
$$M^0_0 = \frac{1}{24} q_d l^2 = \frac{1}{24} \cdot 1.4 \cdot 6^2 = 2.10 \text{ kN.m}, \quad \text{at mid-span}$$
$$M^-_0 = \frac{1}{12} q_d l^2 = \frac{1}{12} \cdot 1.4 \cdot 6^2 = 4.20 \text{ kN.m}, \quad \text{at the support}$$

Here is $q_d = q_{fj,Ed} = 1.4 \text{ kN/m/rib}$. These moments are illustrated in figure 9.13a. The deflection is $\delta_0$.
$$\delta_0 = \frac{1}{384} \frac{q_d l^4}{EI} = \frac{1}{384} \frac{1.4 \cdot 6000^4}{85.0 \cdot 10^9} = 55.6 \text{ mm}$$
The deflection causes an additional moment in the concrete top flange. In figure 9.13b two models are shown to determine these additional moments at mid-span \( M_{\text{addl}}^+ \) and the support \( M_{\text{addl}}^- \).

\[
M_{\text{addl}}^+ = N_{Ed} \cdot \delta_0 = 43 \cdot 10^3 \cdot 55.6 \cdot 10^{-6} = 2.40 \text{ kNm}
\]

\[
M_{\text{addl}}^- = \frac{6EI}{\left(\frac{l}{2}\right)^2} \cdot \delta_0 = \frac{6 \cdot 85.0 \cdot 10^9}{\left(\frac{6000}{2}\right)^2} \cdot 55.6 \cdot 10^{-6} = 3.15 \text{ kNm}
\]

The total additional moments for the deflection in equilibrium state \( \delta_e \) are called the second order moments. By equilibrium state is meant that extra deflection does not lead to extra moment and vice versa. The deflection \( \delta_e \) is approximated with the magnification factor \( n/(n-1) \), where \( n \) is \( F_{cr}/F \). \( F \) is the load on the system. The buckling load is:

\[
F_{cr} = \frac{\pi^2 EI}{l_{cr}^2} = \frac{\pi^2 \cdot 85.0 \cdot 10^9}{3000^2} = 93.2 \text{ kN}
\]

Therefore is \( n = 93.2/43 = 2.17 \) and \( n/(n-1) = 1.86 \).

\[
\delta_e = \delta_0 \cdot \frac{n}{n-1} = 55.6 \cdot 1.86 = 103.4 \text{ mm}
\]

The second order effect is large due to high slenderness of the concrete top flange and large initial deflection. The bending stiffness, used to determine the second order effect, is here taken very conservatively. A detailed calculation could provide a more economic design.

The thicker concrete top flange results in a increased self-weight of 1.31 kN/m². The extra reinforcement is not included.
9.5. BUCKLING OF THE CONCRETE SLAB

Figure 9.13: Modelling of loads, deformations and occurring moments

(a) Concrete top slab loaded in compression and bending (left) and 1\textsuperscript{st} order deformation

(b) Moment due to 1\textsuperscript{st} order deformation: at support (left) and at mid-span (right)
9.5.3. **CONCLUSION**

Buckling of the concrete top flange during fire is prevented in two ways; a non-combustible fill element provides support or the concrete top flange is designed to resist buckling.

Plastic foam fill elements, like EPS, XPS and PUR, are very combustible and will not provide support. PIR, Koolterm, rock wool, and foam glass fill elements are fire resistant, and can be used to prevent buckling of the concrete slab during fire. PIR and foam glass are the most suitable, because these lightweight non-combustible insulation materials are compression and moisture resistant. The extra self-weight compared to the EPS fill element is 0.13 kN/m$^2$ for a foam glass fill element.

The following design measures are required to design the concrete top slab to resist buckling: sagging- and hogging reinforcement, a larger concrete cross-section and higher concrete class. The high slenderness and initial deflection cause that 2$^{nd}$ order effects are large. A calculation where the bending stiffness is taken less conservatively could provide a more economic design. The extra self-weight compared to the EPS fill element is 1.31 kN/m$^2$.

Both solutions provide a fire resistance of 90 minutes. Nevertheless the increase in self-weight for a larger concrete top flange is significant. It is recommended to support the concrete top flange with foam glass elements. For a grounded conclusion a cost analysis and a more detailed calculation are necessary.

9.6. **ANCHORAGE OF THE TENSION REINFORCEMENT**

Anchorage of the reinforcement at the support is required to provide force equilibrium between the concrete compression arch and the steel tensile tie (figure 9.15). The force in the concrete compression arch are directly transferred to the support as illustrated in figure 9.16. Here is anchorage possible. In this section the anchorage for the tension reinforcement is designed.

Assumption for the support condition:
- ASB300 series: support on an ASB beam (right figure 9.16);
- minimum support for the deck is 50 mm (requirement for current deep decks [8]);
- the tension force in the bottom reinforcement is: $N_t = 43$ kN.

9.6.1. **THE REQUIRED ANCHORAGE LENGTH**

Given is that:
- B500 steel reinforcing bar ø16: $\sigma_{sd} = \sigma_{s,max} = \frac{N_t}{A_t} = \frac{43 \cdot 10^3}{201} = 214$ N/mm$^2$;
9.6. ANCHORAGE OF THE TENSION REINFORCEMENT

- concrete class C20/25: \( f_{ct,k,0.05} = 1.5 \text{ N/mm}^2 \);
- design axial strength of concrete: \( f_{ctd} = \frac{a_2 f_{ct,k,0.05}}{1.0} = 1.0 \cdot 1.5 = 1.5 \text{ N/mm}^2 \);
- straight anchorage on ASB300 series: \( l_{\text{provided}} = 136.5 \text{ mm} \).

Design value of the ultimate bond stress [6, p.133]:

\[
 f_{bd} = 2.25 \eta_1 \eta_2 f_{ctd} = 2.25 \cdot 1.0 \cdot 1.0 \cdot 1.5 = 3.37 \text{ N/mm}^2
\]

Where:

\( \eta_1 = 1.0 \) for ‘good’ bond conditions (illustrated in figure 8.2 of EC2);
\( \eta_2 = 1.0 \) for \( \varnothing<32 \text{ mm} \).

The basic required anchorage length \( l_{b,rqd} \) follows from horizontal force equilibrium (figure 9.17).

- bond strength: \( H = l_{bond} \cdot \pi \varnothing \cdot f_{bd} \) (anchorage surface \( \times \) bond strength);
- tension force: \( N_s = \pi \varnothing^2 \cdot \sigma_{s,max} \) (surface bar \( \times \) tension stress).

This gives the following equation:

\[
\sum H = 0: \quad N_s = H \\
\frac{1}{4} \varnothing^2 \pi \cdot \sigma_{s,max} = l_{bond} \cdot \pi \varnothing \cdot f_{bd}
\]

\( \Rightarrow l_{b,rqd} = \frac{\varnothing}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} = \frac{16}{4} \cdot \frac{214}{3.37} = 254 \text{ mm} \)

The required design anchorage length:

\[
l_{bd} = a_1 a_2 a_3 a_4 a_5 l_{b,rqd} \geq l_{b,\text{min}} = \max(0.3l_{b,rqd} ; 10\varnothing ; 100 \text{ mm})
\]

\[
= 1.0 \cdot 254 = 254 \text{ mm} \geq \max(77 \text{ mm} ; 160 \text{ mm} ; 100 \text{ mm})
\]

Figure 9.15: Load-carrying model of the floor during fire

Figure 9.16: Forces at anchorage (left) and assumed end support condition (right)
Conservatively is $\alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5$ taken as 1.

There is not sufficient length to provide the required anchorage, because $l_{bd} > l_{provided}$. However, different design solutions are available to provide sufficient anchorage at the support (figure 9.18).

- anchorage with bend at the end of the bar or a separate hook;
- anchorage where bars are linked through holes in the steel beam;
- anchorage with a separate hook over the steel beam.

The most suitable type depends on the end support conditions. Below is elaborated on the design of anchorage with a bend at the end of the bar.

### 9.6.2. ANCHORAGE WITH A BEND AT THE END OF THE BAR

The available anchorage length will increase if the reinforcement bar is bend at the end of the bar (figure 9.19). The required cover is $c_d = \varnothing + \Delta \text{dev} = 16 + 5 = 21$ mm, to ensures sufficient bond strength and allow for deviation [6, p.49-50].

Eurocode 2 gives design rules for the anchorage with a bend and provides an expression for the minimum corner radius $\varnothing_{m,\text{min}}$ [6, p.132].

- corner radius $r \geq \varnothing \cdot 2$ mm for $\varnothing 16$;
- anchorage length after the corner: $l_2 \geq \varnothing \cdot 5$ mm.

\[
\varnothing_{m,\text{min}} = \frac{F_{bt} \left[ \left( \frac{l_1}{a_b} \right) + \left( \frac{1}{20} \right) \right]}{f_{cd}} = \frac{43 \cdot 10^3 \left[ \left( \frac{1}{29} \right) + \left( \frac{1}{2.16} \right) \right]}{20} = 141.3 \text{ mm}
\]

Neglect bond strength before the corner $l_1$ in figure 9.19 and assume $F_{bt} = 43 \cdot 10^3$ N. Here is $a_b$ the distance from the face of the member to the centre line of the bar: $a_b = c_d + \varnothing/2 = 21 + 16/2 = 29$ mm. The lengths before ($l_1$) and after ($l_2$) the corner are:

\[
l_1 = 136.5 - c_d - \varnothing - r = 136.5 - 29 - 16 - \frac{141.3}{2} = 21 \text{ mm}
\]

\[
l_2 = 262 - c_d - \varnothing - r = 262 - 29 - 16 - \frac{141.3}{2} = 146.4 \text{ mm}
\]

Before the corner a bond strength of $H = l_1 \cdot \pi \varnothing \cdot f_{bd} = 21 \cdot \pi \cdot 16 \cdot 3.37 \cdot 10^{-3} = 3.5$ kN is transferred. The tensile force at start of the bend is therefore: $F_{bt} = 43 - 3.5 = 39.5$ kN. Iteration gives:
9.6. ANCHORAGE OF THE TENSION REINFORCEMENT

Figure 9.19: Bend at end of bar: principle (left), available space, and dimension of bend (right)

- \( \varnothing_{m,\text{min}} = 120 \text{ mm} \);
- \( l_1 = 39 \text{ mm} \);
- \( l_2 = 165 \text{ mm} \);
- \( l_0 = \frac{\pi \cdot 120}{2} = 94 \text{ mm} \);
- \( l_{\text{provided}} = l_1 + l_0 + l_2 = 39 + 94 + 165 = 298 \text{ mm} \).

There is sufficient anchorage length: \( l_{\text{provided}} = 298 \text{ mm} > l_{\text{required}} = 254 \text{ mm} \).

9.6.3. DESIGN AND INSTALLATION

The installation of a reinforcement bar with a bend at the end of the bar is not practical. This is especially the case if a first cast is prefabricated, because the individual elements cannot be stacked (figure 9.20). A solution is to install the bend bar as a separate element and design the required lap length of the tension reinforcement bar and the hook.

The design lap length is obtained with the Eurocode [6, p.139].

\[
l_0 = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 l_{b,\text{reqd}} \geq l_{0,\text{min}} = \max(0.3 \alpha_6 l_{b,\text{reqd}} ; 15\varnothing ; 200 \text{ mm})
\]

\[
= 1.0 \cdot 254 = 254 \text{ mm} > l_{0,\text{min}} = \max(115 ; 240 ; 200 \text{ mm}) = 240 \text{ mm}
\]

Where \( \alpha_6 = 1.5 \) is conservative.

The separate hook is installed with the lap on top of the tension reinforcement bar. The lap length should be at least 254 mm. The provide length \( l_2 \) decreases with \( \varnothing \), but there is still sufficient anchorage to transfer the bond stresses. The design of the anchorage is illustrated in figure 9.21, where:

- \( \varnothing_{\text{hook}} = 16 \text{ mm} \);
- \( l_0 \geq 254 \text{ mm} \);
The total length of the anchorage hook is \( l = l_0 + l_1 + l_2 = 254 + 39 + 94 + 149 = 536 \text{ mm} \). These dimensions are valid if the floor is supported by an asymmetric beam from the ASB300 series.

**9.7. CONCLUSION**

During fire is expected that the thin-walled steel deck will heat up quickly and lose strength at a low fire load. In this fire design is assumed that the deck is not able to transfer forces and that the fill element is burned (or gassed). A new load-carrying principle will develop. In this load-carrying principle are the loads in the floor carried by a compression arch, formed by the concrete top flange, and a tensile tie, formed by the reinforcement bar in the bottom of the ribs. In this chapter these elements are designed to provide a minimum mechanical fire resistance of 90 minutes.

Material properties are reduced to take the effect of the fire into account. The temperature of the tension reinforcement, that is protected by a small concrete beam, is estimated with temperature profiles. The reduced concrete cross-section is based on the 500° isotherms. The concrete slab is reduced 22 mm for 60 minutes and 29 mm for a 90 minute fire load.

It is shown that the floor slab will not fail due to bending or shear. A \( \phi 16 \text{ mm} \) and \( \phi 20 \text{ mm} \) bar are required for 60 minutes and 90 minutes fire resistance, respectively.

The very slender concrete top flange is sensitive for buckling. Buckling of the concrete top flange during fire is prevented in two ways; a non-combustible fill element provides support or the concrete top flange is designed to resist buckling. Both solutions provide a fire resistance of 90 minutes. Nevertheless the increase in self-weight for a larger concrete top flange is significant. It is recommended to support the concrete top flange with foam glass fill element. For a grounded conclusion a cost analysis and a more detailed calculation are necessary.

Anchorage of the reinforcement at the support is required to provide force equilibrium between the concrete top flange and the tension reinforcement. Different types of anchorage are suitable: a bend at the end of the bar, a separate hook, bars through holes in the steel beam, or a separate hook over the steel beam. The most suitable type depends on the end support conditions.

The floor slab is designed for a mechanical fire resistance of 90 minutes. However, fire tests are always obligatory to verify calculations. Besides that, the actual resistance is also determined by the insulation and integrity during fire, both require testing.
Design of the Steel Deck

Decisions in the previous chapters have led to adjustments in the design of the steel deck. Here an overview is given of these adjustments and the structural properties of the deck are calculated. The loads during construction are determined to verify the load-carrying capacity of the deck. In addition, the deck dimensions are structurally optimized to provide maximum load-carrying capacity.

10.1. Dimensions of the Deck

The dimensions of the steel deck are specified in this section. Decisions from the previous chapters give the following requirements.

- less height of the web embossment → height embossment $s_e = 150$ mm;
- longitudinal stiffeners symmetrical in the top- and bottom of the web;
- more concrete in the ribs near the end supports → ribs fully cast up to 600 mm from the end supports;
- heavier fill element in the ribs → density foam glass $\rho_{FOAM} = 165$ kg/m$^3$;
- stiffener in the top flange of the deck → width $b_r = 20$ mm and height $h_s = 6$ mm.

Note that the smaller web embossment, stiffeners in the top flange, and longitudinal stiffeners will increase the load-carrying capacity of the deck. However, the extra concrete and the heavier fill element will result in higher loads during construction. The dimensions of the deck are illustrated in figure 10.1.

Figure 10.1: Single deck panel (left) and dimensions in symbols for half a deck (right)
The deck dimensions (from chapter 6.5):

- height of the deck: \( h_p = 220 \text{ mm} \);
- width of the top flange: \( b_0 = 60 \text{ mm} \);
- angle of the web: \( \nu = 80^\circ \);
- design thickness of the steel: \( t_0 = 1.21 \text{ mm} \) (for the whole cross-section);
- width of the panel: \( b_d = 286 \text{ mm} \).

The dimensions of the longitudinal stiffeners are:

- height of the stiffener: \( h_{sa} = h_{sb} = 10 \text{ mm} \);
- angle of the stiffener: 45°;
- the height between the flange and the begin of the stiffener: \( h_a = h_b = 25 \text{ mm} \).

Where \( h_a, h_b, h_{sa}\) and \( h_{sb}\) are chosen such that the height of the embossments is at least 150 mm. The edge stiffener is assumed as \( c = 20 \text{ mm} \). The slant height \( s_w \), the width of the web \( w_w \), and the width of the bottom flange \( b_u \) are calculated with the dimensions of the deck.

\[
\begin{align*}
    s_w &= \left( \frac{h_a + s_c + h_b}{\sin(v)} + \frac{(h_{sa} + h_{sb})}{\sin(\pi/4)} \right) \frac{(25 + 150 + 25)}{\sin(\pi/4)} + \frac{(10 + 10)}{\sin(\pi/4)} = 231.4 \text{ mm} \\
    w_w &= \left( \frac{h_a + s_c + h_b}{\tan(v)} + \frac{(h_{sa} + h_{sb})}{\tan(\pi/4)} \right) \frac{(25 + 150 + 25)}{\tan(\pi/4)} + \frac{(10 + 10)}{\tan(\pi/4)} = 55.3 \text{ mm} \\
    b_u &= b_d - b_0 - 2 \cdot w_w = 286 - 60 - 2 \cdot 55.3 = 115.5 \text{ mm}
\end{align*}
\]

### 10.2. CROSS-SECTIONAL PROPERTIES

The cross-sectional properties are obtained using the following assumptions.

- neglect embossed area → no contribution to the bending stiffness and strength;
- all other parts are taken into account and assumed to be 100% effective → no reduction for local buckling

Eurocode 3 part 1 appendix C1 gives a method to determine the cross-section properties for thin-walled open cross-sections. This method is used to obtain the cross-sectional properties of the steel deck. Expressions are found in appendix G.1. Figure 10.2 illustrates nodes \( n = 1..16 \) to describe the geometry of half the deck panel. This description is used to calculate the cross-section properties.

The following cross-sectional properties are found.

\[
\begin{align*}
    A_{gr} &= 3190 \text{ mm}^2/\text{m}; \\
    A_{pe} &= 1902 \text{ mm}^2; \\
    I_{eff} &= 2011 \cdot 10^4 \text{ mm}^4/\text{m}; \\
    e_c &= 101.4 \text{ mm}; \\
    e_t &= 124.6 \text{ mm}; \\
    W_{el} &= 161 \cdot 10^3 \text{ mm}^3/\text{m}; \\
    M_{el,Rd} &= 56.4 \text{ kNm}/\text{m}.
\end{align*}
\]

Only the value of gross cross-section \( A_{gr} \) is exact. \( A_{gr} \) is used to calculate the loads during construction.

Note that these properties are used to validate the cross-section properties of the reduced effective cross-section. Section 10.4 elaborates on this.
10.3. **LOADS DURING CONSTRUCTION**

In this section the loads during construction are determined. Firstly, the self-weight of the concrete, the fill element, the reinforcement and the deck is calculated. Secondly, the loads over the span are obtained and is the effect of ponding is included. Finally, the load $q_d(x)$ over the span is given.

10.3.1. **SELF-WEIGHT**

The weight of the floor is not uniform over the length of the span, due to the changing dimensions of the concrete cross-section. In figure 10.3a is:

- $L = 7200$ mm, the total span of the floor;
- $L_{red} = 6000$ mm, length of the fill element;
- $L_x = 600$ mm, no fill element near the supports.

The weight of the floor, taking the non-uniform weight distribution into account, is calculated using two cross-sections, A and C illustrated in figure 10.3b.

Material densities to calculate the weight are:

- wet concrete: $\rho_{c, wet} = 25$ kN/m³;
- lightweight element: $\rho_{FOAM} = 165$ kg/m³ = 1.62 kN/m³;
- structural steel: $\rho_s = 7850$ kg/m³ = 77 kN/m³.

In this section the self-weight per construction element (deck $G_p$, concrete $G_c$, fill element $G_{red}$, and reinforcement $G_r$) is calculated per rib width over a given length ($L$, $L_x$ or $L_{red}$).

**THE CONCRETE AND FILL ELEMENT**

The possible amount of weight reduction in the ribs is determined by the height of concrete bottom flange $h_2$. In chapter 9 is given that $h_2 = 60$ mm. The height of the fill element is than:

$$h_{red} = h_p - h_2 = 220 - 60 = 160$$ mm (over length $L_{red}$)

The cross-section is slightly simplified in figure 10.4 to calculate the weight of the floor.
The angle of the web is here:

\[ \alpha = \arctan \left( \frac{h_p}{w_w} \right) = \arctan \left( \frac{220}{55.3} \right) = 75.9° \]

The area of the concrete \( A_c \) and fill element \( A_{red} \) are calculated to determine the volume and weight of the concrete and the fill element per cross-section.

**Cross-section A (no lightweight element):**

\[ A_c = h_p \left( b_u + \frac{h_p}{\tan(\alpha)} \right) + b_d h_c = 220 \left( 115.5 + \frac{220}{\tan(75.9)} \right) + 286 \cdot 60 = 54727 \text{ mm}^2/\text{rib} \]

\[ V_c = A_c \cdot \frac{2L_x}{1000^3} = 54727 \cdot \frac{2 \cdot 600}{1000^3} = 0.066 \text{ m}^3/\text{rib} \]

\[ G_c = V_c \cdot \rho_{cw} = 0.066 \cdot 25 = 1.642 \text{ kN/rib (distributed over } 2L_x) \]

**Cross-section C (at lightweight element):**

\[ A_{c,red} = \left( b_u + 2 \cdot \frac{h_2/2}{\tan(\alpha)} \right) \cdot h_2 + b_d h_c = \left( 115.5 + 2 \cdot \frac{60/2}{\tan(75.9)} \right) \cdot 60 + 286 \cdot 60 = 24994 \text{ mm}^2/\text{rib} \]

\[ V_{c,red} = A_{c,red} \cdot \frac{L_{red}}{1000^3} = 24994 \cdot \frac{6000}{1000^3} = 0.150 \text{ m}^3/\text{rib} \]

\[ G_{c,red} = V_{c,red} \cdot \rho_{cw} = 0.150 \cdot 25 = 3.75 \text{ kN/rib (distributed over } L_{red}) \]

\[ A_{red} = \left( 2 \cdot \frac{h_2 + \left( \frac{h_{red}}{2} \right)}{\tan(\alpha)} + b_u \right) \cdot h_{red} = \left( 2 \cdot \left( 60 + \frac{160}{2} \right) \right) \cdot 115.5 \cdot 160 = 29733 \text{ mm}^2/\text{rib} \]

\[ V_{red} = A_{red} \cdot \frac{L_{red}}{1000^3} = 29733 \cdot \frac{6000}{1000^3} = 0.178 \text{ m}^3/\text{rib} \]

\[ G_{red} = V_{red} \cdot \rho_{FOAM} = 0.178 \cdot 1.62 = 0.289 \text{ kN/rib (distributed over } L_{red}) \]

---

(a) Cross-section of the floor in direction of the span

(b) Cross-section of ribs: A, B and C

Figure 10.3: Cross-sections used to determine the weight of the floor
The deck

The area of the gross cross-section of the deck is $A_{gr} = 3190 \text{ mm}^2/\text{m}$. The weight of a single deck panel is calculated.

$$ A_p = \frac{A_{gr}}{1000} \cdot b_d = \frac{3190}{1000} \cdot 286 = 912 \text{ mm}^2/\text{rib} $$

$$ V_p = A_p \cdot \frac{L}{1000^3} = 912 \cdot \frac{7200}{1000^3} = 0.0066 \text{ m}^3/\text{deck panel} $$

$$ G_p = V_p \cdot \rho_s = 0.0066 \cdot 77 = 0.506 \text{ kN/\text{deck panel (distributed over L)}} $$

The reinforcement

The amount of reinforcement is calculated, because the contribution is relatively small the total weight of the reinforcement is uniformly distributed over the floor.

1. standard reinforcement mesh: $\phi_1 8 - 200$ mm over the whole floor span
   - $A_{s,1} = 2 \cdot \frac{b_d}{200} \cdot \frac{\pi \phi_1^2}{4} = 2 \cdot \frac{286}{200} \cdot \frac{\pi 8^2}{4} = 144 \text{ mm}^2$ over $l_1 = 7200$ mm

2. hogging moment reinforcement: $1\phi_2 10$ mm/rib with a length of 1 m over each support
   - $A_{s,2} = 1 \cdot \frac{\pi \phi_2^2}{4} = 1 \cdot \frac{\pi 10^2}{4} = 79 \text{ mm}^2$ over $l_2 = 2000$ mm

3. the hook used for anchorage: $\phi_4 16$ mm/rib with a length of 550 mm at each end support
   - $A_{s,4} = 1 \cdot \frac{\pi \phi_4^2}{4} = \frac{\pi 16^2}{4} = 201 \text{ mm}^2$ over $l_4 = 900$ mm

4. the tension reinforcement in the bottom: $\phi_5 16$ mm/rib over the the floor span
   - $A_{s,5} = \frac{\pi \phi_5^2}{4} = \frac{\pi 16^2}{4} = 201 \text{ mm}^2$ over $l_5 = 7200$ mm

The total volume $V_s$ and weight $G_s$ of the reinforcement is:

$$ V_s = \sum_{i=1}^{4} A_{s,i} \cdot \frac{l_i}{1000^3} = (144 \cdot 7200 + 79 \cdot 2000 + 201 \cdot 900 + 201 \cdot 7200) / 1000^3 = 0.0028 \text{ m}^3/\text{rib} $$

$$ G_s = V_s \cdot \rho_s = 0.0028 \cdot 77 = 0.218 \text{ kN/rib (distributed over L)} $$

10.3.2. Loads

The self-weight of the floor is calculated for three regions of the span (figure 10.5). Here the uniform distributed loads $q$ (kN/m$^2$) per region are calculated in SLS, $\gamma_g=1$ and $\gamma_q=1$, and the construction loads is
10.3.3. Deflection and ponding

The deflection of the deck is calculated to determine the effect of ponding. The loads in SLS take the weight of the deck, the reinforcement, the fill element and the wet concrete into account.

The load, as illustrated in figure 10.5, is written in one function over the length of the span \( x \) with a Heaviside-function. The Heaviside-function \( H(x) \) has the property that it is 0 for all \( x < 0 \) and 1 for all \( x \geq 1 \).

\[
q(x) = q_1 + q_2 \cdot (H(x - L_x) - H(x - (L - L_x))) + q_3 \cdot ((H(x - 0) - H(x - L_x)) + (H(x - (L - L_x)) - H(x - (L)))
\]

Here \( q(x) \) is given for \( 0 \leq x \leq 7200 \). The deflection is obtained by solving the differential equation (DE) for a bending beam.

\[
EI \frac{d^4w}{dx^4} = q(x)
\]

The derivation of this DE is given in appendix A.3. This 4\(^{th}\) order DE is solved with 4 boundary conditions. Two for each support, where the moments and deflections are equal to 0. The bending stiffness of the deck is, \( EI = E \cdot I_{eff} = 2.1 \cdot 10^5 \cdot 2011 \cdot 10^4 = 4.32 \cdot 10^{12} \text{ Nmm}^2 \). This DE is solved in Maple and the expressions and Maple sheet are found in appendix G.3.

The largest deflection of the deck under this load is 23.1 mm. The effect of ponding is included as a uniform load of \( q_0 = 0.7 \cdot w_{max} \rho_c w \cdot 10^{-3} = 0.7 \cdot 23.1 \cdot 25 \cdot 10^{-3} = 0.41 \text{ kN/m}^2 \), over the whole length of the span. The final load \( q_d(x) \) is obtained.

\[
q_d(x) = \begin{cases} 
\gamma_g (q_0 + q_1) + \gamma_q q_i & \text{for } 0 \leq x \leq L \\
\gamma_g q_2 & \text{for } L_x \leq x \leq (L - L_x) \\
\gamma_g q_3 & \text{for } 0 \leq x < L_x \text{ and } (L - L_x) < x \leq L 
\end{cases}
\]

Where:

\[
q_0 = 0.41 \text{ kN/m}^2; \\
q_1 = 0.35 \text{ kN/m}^2;
\]
10.4. EFFECTIVE CROSS-SECTIONAL PROPERTIES

In this section the amount of effective steel in the cross-section is determined. Eurocode 3 part 3 (EC3-3) gives, in addition with Eurocode 3 part 5, design rules for the elements in the compression zone of the deck. This section elaborates on the effect of the intermediate- and edge stiffener in the top flange and the longitudinal stiffeners in the web. With this the effective cross-sectional properties are obtained.

In appendix G.2 the design rules of EC3-3 are used to calculate the effective areas of the compression elements in the deck. A brief overview is given of the results. The effective area of the intermediate stiffener of the top flange is determined with a reduced thickness \( t_{\text{red}} = 1.17 \text{ mm} \). The effective area of the edge stiffener of the top flange is determined with a reduced length \( c_{\text{eff}} = 16.2 \text{ mm} \). The web stiffener is fully effective and no reductions are required.

EC3-3 prescribe that in a SLS calculation it is allowed to calculate the effective section properties of the stiffeners with the design thickness \( t_0 = 1.21 \text{ mm} \). This means that the intermediate flange stiffener is not reduced when the deflections are verified.

The effective cross-sectional properties are obtained with the same method used in section 10.2. The reduced thickness of the intermediate stiffener and the reduced length of the edge stiffener are taken into account. The following effective cross-sectional properties are found used for a SLS calculation.

\[
A_{pe} = 1870 \text{ mm}^2/\text{m}; \\
I_{\text{eff}} = 1991 \cdot 10^4 \text{ mm}^4/\text{m}; \\
e_c = 102.7 \text{ mm}; \\
e_t = 123.3 \text{ mm}.
\]

The following effective cross-sectional properties are found used for a ULS calculation.

\[
W_{el} = 161 \cdot 10^3 \text{ mm}^3/\text{m}; \\
M_{el,Rd} = 56.4 \text{ kNm}/\text{m}.
\]

The values are comparable with the cross-sectional properties found in section 10.2. The reductions to obtain the effective cross-sectional properties are small. The reason is that the elements in the compression zone are relatively short and local buckling has little effect.

10.5. DESIGN VERIFICATIONS

In this section the load-carrying capacity of the deck design is verified. First the deflections are verified in serviceability limit state. Secondly, the moment- and shear resistance are verified in ultimate limit state.

10.5.1. SERVICEABILITY LIMIT STATE

The deflection of the deck during construction is verified.

The second moment of area of the effective cross-section was found in section 10.4. Here used to obtain the bending stiffness of the deck.

\[
EI = E \cdot I_{\text{eff}} = 2.1 \cdot 10^5 \cdot 1991 \cdot 10^4 = 4.18 \cdot 10^{12} \text{ Nmm}^2
\]

The loads and maximum deflection are calculated with the method given in section 10.3. The maximum deflection of the deck is \( \delta_0 = 26.4 \text{ mm} \).
Unity check:

\[ UC = \frac{\delta_0}{\delta_{s,\text{max}}} = \frac{26.4}{40} = 0.66 \]

The deck design has sufficient bending stiffness to limit the deflections during construction.

10.5.2. **ULTIMATE LIMIT STATE**

The moment- and shear resistance of the deck during construction are verified.

The loads for ULS verifications are determined using the same approach as in section 10.3. However, the construction load \( q_c \) of 0.75 kN/m² is included and the load factors are adapted to \( \gamma_g = 1.2 \) and \( \gamma_q = 1.5 \). The maximum bending moment and shear force per meter deck width are:

\[ M_{Ed} = 32.8 \text{kNm/m}; \]
\[ V_{Ed} = 19.7 \text{kN/m}. \]

**MOMENT RESISTANCE**

The effective elastic moment resistance of the deck is \( M_{el,Rd} = 56.4 \text{kNm/m} \).

Unity check:

\[ UC = \frac{M_{Ed}}{M_{el,Rd}} = \frac{32.8}{56.4} = 0.58 < 1 \]

The deck design has sufficient moment resistance.

**SHEAR RESISTANCE**

The shear resistance of the steel deck is already verified in chapter 7. The webs of the deck are designed to resist a shear force of at least 5 kN/web (\( V_{Rd} = 35 \text{kN/m width} \)).

Unity check:

\[ UC = \frac{V_{Ed}}{V_{Rd}} = \frac{19.7}{35} = 0.56 < 1 \]

The deck design has sufficient shear resistance.

10.6. **DECK DESIGN**

In this section the structural performance of the deck design is evaluated and combined with other design requirements redesigned. The self-weight of the steel-concrete floor slab is calculated and determined if the steel-concrete composite floor slab has sufficient load-carrying capacity. Finally, two additional design improvements are given.

10.6.1. **STRUCTURAL OPTIMIZATION**

From the previous sections is obtained that the compression zone of the deck is almost 100% effective. However, in this design the location of the neutral axis not optimal. The height of the compression zone is \( e_c = 102.7 \text{mm} \) and the height of the tension zone is \( e_t = 123.3 \text{mm} \). The position of the neutral axis is most favourable at half the height of the deck. In this situation the largest bending stiffness \( EI \) and elastic bending moment resistance \( M_{el,Rd} \) is provided for the least amount of steel \( A_{gr} \).

The position of the neutral axis is influenced positively for an increased amount of steel in the tension zone (n.a. shifts to the middle). The width of the bottom flange is increased to provide more steel in the tension zone. The calculations and methods used in this chapter are iterated to optimize the structural performance of this deck. The following effective cross-sectional properties are found for a deck width of 326 mm.
The neutral axis of this design is very favourable. Note that the amount of steel per meter width $A_{gr}$ is decreased, but the bending stiffness $I_{eff}$ and the elastic bending resistance $M_{el,Rd}$ are increased. However, the choice of the width of the deck panel is not only determined by the structural performance of the deck during construction.

The deck panel should have a width that is suitable for building practice. Design multiplications of 600 mm are here common. Besides that, the weight of a single deck panel should not exceed 50 kg. Therefore is chosen for a deck width of 300 mm. The properties of this deck panel are:

$$
\begin{align*}
A_{gr} &= 3098 \text{ mm}^2/\text{m}; \\
A_{pe} &= 1839 \text{ mm}^2/\text{m}; \\
I_{eff} &= 1981 \cdot 10^4 \text{ mm}^4/\text{m}; \\
e_c &= 106.5 \text{ mm}; \\
e_t &= 119.5 \text{ mm}; \\
M_{el,Rd} &= 57.9 \text{ kNm/m}.
\end{align*}
$$

This deck panel has sufficient load-carrying capacity, has a suitable dimensions, and weighs 52 kg. The weight of a single panel exceeds its maximum.

The verifications in section 10.5 show that this deck has sufficient load-carrying capacity during construction ($UC \leq 0.57$). Therefore is recommended to choose a thinner steel thickness that still provides sufficient capacity, but reduces the weight per deck panel. Note that the compression elements of a deck with a thinner steel plate have a higher slenderness and are more susceptible for local buckling. The properties of this deck panel are:

$$
\begin{align*}
t &= 1.0 \text{ mm (thickness of the steel sheet)}; \\
b_d &= 300 \text{ mm}^2/\text{m (width of the deck panel)}; \\
A_{gr} &= 2458 \text{ mm}^2/\text{m}; \\
A_{pe} &= 1441 \text{ mm}^2/\text{m}; \\
I_{eff} &= 1559 \cdot 10^4 \text{ mm}^4/\text{m}; \\
e_c &= 107.5 \text{ mm}; \\
e_t &= 118.5 \text{ mm}; \\
M_{el,Rd} &= 45.9 \text{ kNm/m}.
\end{align*}
$$

The self-weight of a single deck panel is 42 kg. In this design are the other dimensions similar to the previous designs of this chapter. This design is verified and provides sufficient load-carrying capacity during construction ($UC \leq 0.87$), where the deck deflection is governing ($\delta_0 = 34.7 \text{ mm}$). Note that this deck design is structurally not optimal, because other design specifications are more decisive.

The choice of a thinner steel sheet and wider ribs has influence on the design of the steel-concrete composite floor.

- higher shear loads and less capacity per web $\rightarrow$ verify the shear capacity of webs ($V_{Rd} \geq V_{Ed}$);
- wider ribs means that less dowels are provided $\rightarrow$ verify the longitudinal shear capacity ($V_{l,Rd} \geq V_{l,Ed}$).

In the next part is the self-weight of the steel-concrete composite floor calculated to determine if the vertical and/or longitudinal shear capacity of the floor are critical.
10.6.2. **Self-weight of the floor slab**

The self-weight of the steel-concrete composite floor slab determined. For simplicity is assumed that the weight of the floor is uniform over the whole length of the span.

Firstly, the weight of the deck \(G_p\), the reinforcement \(G_s\) and the fill element \(G_{red}\) are written in a uniform load over the floor span.

\[
g_p = \frac{G_p}{L} \cdot \frac{1000}{b_d} = \frac{0.22}{7200/1000} \cdot \frac{1000}{400} = 0.19 \text{ kN/m}^2
\]

\[
g_s = \frac{G_s}{L} \cdot \frac{1000}{b_d} = \frac{0.41}{7200/1000} \cdot \frac{1000}{400} = 0.10 \text{ kN/m}^2
\]

\[
g_{red} = \frac{G_{red}}{L} \cdot \frac{1000}{b_d} = \frac{0.31}{7200/1000} \cdot \frac{1000}{400} = 0.14 \text{ kN/m}^2
\]

Secondly, the load during construction \(q_d(x)\) is written as a uniform load over the whole span.

\[
q_d = \frac{(q_0 + q_1)L + q_2L_{red} + 2q_3L_x}{L} = \frac{(0.51 + 1.76) \cdot 7.2 + 0.90 \cdot 6 + 2 \cdot 3.39 \cdot 0.6}{7.2} = 3.07 \text{ kN/m}^2
\]

With this is the weight of the wet concrete obtained:

\[
g_{c,wet} = q_d - g_p - g_s - g_{red} = 3.07 - 0.19 - 0.10 - 0.14 = 3.14 \text{ kN/m}^2
\]

The weight of the dry concrete is calculated with the density of wet- and dry concrete. The density of dry concrete is \(\rho_{c, dry} = 2400 \text{ kg/m}^3\).

\[
\rho_c = \frac{2400 \cdot 9.81}{1000} = 23.54 \text{ kN/m}^3
\]

\[
g_{c, dry} = \frac{g_{c, wet} \cdot \rho_{c, dry}}{\rho_{c, wet}} = \frac{3.14 \cdot 23.54}{25} = 2.96 \text{ kN/m}^2
\]

The self-weight of the steel-concrete composite floor is:

\[
g_k = g_p + g_s + g_{red} + g_{c, dry} = 0.19 + 0.10 + 0.14 + 2.96 = 3.4 \text{ kN/m}^2
\]

10.6.3. **Steel-concrete composite floor**

In chapter 7 the web of the deck is designed to resist a maximum shear load of \(V_{Rd} = 5117 \text{ N per web}\). In chapter 8 dowels on the top flange are designed to resist a maximum moment of 22.5 kNm per rib. Both designs were based on a design load of 10.4 kN/m².

The design load for the redesigned steel deck is:

\[
q_d = \gamma_g g_k + \gamma_q q_k = 1.2 \cdot 3.4 + 1.5 \cdot 4.0 = 10.1 \text{ kN/m}^2
\]

The moment per rib is \(M_{Ed} = 19.6 \text{ kNm}\). The longitudinal shear resistance is therefore sufficient.

The maximum shear load on a single web is \(V_{Ed} = 4545 \text{ N per web}\). The reduced thickness of the steel sheet has influence on the design shear resistance of the deck. The calculations of chapter 7 need to be performed again for \(t = 1.0 \text{ mm}\) to determined if the web of the deck has sufficient capacity to resist this load.

The section properties of the stiffened- and unstiffened part are recalculated with the methods given in chapter 7 and appendix D.1. The required Euler buckling resistance \(N_{cr}\) and the required length of the embossment \(l\) are determined for different sheet thicknesses \(t\) in table 10.1. Here is found that a sheet thickness of 1.0 mm is not sufficient to prevent buckling of the web.

Therefore is chosen to use a sheet with a thickness of 1.20 mm to provide sufficient shear resistance. The properties of this deck panel are:

\[
t = 1.20 \text{ mm (thickness of the steel sheet)};
\]

\[
b_d = 300 \text{ mm (width of the deck panel)};
\]
10.7. CONCLUSION

In this chapter the steel deck is redesigned. Decisions made in previous chapters have let to adjustments in the design of the steel deck. These adjustments are incorporated and the structural properties of the deck are calculated.

In the deck design the compression zone is stiffened to increase the amount of effective steel. With Eurocode 3 part 3 and 5 is found that this design is very effective. Local buckling has little effect, because the compression

<table>
<thead>
<tr>
<th>Thickness</th>
<th>t = 1.0 mm</th>
<th>t = 1.20 mm</th>
<th>t = 1.25 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stiffened part</td>
<td>$A_{gr}$ mm$^2$</td>
<td>217.0</td>
<td>262.2</td>
</tr>
<tr>
<td></td>
<td>$I_{gr}$ mm$^4$</td>
<td>116.1</td>
<td>208.5</td>
</tr>
<tr>
<td></td>
<td>$A_{eff,1}$ mm$^2$</td>
<td>110.6</td>
<td>150.2</td>
</tr>
<tr>
<td>Unstiffened part</td>
<td>$I_{gr}$ mm$^4$</td>
<td>16.2</td>
<td>28.6</td>
</tr>
<tr>
<td></td>
<td>$A_{eff,2}$ mm$^2$</td>
<td>78.7</td>
<td>111.1</td>
</tr>
<tr>
<td>Required buckling strength</td>
<td>$N_{cr}$ N</td>
<td>5643</td>
<td>5416</td>
</tr>
<tr>
<td>Required length embossment</td>
<td>$l$ mm</td>
<td>&gt;220</td>
<td>139</td>
</tr>
</tbody>
</table>

Table 10.1: Section properties, required buckling strength, and required length of the web embossment for different sheet thicknesses $t$

$A_{gr} = 2970$ mm$^2$/m;
$A_{pe} = 1759$ mm$^2$/m;
$I_{eff} = 1896 \cdot 10^4$ mm$^4$/m;
$e_c = 106.7$ mm;
$e_t = 119.3$ mm;
$M_{el,Rd} = 55.5$ kNm/m.

The self-weight of a single deck panel is 50 kg and this deck can therefore be man-handled. The deck deflection during construction is $\delta_0 = 28.1$ mm. This deck panel meets all criteria that were given in 3.

10.6.4. STEEL DECK DESIGN

Two design improvements are here discussed that were not addressed in this chapter.

First is suggested to bent the corners in the top flange using a larger corner radius. This is a current development in the design of steel decks and is used to increase the amount of effective steel in the compression zone [8]. The result is a deck with a higher bending stiffness. An example of such a deck is the Comflor 95 illustrated in figure 10.6.

Secondly is suggested two design two longitudinal stiffeners in the bottom flange. The advantage of these stiffeners is that the tension reinforcement is easily positioned during the installation. These stiffeners will not increase the amount of the effective steel, because there located in the tension zone of the deck.

Secondly is suggested two design two longitudinal stiffeners in the bottom flange. The advantage of these stiffeners is that the tension reinforcement is easily positioned during the installation. These stiffeners will not increase the amount of the effective steel, because there located in the tension zone of the deck.

10.7. CONCLUSION

In this chapter the steel deck is redesigned. Decisions made in previous chapters have let to adjustments in the design of the steel deck. These adjustments are incorporated and the structural properties of the deck are calculated.

In the deck design the compression zone is stiffened to increase the amount of effective steel. With Eurocode 3 part 3 and 5 is found that this design is very effective. Local buckling has little effect, because the compression

![Figure 10.6: Cross-section of the Comflor 95 (8, p.12)](image-url)
elements are relatively short and supported by stiffeners.

The loads during construction are calculated taking the non-uniform load distribution into account. The load-carrying capacity of the deck is verified and it is found that the deck design has sufficient resistance.

The width of the deck panel is determined. This choice is not only influenced by optimal structural performance, but also by the maximum weight and the building practice. Here is chosen for a deck width of 300 mm. A unity check shows that this deck has sufficient load-carrying capacity. However, the self-weight of a deck panel exceeds its limit. Therefore is recommended to choose a thinner steel thickness, $t = 1.0$ mm. This provides sufficient capacity and reduces the weight per deck panel.

The choice of a thinner steel sheet and wider ribs has influence on the design of the steel-concrete composite floor. The self-weight of the steel-concrete composite floor is calculated to determine if the vertical- or longitudinal shear capacity of the floor is critical. It is found that the floor can provide sufficient longitudinal shear resistance. However, the reduced thickness of the steel sheet has influence on the design shear resistance of the deck. It is found that a sheet thickness of $t = 1.0$ mm is insufficient, but that a sheet thickness of $t = 1.20$ mm provides sufficient shear resistance. This deck design meets all criteria that were given in chapter 3.

Finally, two design improvements are suggested. Firstly, to apply a larger corner radius in the top flange to increase the amount of effective steel in the compression zone. Secondly, is suggested apply two longitudinal stiffeners in the bottom flange to position the tension reinforcement during the installation.

The design of the steel deck is illustrated in figure 10.7. Note that this deck can be installed with a double cast (in-situ) or with a single cast (prefab).
Figure 10.7: Design of the steel deck

(a) Section through profile

(b) Side view
11

STEEL-CONCRETE COMPOSITE FLOOR

In this chapter attention is paid to the structural behaviour of the steel-concrete composite floor slab, and two different installation processes are described. In addition, the steel-concrete composite floor slab is verified during service life. The following design criteria are covered:

- continuous floor: the hogging moment region (ULS);
- local floor load: failure of the concrete top slab (ULS);
- floor vibrations (SLS);
- total- and additional floor deflection (SLS);
- cracking of the concrete top slab (SLS).

The verifications are carried out for the steel-concrete composite floor slab with the deck design shown in figure 10.7. The self-weight of this floor slab is 3.4 kN/m$^2$. The design load for SLS verifications is 7.4 kN/m$^2$ and for ULS verifications 10.1 kN/m$^2$.

Note that vertical- and longitudinal shear resistance of the floor slab are designed and verified in chapter 7 and 8.

11.1. LOAD-CARRYING BEHAVIOUR

Figure 11.1 shows two cross-sections over the length of the floor span. The first illustrates the deck with the concrete top flange. The second illustrates the fill element, the fully cast ribs near the supports, the reinforcement bar in the bottom of the ribs, and the concrete in the bottom of the ribs.

The resistance of the floor slab during service life is determined only considering load-carrying mechanism 1 (figure 11.1a). The tension reinforcement in the bottom of the ribs is not taken into account. For this reason, the load-carrying principle of the floor design with a fill element during service life is comparable to a concrete slab with the deck as downstand truss beam.

The truss beam is formed by the following structural elements of the floor:

- compression flange: concrete top slab and top of the deck → in compression;
- struts and ties: web of the deck → transfer vertical- and longitudinal shear forces;
- tension flange: bottom of the deck → in tension.

The fully cast concrete ribs near the supports have the following function.

- contribute to the shear resistance;
- resist hogging moment → for a double- or continuous floor span;
• anchor the reinforcement bar in the bottom of the ribs;
• guide the compression force of the concrete top flange to the support → for the fire design.

The concrete in the bottom of the ribs is only used to protect the reinforcement bar during fire.

In figure 11.1b a second load-carrying mechanism is illustrated. This load-carrying mechanism, where the concrete top flange forms a compression arch and the bottom reinforcement a tensile tie, is considered to calculate the resistance of the floor slab during fire. The actual resistance of the floor slab during service life should be obtained taking both load-carrying mechanisms into account. The difference in stiffness of both mechanisms determines which system mainly carries the loads.

Figure 11.1: Cross-sections of the floor and load-carrying mechanism
11.2. INSTALLATION

The design of the steel-concrete composite floor slab with a fill element could be fabricated and installed in two ways, both having their advantages and disadvantages.

- first and second cast at site (in-situ);
- first cast is prefabricated and second cast at site (prefabrication).

11.2.1. IN-SITU

The first cast is required to install the tension reinforcement in the bottom of the ribs. This concrete in the bottom part of the rib will not contribute to the bending stiffness of the deck during the construction of the second cast. Since, the concrete in the bottom part of the rib is under tension and there is no shear connection between the steel and the concrete. The fill elements are placed after the first cast. The top reinforcement is positioned and the slab is finished with a second cast. The advantage of this installation process is the efficient stacking of the decks and that the panels can be manhandled. A disadvantage are the two casting stages at site that increase the required construction time.

11.2.2. PREFABRICATION

In a prefabricated variant the ribs are precast and only one cast is required at site. The deck panels cannot be manhandled and the stacking height increases. Placing the deck panels at site requires more effort, but the floor is finished with one cast at site. Note that the shape of the deck panel is suitable to function as mold for the first cast.

11.3. HOGGING MOMENT

In this section the hogging moment resistance is verified for a continuous floor. The hogging moment is a negative bending moment that occurs at the supports of a continuous floor. Figure 11.2 illustrates the hogging moment for three different situations: single span, double span and a continuous span. The moments distribution is here given for a floor with a continuous bending stiffness. Note that the actual moment distribution depends on the ratio of the positive- and negative bending stiffness of the floor.

Eurocode 4 prescribes a maximum reduction of 30% to allow for a redistribution of forces. This redistribution of forces is caused by the cracking of the concrete in the hogging moment region and will result in a larger sagging moment. Note that the floor is already designed to resist the maximum sagging moment (design situation of a single span).

The design hogging moment:

\[ M_{Ed} = \frac{1}{8} q_d L^2 = \frac{1}{8} \cdot 10.1 \cdot 7.2^2 = 65.5 \text{ kNm/m width} = 19.7 \text{ kNm/rib} \]

\[ M_{Ed,red} = 0.70 \cdot M_{Ed} = 0.70 \cdot 19.7 = 13.8 \text{ kNm/rib (30% reduction due to redistribution of forces)} \]

In figure 11.3 (left) the cross-section at the hogging moment region is simplified to calculated the negative moment resistance \( M_{Rd} \). Given that:

- the ribs are fully casted at the support;
- standard reinforcement mesh ø8 – 200 mm: \( A_s = 72 \text{ mm}^2 \) per rib;
- effective width of cross-section: \( b = 130 \text{ mm} \) (minimum width of the concrete);
- effective depth of the cross-section: \( d = h - c_d - \sigma/2 = 280 - 25 - 8/2 = 251 \text{ mm} \).

The bending moment resistance is calculated assuming that the reinforcement is yielding \( \epsilon_s > \epsilon_{sy} \). The strains, stresses, and internal forces under this assumption are illustrated in figure 11.3.

\[ \epsilon_s > \epsilon_{sy} = \frac{\sigma_{sd}}{E_s} = \frac{f_{sd}}{E_s} = \frac{435}{200000} = 2.17\% \]
Figure 11.2: Different moment lines $M$ of a floor with continuous bending stiffness

Figure 11.3: Hogging moment resistance: stress-strain-diagram and internal forces
A first estimation of the required amount of steel is calculated under the following assumptions.

- \( N_s = A_s z < N_c \);
- \( z = 0.9 d = 0.9 \cdot 251 = 226 \text{ mm} \);
- \( M_{Rd} \geq M_{Ed} \).

The required amount of steel is:

\[
M_{Rd} = N_s z = A_s f_{sd} z \geq M_{Ed}
\]
\[
\Rightarrow A_s \geq \frac{M_{Ed}}{f_{sd} z} = \frac{13.8 \cdot 10^6}{435 \cdot 226} = 139.8 \text{ mm}^2/\text{rib}
\]

The reinforcement mesh provides already for 72 \text{ mm}^2/\text{rib}. Therefore 68 \text{ mm}^2/\text{rib} extra reinforcement is required to resist the maximum hogging moment. Here is chosen to apply \( \phi 10 \) per rib. The total amount of hogging moment reinforcement is \( A_s = 72 + 78 = 153 \text{ mm}^2/\text{rib} \).

The hogging moment resistance is verified. The height of the compression zone \( x_u \) follows from horizontal equilibrium.

\[
\Sigma F_H = 0 : \quad N_c = N_s
\]
\[
\Rightarrow x_u = \frac{A_s f_{sd}}{ab f_{cd}} = \frac{153 \cdot 435}{0.75 \cdot 130 \cdot 13.33} = 51.2 \text{ mm}
\]

The internal leverarm:

\[
z = d - \beta x_u = 251 - 0.39 \cdot 51.2 = 231 \text{ mm}
\]

The bending moment resistance:

\[
M_{Rd} = N_s z = A_s f_{sd} z = 153 \cdot 435 \cdot 231 \cdot 10^{-6} = 15.3 \text{ kNm/rib}
\]

Unity check:

\[
UC = \frac{M_{Ed,red}}{M_{Rd}} = \frac{13.8}{15.3} = 0.90 < 1
\]

The cross-section has sufficient capacity to resist the maximum hogging moment if \( \phi 10 \) is applied in each rib.

Verify the assumption that the steel is yielding:

\[
\varepsilon_s = \left(\frac{d - x_u}{x_u}\right) \cdot \varepsilon_{cu3} = \left(\frac{251 - 51.2}{51.2}\right) \cdot 3.5\% = 13.7\% > \varepsilon_{sy}
\]

The steel is yielding and the maximum steel stress is not exceeded: \( \varepsilon_s = 13.7\% < \varepsilon_{ud} = 0.9 \cdot \varepsilon_{uk} = 0.9 \cdot 25\% = 22.5\% \). The hogging moment region will fail due to crushing of the concrete after a safe warning, because there is sufficient deformation (yielding of the steel).

It is expected that the hogging moments are less than calculated above, because the bending stiffness of the hogging region is significantly smaller than the bending stiffness of the sagging region.

The bending stiffness of the hogging moment region:

\[
\kappa_{Rd} = \frac{\varepsilon_{cu3} + \varepsilon_s}{d} = \frac{3.5 + 13.7}{251} = 0.0685 \text{ m}^{-1}
\]
\[
EI_{Rd} = \frac{M_{Rd}}{\kappa_{Rd}} = \frac{15.3}{0.0685} = 223 \text{ kNm}^2/\text{rib}
\]

The bending stiffness of the sagging moment region is obtained with the second moment of area of the floor as calculated in chapter 8.

\[
EI_{Rd} = I_c \cdot E_s = (1342.2 \cdot 10^4 \cdot 2.1 \cdot 10^5) \cdot 10^{-9} = 2818 \text{ kNm}^2/\text{rib}
\]

Therefore, is assumed that 30% reduction is valid and hogging moment reinforcement of \( \phi 10 \) per rib is sufficient.
11.4. LOCAL LOAD

In this section the concrete top slab is verified to resist a high local load. In the design of concrete floors is this failure mechanism better known as punching shear. Here is:

- punching shear resistance verified with Eurocode 2 for slabs without shear reinforcement [6, p.97-105];
- slab reinforcement: standard mesh ø8 – 200 mm;
- design load: \( V_{Ed} = Q_d = \gamma_q Q_k = 1.5 \cdot 7 = 10.5 \text{ kN} \) on a surface of 100×100 mm.

The assumption for this verification is that the design load on the surface of the concrete top slab is comparable to a flat concrete floor slab supported on a column of 100×100 mm.

The punching shear resistance of a concrete slab supported by a column is obtained from the following expressions.

\[

\nu_{Rd,c} = C_{Rd,c} \left[ \frac{100 \rho f_{ck}}{d} \right]^{1/3} \geq \nu_{min} = 0.035 \frac{1}{f_{ck}^{1/2}} \text{ [N/mm}^2]\]

\[

V_{Rd,c} = V_{Rd,c}C_p \text{ [kN]}
\]

Where:

- \( k = 1 + \sqrt{\frac{d}{200}} \leq d \text{, with } d \text{ in mm;} \)
- \( \rho_l = \sqrt{\rho_{lz} \cdot \rho_{ly}} \leq 0.02; \)
- \( d \) is the effective depth;
- \( C_p \) is the control perimeter.

The effective depth is taken as:

\[

d = \frac{d_y + d_z}{2} = \frac{29 + 37}{2} = 33 \text{ mm}
\]

Where \( d_y \) and \( d_z \) are the effective depths of the reinforcement in two orthogonal directions. The control perimeter is within a distance of 2\( d \) of the area of the applied force. For an area of 100×100 mm is the perimeter:

\[

C_p = \pi (2d)^2 + 4 \cdot 100 = \pi (2 \cdot 33)^2 + 4 \cdot 100 = 14084 \text{ mm}
\]

\( \rho_{lz} \) and \( \rho_{ly} \) are calculated taking into account a slab width equal to the column width plus 3\( d \) at each column side.

\[

\rho_l = \frac{A_y}{A_c} = \frac{\pi d_y^2 \cdot 2.3d + 100}{200} \quad \text{and} \quad \frac{\pi d_z^2 \cdot 2.3d + 100}{200} = 74.9 \quad \text{and} \quad 9834 = 0.0076
\]

The code gives that \( C_{Rd,c} = 0.18/\gamma_c = 0.18/1.5 = 0.12 \). And the value for \( k \) is:

\[

k = 1 + \sqrt{\frac{33}{200}} = 1.41
\]

The following punching shear resistance is obtained:

\[

\nu_{Rd,c} = 0.12 \cdot 1.38 \left( 100 \cdot 0.0076 \cdot 20 \right)^{1/3} = 0.410 \geq \nu_{min} = 0.035 \cdot 1.38 \cdot 20^{1/2} = 0.174 \text{ N/mm}^2
\]

\[

V_{Rd,c} = 0.410 \cdot 14084 \cdot 10^{-3} = 5.9 \text{ kN}
\]

Unity check:

\[

\frac{V_{Ed}}{V_{Rd}} = \frac{10.5}{5.9} = 1.78 > 1
\]

The concrete top slab has insufficient capacity to resist the local load. However, the local design load of \( Q_d = 10.5 \text{ kN} \) is a very high demand.
This local load is obtained from the national annex of Eurocode 1 part 1 for actions on structures [41, p.4]. This code gives two characteristic values for concentrated imposed loads. The first is a load of 3 kN applicable for dwelling and office buildings. The second is a load of 7 kN applicable for correction rooms and shopping centres. In most cases would a design load \( Q_d = \gamma_d Q_k = 1.5 \cdot 3 = 4.5 \text{ kN} \) be sufficient. The concrete top slab has sufficient capacity to resist this load. If more capacity is required a double mesh can be applied to provide more punching shear resistance.

### 11.5. Deflections

In this section the criteria for the deflections are verified.

The total floor deflection \( w_{\text{tot}} \) is the sum of the deflection during construction \( \delta_0 \) and the additional deflection during service life \( w_{\text{addl}} \). The criteria for the total deflection is given in Eurocode 0 [42, p.13-14].

\[
 w_{\text{tot}} \leq 0.004L = 0.004 \cdot 7200 = 28.8 \text{ mm}
\]

This criteria should be met if aesthetics are important.

The additional floor deflection \( w_{\text{addl}} \) is the time dependent floor deflection of the steel-concrete composite slab. It is obtained from the time dependent deflection of the slab under the total load (permanent plus variable) minus the time independent deflection of the slab under the permanent load.

The criteria for the additional deflection are given in Eurocode 0 [42, p.13-14].

\[
 w_{\text{addl}} \leq 0.002L = 0.002 \cdot 7200 = 14.4 \text{ mm}
\]

This criteria should be met to prevent damage of the non-structural members and finishes. If for example partition walls are used that are sensitive for cracking.

First the bending stiffness of the steel-concrete composite floor calculated.

**Bending stiffness**

The time independent- and time dependent bending stiffness are determined with the method used in section 8.2.

**Assumptions:**

- only the concrete in compression is taken into account;
- fictitious steel cross-section with: \( n_0 = \frac{E_a}{E_{cm}} \approx 2.1 \cdot 10^5 \cdot \frac{30000}{50000} = 7; \)
- time dependent effects included with: \( n = \frac{E_a}{E_{cm/2}} = 2.1 \cdot 10^5 \cdot \frac{30000}{50000/2} = 14; \)
- the effective width \( b_{\text{eff}} \) is equal to the c.t.c. distance of the ribs: \( b_{\text{eff}} = 300 \text{ mm}. \)

**Properties of the deck:**
First the time independent bending stiffness is calculated with $n_0 = 7$. The position of the neutral axis is:

$$x_{el} = \frac{A_{pe}d_p + \frac{b_{eff}h_c}{n_0}}{A_{pe} + \frac{b_{eff}h_c}{n_0}} = \frac{527.7 \cdot 166.7 + \frac{300 \cdot 60}{7}}{527.7 + \frac{300 \cdot 60}{7}} = 51.1 \text{ mm}$$

The second moment of area is:

$$I_{c,0} = \frac{b_{eff}h_c^3}{12n_0} + \frac{b_{eff}h_c}{n_0} \left( x_{el} - \frac{h_c}{2} \right)^2 + I_{eff} + A_{pe} (d_p - x_{el})^2$$

$$= \frac{300 \cdot 60^3}{12 \cdot 7} + \frac{300 \cdot 60}{7} \left( 51.1 - \frac{60}{2} \right)^2 + 569.1 \cdot 10^4 + 527.7(166.7 - 51.1)^2$$

$$= 1487 \cdot 10^4 \text{ mm}^4/\text{rib} = 4959 \cdot 10^4 \text{ mm}^4/\text{m width}$$

The time independent bending stiffness is:

$$EI_0 = E \cdot I_{c,0} = 2.1 \cdot 10^5 \cdot 4959 \cdot 10^4 = 10.41 \cdot 10^{12} \text{ N/mm}^2$$

The same method is used to calculate the time dependent bending stiffness of the floor with. Factor $n$ is used to take account for the effects of creep and shrinkage.

$$n = \frac{E_a}{\frac{1}{2}(E_{cm} + \frac{5}{3}E_{cm})} = \frac{210000}{\frac{1}{2}(30000 + \frac{5}{3}30000)} = 10.5$$

The position of the neutral axis and the second moment of area are:

$$x_{el} = 59.1 \text{ mm}$$

$$I_c = 4671.3 \cdot 10^4 \text{ mm}^4/\text{m width}$$

Note that a part of the deck is under compression, because the neutral axis is located in the deck. The time dependent bending stiffness is:

$$EI = E \cdot I_c = 2.1 \cdot 10^5 \cdot 4671.3 \cdot 10^4 = 9.81 \cdot 10^{12} \text{ N/mm}^2$$

**Total deflections**

In chapter 10 is calculated that the deck deflection during construction is $\delta_0 = 28.1 \text{ mm}$. The total floor deflections $w_{tot}$ are therefore always larger than the criteria given in the Eurocode. This floor is therefore not suitable if there is a high requirement for aesthetics.

**Additional deflections**

The additional floor deflection is determined.

The time dependent floor deflection is:

$$w_2 = \frac{5}{384} \frac{(g_k + q_k)L^4}{EI} = \frac{5}{384} \frac{(3.4 + 4.0) \cdot 7200^4}{2.1 \cdot 10^5 \cdot 4671.3 \cdot 10^4} = 26.4 \text{ mm}$$

The time independent deflection under the permanent load is:

$$w_1 = \frac{5}{384} \frac{g_kL^4}{EI_0} = \frac{5}{384} \frac{3.4 \cdot 7200^4}{2.1 \cdot 10^5 \cdot 4959 \cdot 10^4} = 13.4 \text{ mm}$$
The additional deflection is:

\[ w_{\text{addl}} = w_2 - w_1 = 26.4 - 13.4 = 13.0 \text{ mm} < 14.4 \text{ mm} \]

The additional deflections meet the criteria: \( UC = \frac{13.0}{14.4} = 0.90 \). Here is a single span floor considered. In building practice it is customary to design continuous floors and the additional deflection are further limited. To illustrate this are the additional deflections for a double and continuous floor determined. The different deflections shapes and the corresponding mid-span deflection, for a single-, double- and continuous floor span, are given in figure 11.5

![Figure 11.5: Different deflection shapes of the floor with expressions for the mid-span deflection \( w_3 \) [9, p.26]](image)

For a double span:
The time dependent floor deflection is:

\[ w_2 = \frac{1}{192} \left( \frac{g_k + q_k}{EI} \right) L^4 = \frac{1}{192} \left( \frac{3.4 + 4.0}{2.1 \cdot 10^5 \cdot 4671.3 \cdot 10^4} \right) = 10.6 \text{ mm} \]

The time independent deflection under the permanent load is:

\[ w_1 = \frac{1}{192} \left( \frac{g_k L^4}{EI_0} \right) = \frac{1}{192} \left( \frac{3.4 \cdot 7200^4}{2.1 \cdot 10^5 \cdot 4959 \cdot 10^4} \right) = 4.6 \text{ mm} \]

The additional deflection is:

\[ w_{\text{addl}} = w_2 - w_1 = 10.6 - 4.6 = 6.0 \text{ mm} < 14.4 \text{ mm} \]

For a continuous span:
The time dependent floor deflection is:

\[ w_2 = \frac{1}{384} \left( \frac{g_k + q_k}{EI} \right) L^4 = \frac{1}{384} \left( \frac{3.4 + 4.0}{2.1 \cdot 10^5 \cdot 4671.3 \cdot 10^4} \right) = 5.3 \text{ mm} \]

The time independent deflection under the permanent load is:

\[ w_1 = \frac{1}{384} \left( \frac{g_k L^4}{EI_0} \right) = \frac{1}{384} \left( \frac{3.4 \cdot 7200^4}{2.1 \cdot 10^5 \cdot 4959 \cdot 10^4} \right) = 2.3 \text{ mm} \]

The additional deflection is:

\[ w_{\text{addl}} = w_2 - w_1 = 5.3 - 2.3 = 3.0 \text{ mm} < 14.4 \text{ mm} \]
11.6. **Vibrations**

The floor vibrations are verified with the eigenfrequencies of the floor.

Quasi-permanent load-combination:

\[ q_d = g_k + \Psi_2 q_k = 3.4 + 0.3 \cdot 4 = 4.6 \text{ kN/m}^2 < 5 \text{ kN/m}^2, \]

verification is required.

Where \( \Psi_2 = 0.3 \) found in table A.1.1 of Eurocode 0.

The modular ratio for short-term loading and the short-term bending stiffness of the composite slab are determined in the previous section.

Maximum deflection of the composite slab under a quasi-permanent load:

\[ \delta = \frac{5}{384} \frac{q_d L^4}{EI_{c,0}} = \frac{5}{384} \frac{4.6 \cdot 7200^4}{2.1 \cdot 10^5 \cdot 4959 \cdot 10^4} = 15.5 \text{ mm} \]

NEN 6702 gives an expression to determine the eigenfrequency:

\[ f_e = \sqrt{\frac{a}{\delta}} \]

Where:

- \( a \) is a value for the type of dynamic system: 0.315 m/s\(^2\) for a single span beam with uniform load;
- \( \delta \) is the maximum deflection of the composite slab under a quasi-permanent load in meter.

This gives that: \( f_e = \sqrt{0.315/0.01546} = 4.51 \text{ Hz} > 3 \text{ Hz} \), and therefore is the performance of the floor sufficient for office buildings. Note that a higher bending stiffness is required if the floor is applied in a gym or dancing hall, where people are dancing and jumping.

11.7. **Cracking**

According to Eurocode 4 only a minimum amount of anti-crack reinforcement is required if the steel-concrete composite slab is designed as simply supported [12, p.104 and 110]. The minimum amount of anti-crack reinforcement \( A_s \) is 0.4% of the cross-sectional area of the concrete above the ribs for unpropped constructions.

\[ A_s \geq 0.4\% \cdot bh_c = 0.004 \cdot 1000 \cdot 60 = 240 \text{ mm}^2/\text{m} \]

Applied is ø8-200 mm providing sufficient reinforcement: \( A_s = 251 \text{ mm}^2/\text{m} > 0.4\% \cdot bh_c \)

11.8. **Conclusion**

In this chapter the structural behaviour of the steel-concrete composite floor slab during service life is described and verified.

The floor slab has two load-carrying mechanism during service life. The first is comparable to a concrete slab with the deck as downstand truss beam. In the second, the concrete top flange forms a compression arch and the bottom reinforcement a tensile tie. The first mechanism is used to obtain the resistance of the floor during service life and the second to obtain the resistance of the floor during fire. The actual resistance of the floor slab during service life should be obtained taking both load-carrying mechanisms into account. The difference in stiffness of both mechanisms determines which system mainly carries the loads.

The design of the steel-concrete composite floor slab with a fill element could be installed in two ways: in-situ or prefabricated. The in-situ installation is more flexible, but requires a double cast at site. The design of the deck profile is suitable for both types of installation.

The following is verified.

- the hogging moment region of the floor;
• a concentrated load on the concrete top slab;
• floor vibrations;
• floor deflections;
• cracking of the concrete top slab.

The required amount of hogging moment reinforcement is determined for a continuous floor. It is found that each rib requires a ø10 reinforcing bar over the support to provide sufficient resistance.

The concrete top slab has insufficient capacity to resist a concentrated load of 7 kN (characteristic value). However, these high concentrated loads are only considered if the floor is designed for a corrugation room or shopping centre. The Eurocode prescribes a concentrated load of 3 kN for dwelling and office buildings. The concrete floor slab has sufficient capacity to resist this load. If more capacity is required a double mesh can be applied to provide more resistance.

The total deflections do not meet the criteria of the Eurocode, due to the relatively large deflections during construction (28.1 mm). However, this criteria is only important regarding the aesthetics of the building. A lower ceiling can be applied to make the structure more appealing for the users of the building. The Eurocode gives a criteria for the additional floor deflections to prevent damage of the non-structural members and finishes. This criteria is met for the most critical situation, a single span ($UC = 0.90$).

The floor vibrations are verified with the eigenfrequencies of the floor. The criteria for vibrations aim to prevent resonance of the floor that is experienced as unpleasant by the users of a building. Dwelling and office buildings require a eigenfrequency of at least 3 Hz and gyms or dancing halls at least 5 Hz. It is found that the performance of the floor is sufficient to limit the floor vibrations for office buildings. A higher bending stiffness is required if the floor is applied in a gym or dancing hall, where people are dancing and jumping.

The steel-concrete composite slab is designed as simply supported and therefore only a minimum amount of anti-crack reinforcement is required. The applied mesh of ø8-200 mm is sufficient to limit the cracks in the concrete top slab.
Part 4

What Have We Learned?
12.1. **CONCLUSIONS**

**LITERATURE REVIEW**

In part 1 findings from the literature review were used to write specifications for the new design. The specifications for the design were formulated based on the manufacturing process, the current advantages of the steel deck and steel-concrete composite floor slab, and design rules obtained form the Eurocode. Important aspects taken into account are the efficient installation of the decks, a low self-weight, and a 90 minutes fire resistance of the floor. This study is aimed at a steel thickness of 1.0 to 1.25 mm and a minimum amount of steel and construction height.

**SPECIFICATIONS FOR THE DESIGN**

Table 12.1 presents a brief overview of the most important specifications.

<table>
<thead>
<tr>
<th>Achieve:</th>
<th>Aim at:</th>
</tr>
</thead>
<tbody>
<tr>
<td>$h$ : $\leq$ 300 mm</td>
<td>‘minimum’ construction height</td>
</tr>
<tr>
<td>$L$ : $\geq$ 7.2 m</td>
<td>‘low’ self-weight</td>
</tr>
<tr>
<td>$G_{deck}$ : $\leq$ 50 kg/deck panel</td>
<td>‘large’ panel width</td>
</tr>
<tr>
<td>$t_i$ : $\geq$ 90 min</td>
<td>‘minimum’ amount of steel</td>
</tr>
<tr>
<td>UC : $\leq$ 1</td>
<td>thickness deck 1.0 up to 1.25 mm</td>
</tr>
</tbody>
</table>

Table 12.1: Most important specifications for the design

**PRELIMINARY ANALYSIS**

In a preliminary analysis was confirmed that the bending stiffness of the deck is governing during construction and not the bending strength. From comparison with current decking was found that the steel deck is able to span 7.2 m unpropped if the bending stiffness is increased with 60%.
DESIGN CONCEPTS

Different design concepts for steel-concrete composite floors with bigger spans were studied and their feasibility was assessed. It is concluded that concrete in the ribs of the steel-concrete composite floor is important to provide a minimum fire resistance of 30 minutes. For this reason, the shape of the deck should be trapezoidal with a reinforcement bar embedded in a concrete rib. An unpropped span of 7.2 m could be achieved by a deck that has more load-carrying capacity and/or a reduction of the self-weight of the floor.

STEEL-CONCRETE COMPOSITE FLOOR WITH RIBS

Two design concepts were studied: (1) a deck design with small ribs, and (2) a deck design with wide ribs and a lightweight fill element in these wide ribs. The first design concept results in a heavy floor slab and is not further developed. The second design results in a light floor slab. It was found that the weight reduction is very efficient if the fill elements are placed on top of a first cast and the slab is finished with a second cast. The first cast in the bottom of the ribs protects the reinforcement bar that provides fire resistance. The installation process of this design proved to be practical and a prefab variant is possible.

The second design concept structurally does not behave the same as current steel-concrete composite floors. There is no concrete in a large part of the ribs. Three critical aspects were studied to determine if this design is feasible: (1) the shear resistance of the steel deck, (2) the steel-concrete composite connection, and (3) the resistance to fire.

DESIGN OF THE DECK TO RESIST SHEAR

The tension field method of Basler was used to determine the post-buckling behavior of the thin-walled stiffened web of the deck. This method allows to determine the load-carrying capacity of the web using a truss model. In this model the compression struts are formed by transverse stiffeners and the tensile ties by the tension fields in the web. From calculations it was obtained that the transverse stiffeners and the tension fields provide sufficient resistance. Based on a buckling model the stiffeners were also designed to prevent local and overall buckling. It was found that a minimum height of 131 mm was necessary, a height of 150 mm was applied.

DESIGN OF THE SHEAR CONNECTOR

The steel-concrete shear connection is established with dowels on the top flange of the deck. In this study the connection between the steel and concrete was assumed to be rigid, meaning that no redistribution of forces is taken into account. Test results for the Comflor 210 deck provide a safe design value for the longitudinal shear resistance of a single dowel. The dowels also connect the individual deck panels during construction. In this study the minimum amount of shear connectors was calculated using the maximum bending moment of 63 kNm/m. It was found that 18 connectors per half-span are needed to make a partial shear connection. The position of the connectors over the floor span were designed to ensure that each connector is loaded equally, assuming a linear elastic longitudinal stress distribution. To also prevent the vertical separation between the steel and concrete a new type of shear connector was developed, which is visualized in figure 8.13.

DESIGN OF THE FLOOR SLAB DURING FIRE

Due to the weight reducing elements it was expected that the thin-walled deck will quickly heat up and lose strength during fire. In such a situation a new load path will develop. The loads in the floor are carried by a compression arch, formed within the concrete top flange, and a tensile tie, formed by the reinforcement bar in the bottom of the ribs. The floor elements are designed to initially provide a mechanical fire resistance of 60 and 90 minutes. It was found that respectively a reinforcement bar of ø16 mm and ø20 mm was needed.

The very slender concrete top flange shows a sensitivity to buckling. This phenomenon can be prevented using (1) a non-combustible fill element that provides support, or (2) a concrete top flange that is designed to resist the buckling. Both can be used to provide a fire resistance of 90 minutes. To establish an equilibrium of forces between the concrete top flange and the tension reinforcement bar different types of anchorage are suitable: (1) a bended or separate hook at the end of the bar, (2) a bar through holes in the steel beam, and (3) a separate hook over the support. The most suitable type depends on the end-support conditions.
12.1. Conclusions

Design of the Steel Deck

The load-carrying capacity of the deck was verified using Eurocode 3. It was found that the deck design has sufficient resistance against the increasing load due to the extra concrete near the end supports and the non-combustible fill elements. In the new design the load-carrying capacity of the steel deck is increased by the smaller web embossments, stiffeners in the top flange, and the longitudinal web stiffeners. The compression zone was stiffened to increase the effectiveness of the steel used in the deck, and the width of a single deck panel was determined. The choice of width was not only influenced by an optimal structural performance, but also by the maximum weight of a single panel and therefore the possible application during execution. A deck was chosen with a width of 300 mm and a steel thickness of 1.20 mm.

Design of the Steel-Concrete Composite Floor Slab

The mechanical functioning of the floor can be described using two mechanisms: (1) a concrete slab with the deck modelled as a downstand truss beam, and (2) a concrete top flange forming a compression arch and the bottom reinforcement bar as a tensile tie. The functioning during service life was described using the first mechanism, although a description with both is also possible. The difference in stiffness of both mechanisms will determine which system will dominate. The functioning during fire was described using the second mechanism.

The steel-concrete composite floor slab with fill elements can be installed in two ways: (1) in-situ or (2) prefabricated. The in-situ installation is more flexible, but requires a double cast at site. The design of the deck makes it suitable to apply both types of installation. The required amount of hogging moment reinforcement was determined on a ø10 reinforcing bar per rib. This is for a continuous floor. Note that in this study all designs and verifications were made for a simple span floor. The concrete top flange can resist a concentrated load of 3 kN, which is sufficient for dwelling and office buildings. If the function of the building requires more capacity a double mesh in the concrete top flange can provide more resistance.

Additionally the serviceability limit state criteria (SLS) were discussed. These criteria are not primary, and mostly have an esthetically value, rather than a safety value. The total deflection found - based on a linear analysis - did meet the criterion of the Eurocode, taking into account that the relatively large deflection during construction does not have to be taken into account. The Eurocode also gives criterion on the additional floor deflection, to prevent damage to non-structural members and finishing. This criterion was met for the most critical situation, the single span (UC = 0.9).

The floor vibrations were verified, determining the eigenfrequencies of the floor. It was found that the performance of the floor is sufficient for office buildings. A higher bending stiffness is required if the floor is applied in a gym or dancing hall, where people are dancing and jumping. The steel-concrete composite slab was designed as simple supported and therefore only a minimum amount of anti-crack reinforcement is required. The applied mesh of ø8-200 mm is sufficient to limit the cracks in the concrete top slab.

The JorFlor

The general conclusion of this thesis, and respectively the answer to the main research question is the introduction of a new design for the deck of a steel-concrete composite floor slab, here introduced as the ‘JorFlor’.

Figure 12.1 illustrates the design of this floor. The total height of the floor slab is 280 mm. The deck panels are 7.2 m long, 220 mm high, 1.20 mm thick, 300 mm wide and weigh 50 kg. The top flanges of the decks are connected with shear connectors that provide the steel-concrete composite action in the floor slab. A first cast in the ribs is required to encase the bottom reinforcement. After that, fill elements are positioned in the ribs and the top reinforcement is installed. Different types of insulation materials are suitable as fill element: PIR foam, rock wool, resol foam or foam glass. The floor is finished with a second cast. A prefab variant of the JorFlor is also possible, where the concrete in the bottom of the ribs is prefabricated. The principle cross-sections of the floor are illustrated in figure 12.2. In these cross-sections the different elements of the floor are indicated. In addition, the properties of the JorFlor are given in table 12.2.

The JorFlor meets the design specifications as formulated in the first part of the research. It can therefore be said that the design is successful. Based on a theoretical study it can be concluded that it is possible to
design a steel deck for a steel-concrete composite floor slab, which can span 7.2 meter and can be constructed without the need of temporary supports. The JorFlor knows two practical applications. First of all, the ‘in-situ’ JorFlor can be applied in special circumstances where a big unpropped span is required, but prefabricated elements cannot be installed. Second of all, the ‘prefab’ JorFlor suits the current Dutch building practice. The installations process is simple and the construction speed is high. The installation of the decks requires light hoisting equipment at site.

12.2. RECOMMENDATIONS

SHEAR RESISTANCE OF THE WEB

Looking at the web of the steel decks it was found that the transverse stiffeners and the tensions fields provide sufficient shear capacity. Further studies have to indicate whether the web will react according to the truss model and the theory of Basler. Tests have to indicate if and how the actual tension field is formed. The formation of the tension field is influenced by the position and stiffness of the transverse stiffeners, and the shear buckling capacity of the web. The tension field on the other hand has an influence on the behavior of the embossment (transverse stiffeners). The tension in the web causes the embossment to elongate and weaken. It is recommended to make an approximation of the behavior of the web with stiffeners using FEM modeling, substantiated by tests.
12.2. RECOMMENDATIONS

(a) Cross-section of the span of the floor: deck, shear clips and concrete top slab

(b) Cross-section of the span of the floor: fill element and reinforcement

(c) Cross-section transverse to the span of the floor

Figure 12.2: Principle cross-sections of the JorFlor

Table 12.2: Properties of the JorFlor compared with existing deep decks

<table>
<thead>
<tr>
<th>Deck properties&lt;sup&gt;1&lt;/sup&gt;</th>
<th>JorFlor</th>
<th>CF210</th>
<th>CF225</th>
</tr>
</thead>
<tbody>
<tr>
<td>$b_d$ (mm)</td>
<td>300</td>
<td>600</td>
<td>600</td>
</tr>
<tr>
<td>$h_p$ (mm)</td>
<td>226</td>
<td>210</td>
<td>225</td>
</tr>
<tr>
<td>$t$ (mm)</td>
<td>1.20</td>
<td>1.25</td>
<td>1.25</td>
</tr>
<tr>
<td>$A_p$ (mm$^2$/m)</td>
<td>2970</td>
<td>2017</td>
<td>2278</td>
</tr>
<tr>
<td>$A_{pe}$ (mm$^2$/m)</td>
<td>1759</td>
<td>1426</td>
<td>1717</td>
</tr>
<tr>
<td>$I_{eff}$ (cm$^4$/m)</td>
<td>1896</td>
<td>816</td>
<td>1090</td>
</tr>
<tr>
<td>$M_{el,Rd}$ (kNm/m)</td>
<td>55.5</td>
<td>23.1</td>
<td>30.8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Floor properties</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$h$ (mm)</td>
<td>280</td>
<td>290</td>
<td>305</td>
</tr>
<tr>
<td>$L$ (m)</td>
<td>7.20</td>
<td>5.30</td>
<td>5.45</td>
</tr>
<tr>
<td>$G&lt;sup&gt;2&lt;/sup&gt;$ (kg/m$^2$)</td>
<td>301</td>
<td>303</td>
<td>366</td>
</tr>
<tr>
<td>$V_c$ (l/m$^2$)</td>
<td>107</td>
<td>121</td>
<td>145</td>
</tr>
<tr>
<td>$G_p$ (kN/m$^2$)</td>
<td>0.228</td>
<td>0.157</td>
<td>0.171</td>
</tr>
<tr>
<td>$G_{deck}$ (kg/deck)</td>
<td>50</td>
<td>70</td>
<td>79</td>
</tr>
<tr>
<td>$G_{ponding}^3$ (kg/m$^2$)</td>
<td>41</td>
<td>34</td>
<td>32</td>
</tr>
<tr>
<td>$t_i$ (min)</td>
<td>90</td>
<td>90</td>
<td>90</td>
</tr>
</tbody>
</table>

<sup>1</sup> Dimensions of deck shown in figure 10.7

<sup>2</sup> Ponding not included in the self-weight

<sup>3</sup> Ponding given for the maximum unpropped span
Buckling of the transverse stiffeners is affected by the longitudinal stiffeners in the web, and by the top flange and bottom flange. For the buckling model conservative assumptions were used to include both these elements. It is expected that the effect of these elements is underestimated in the design. It is therefore recommended that their influence is further tested, taking into account the influence of the concrete.

**Design of the shear connector**

The shear connections are designed on a variable load of 4 kN/m², realising a partial shear connection. Two designs of shear connectors to prevent vertical separation of the deck and the concrete are created. At this moment in time no calculation rules are available to determine the longitudinal behavior of these connectors. A FEM model to analyze this behavior is not recommended, because the actual interaction between the steel and the concrete is very difficult to model. It is recommended to perform a ‘push tests’; in accordance with Eurocode 4, to determine the longitudinal shear resistance and the so called load-slip behaviour per connector. There are four existing types of ‘push test’ to determine the longitudinal shear bond characteristics of profiled sheeting: Daniel’s, Patrick’s, Porter’s, and Stark’s push test. Literature shows that Stark’s push tests is most realistic for this situation.

**Design of the floor slab during fire**

The load transfer in the floor under fire circumstances is modelled using a compression arch and a tension tie. In the estimation of the fire resistance it is assumed that the deck looses its structural functioning completely, leaving a concrete top flange under pressure. The buckling of the concrete top flange during fire is prevented in two ways: (1) support provided by a non-combustible fill element or (2) resistance created by a concrete top flange. Both solutions provide a fire resistance of 90 minutes. The concrete top flange had to increase self-weight significantly in order to resist the buckling. To reduce the weight and create a certain support for the concrete top flange, a non-combustible material ‘rock wool’ can be used as fill elements. It is recommended to make a cost analysis and a detailed fire calculation to visualize the feasibility of both solutions.

In conclusion, the floor slab is designed for a mechanical fire resistance of 90 minutes. However, fire tests are always obligatory to verify the calculations. These test should also show the actual influence of the insulation and the integrity during fire.

**Economy**

The economic aspects of the floor design were not in the scope of this thesis. This thesis was aimed at finding a structurally effective design. A design that is structurally effective is often economically attractive. The aspects that determine the economic footprint of the floor are material costs, the production process, transport, installation, construction time, and material. It is therefore complex to draw a simple conclusion. Nevertheless, two aspects from an economic point of perspective will be discussed in the underlying text.

The amount of steel in the deck is much higher than compared to current composite floors. This will lead to an increase in costs per square meter floor area. It is recommended to further study whether the increase of the unpropped span can weigh up to these increases in costs. Additionally, the installation process of the JorFlor is more demanding. The in-situ double cast will lead to an increase in construction time. The prefab variant will lead to an increase in transportable volume. Therefore, it is recommended to conduct a cost analysis, whether the JorFlor is cheaper than other competitive floor systems.


[38] NEN-EN13501-1+A1, *Fire classification of construction products and building elements - Part 1: Classification using data from reaction to fire tests* (CEN, 2007).


PRELIMINARY ANALYSIS: TABLES AND EXPRESSIONS

A.1. LOADS DURING CONSTRUCTION

<table>
<thead>
<tr>
<th>Area</th>
<th>Value</th>
<th>Unit</th>
<th>Includes</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.75</td>
<td>kN/m²</td>
<td>$Q_{ca}^{(1)}$</td>
<td>Outside working area</td>
</tr>
<tr>
<td>2</td>
<td>$0.75 \leq 10%$ self-weight of concrete $\leq 1.5$ kN/m²</td>
<td>$Q_{ca}$ and $Q_{cf}^{(2)}$</td>
<td>Working area (3x3 m)</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>$0.5^{(4)} +$ self-weight of fresh concrete kN/m²</td>
<td>$Q_{cc}$ and $Q_{cf}$</td>
<td>Actual area</td>
<td></td>
</tr>
</tbody>
</table>

1) Working personnel with small equipment
2) Loads from part of structure in temporary state
3) Formwork and load bearing members
4) Value is true if the self-weight of the steel structure ($G_p$) is smaller than 0.5, otherwise $G_p$

Key for the different areas is illustrated in figure A.1

Table A.1: Construction loads (characteristic values) [17]

Figure A.1: Load during construction (steel deck as shuttering; key in table A.1)
### A.2. Material Properties

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Value</th>
<th>Unit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \rho_s )</td>
<td>7850</td>
<td>kg/m(^3)</td>
<td>Steel density</td>
</tr>
<tr>
<td>( \rho_c )</td>
<td>2400(^{1,2})</td>
<td>kg/m(^3)</td>
<td>Density of normal weight concrete (NWC)</td>
</tr>
<tr>
<td>( f_{ck} )</td>
<td>20</td>
<td>N/mm(^2)</td>
<td>Characteristic 5% cylinder strength of C20/25 NWC</td>
</tr>
<tr>
<td>( E_s )</td>
<td>(2.1 \cdot 10^5)</td>
<td>N/mm(^2)</td>
<td>Modulus of elasticity of steel</td>
</tr>
<tr>
<td>( E_{cm} )</td>
<td>(30 \cdot 10^3)</td>
<td>N/mm(^2)</td>
<td>Mean modulus of elasticity of C20/25 NWC</td>
</tr>
</tbody>
</table>

\(^{1}\) Increase by 1 kN/m\(^3\) for normal percentage of reinforcing steel

\(^{2}\) Increase by 1 kN/m\(^3\) for unhardened concrete

Table A.2: Material properties [6, 18]

### A.3. Euler-Bernoulli Beam

The Euler-bernoulli bending beam theory assumes the following:

- shear strains are approximately zero; only bending;
- theory is only valid under the assumptions of small displacements;
- beam has a straight longitudinal axis, any shape of cross-section, and is symmetrical around the y-axis;
- only in-plane deformations are allowed (xy-plane).

These assumptions are valid, because the design specifications only allow for small deflections where only in-plane deformations are considered. Figure A.2 illustrates the axes and sign convention to solve this problem.

![Figure A.2: Sign convention of a bending beam](10, p.10)
The differential equations (DE) that describe the Euler-bernoulli bending beam [10, p.14]:

\[ E I \frac{d^4 w}{dx^4} = q \]
\[ \frac{d^2 w}{dx^2} = -\frac{M}{EI} \]

Where:
- \( E \) is the modulus of elasticity of the beam in N/mm\(^2\);
- \( I \) is the second moment of area of the beam in mm\(^4\);
- \( q \) is a uniform load on the beam in kN/m;
- \( M \) is the internal moment in the beam in Nmm.

Integrating the DE gives the expression for the deflection \( w(x) \).

\[ \frac{d^4 w}{dx^4} = q \]
\[ \frac{d^3 w}{dx^3} = qx + C_1 \]
\[ \frac{d^2 w}{dx^2} = \frac{1}{2} qx^2 + C_1 x + C_2 \]
\[ \frac{d^1 w}{dx^1} = \frac{1}{6} qx^3 + \frac{1}{2} C_1 x^2 + C_2 x + C_3 \]
\[ EIw = \frac{1}{24} qx^4 + \frac{1}{6} C_1 x^3 + \frac{1}{2} C_2 x^2 + C_3 x + C_4 \]

Here are \( C_1 \) to \( C_4 \) the integration constants. The integration constants can be solved with four boundary conditions. It is assumed that the deck, here approached as a beam, is simply supported. The four boundary conditions are: \( w(0)=0 \), \( w(L)=0 \), \( M(0)=0 \) and \( M(L)=0 \).

\[ w(0) = 0 \Rightarrow C_4 = 0 \]
\[ M(0) = 0 \Rightarrow C_2 = 0 \]
\[ M(L) = 0 \Rightarrow C_1 = -\frac{1}{2} qL \]
\[ w(L) = 0 \Rightarrow C_3 = \frac{1}{24} qL^3 \]

Solving this results in the expressions for the deflection \( w(x) \) and bending moment \( M(x) \), both depending on \( x \).

\[ w(x) = \frac{1}{EI} \left( \frac{1}{24} qx^4 - \frac{1}{12} qLx^3 + \frac{1}{24} qL^3 x \right) \]
\[ w_{max} = w\left(\frac{L}{2}\right) = \frac{5}{384} \frac{qL^4}{EI} \]
\[ M(x) = \frac{1}{2} \left( Lx - x^2 \right) \]
\[ M_{max} = M\left(\frac{L}{2}\right) = \frac{1}{8} qL^2 \]  

(A.1)

The maximum deflection \( w_{max} \) and the maximum bending moment \( M_{max} \) are found at half the span for a simply supported beam.

**A.4. Effect of ponding**

During construction the deck carries its self-weight, the fresh poured concrete and the construction loads (table A.1). However, these loads do not incorporate the extra weight of the wet concrete that accumulates due to the deflection of the deck.
Eurocode 4 prescribes that the ponding effect should be taken into account if the maximum deflection \( w_{\text{max}} \) is bigger than 10% of the total floor height. The ponding effect is taken into account by increasing the nominal concrete thickness by \( 0.7 \cdot w_{\text{max}} \) [12, p.98].

Here a more accurate approach is given and verified with the rule from Eurocode 4. The volume for concrete accumulation is determined by integrating the deflection \( w(x) \) as found in expression A.1 over the span length \( L \).

\[
\text{Area} = \int_{0}^{L} w(x) \, dx = \frac{1}{EI} \left[ \frac{1}{120} q x^5 - \frac{1}{48} q L x^4 + \frac{1}{48} q L^3 x^2 \right]_{0}^{L} = \frac{1}{120} \frac{q L^5}{E I} \text{ (mm}^2) \quad (A.2)
\]

This is translated into a load increment. The area is equally divided over the span length to simplify the calculation. The ponding load is equal to:

\[
h_{\text{ponding}} = \frac{\text{Area}}{L} = \frac{1}{120} \cdot \frac{q L^4}{E I} \text{ (mm)}
\]

\[
q_{\text{ponding}} = \left( \frac{h_{\text{ponding}}}{1000} \cdot \rho_c \cdot g \right) / 1000 \text{ (kN/m}^2) \quad (A.3)
\]

To verify this approach an example calculation is made. The load \( q \) is 4.19 kN/m\(^2\), the span \( L \) is 7200 mm, the deck height is \( h_p \), 225 mm and the concrete top layer \( h_c \) is 60 mm. There is designed for a maximum deck deflection of 40 mm. According to Eurocode 4 should the ponding effect been taken into account, because 40 mm is bigger 10% of the total floor slab.

The two methods given above are compared by calculating the minimum second moment of area of the deck using expression 4.2.

\[
I \geq \frac{75}{32} q \cdot L^3 \cdot E = \frac{75}{32} 4.19 \cdot 7200^3 \cdot 2.1 \cdot 10^5 = 1746 \cdot 10^4 \text{ mm}^4/m
\]

\[
\text{Area} = \frac{1}{120} \cdot \frac{4.19 \cdot 7200^5}{2.1 \cdot 10^5 \cdot 1746 \cdot 10^4} = 184260 \text{ mm}^2/m
\]

\[
h_{\text{ponding}} = \frac{26}{1000} \cdot 2400 \cdot 9.81 = 60 \text{ mm}
\]

\[
q_{\text{new}} = q + q_{\text{ponding}} = 4.19 + 0.60 = 4.79 \text{ kN/m}^2
\]

\[
L_1 \geq \frac{75}{32} 4.79 \cdot 7200^3 \cdot 2.1 \cdot 10^5 = 1996 \cdot 10^4 \text{ mm}^4/m
\]

Where \( I \) is the initial required second moment of area where the effect of ponding is not included and \( I_1 \) is the recalculated required second moment of area including the effect of ponding.

For a comparison the rule given in Eurocode 4 rule is applied:

\[
h_{\text{ponding}} = 0.7 \cdot 40 = 28 \text{ mm}
\]

\[
q_{\text{ponding}} = \left( \frac{28}{1000} \cdot 2400 \cdot 9.81 \right) / 1000 = 0.66 \text{ kN/m}^2
\]

\[
q_{\text{new}} = q + q_{\text{ponding}} = 4.19 + 0.66 = 4.85 \text{ kN/m}^2
\]

\[
L_1 \geq \frac{75}{32} 4.85 \cdot 7200^3 \cdot 2.1 \cdot 10^5 = 2021 \cdot 10^4 \text{ mm}^4/m
\]

The difference between the two methods is relatively small. Here is assumed that the method given in expressions A.2 and A.3 is more accurate. Note that multiple iterations are required to provide a safe estimation of the ponding effect.
**B.1. Verification of the Deck and the Composite Slab**

**B.1.1. Introduction**

In this example the calculation procedure is described to verify the strength and serviceability of a steel-concrete composite floor design. This calculation example illustrates the expressions and design rules given in the Eurocode [6, 12, 18, 28, 31].

The verification is performed for both the construction stage (deck only) and the service life (steel-concrete composite slab). The floor is designed as single span. The composite floor slab, that might be continues over multiple spans, is designed as a series of simply supported spans. This approach is conservative, because the positive effect of the continues floor slab over the supports is ignored.
**Dimensions and Material Properties**

**Floor slab**
- Total depth of slab: \( h = 270 \text{ mm} \)
- Weight of the slab: \( G = 348 \text{ kg/m}^2 \)

**Deck**
- Width of deck panel: \( b_d = 260 \text{ mm} \)
- Width top flange: \( b_0 = 210 \text{ mm} \)
- Width bottom flange: \( b_u = 50 \text{ mm} \)
- Thickness of the deck: \( t = 1.25 \text{ mm} \)
- Design thickness of the deck: \( t_0 = 1.21 \text{ mm} \)
- Deck height: \( h_p = 210 \text{ mm} \)
- Cross-section area of the deck: \( A_p = 3397 \text{ mm}^2/\text{m} \)
- Effective cross-section area of the deck: \( A_{pe} = 2141 \text{ mm}^2/\text{m} \)
- Second moment of area of the deck: \( I_{eff} = 1786 \text{ cm}^4/\text{m} \)
- Elastic bending resistance (sagging): \( M_{el,Rd} = 48.03 \text{ kNm/m} \)
- Plastic bending resistance (sagging): \( M_{pl,Rd} = 52.48 \text{ kNm/m} \)
- Height elastic neutral axis (e.n.a.) from bottom: \( e = 107.7 \text{ mm} \)
- Height plastic neutral axis (p.n.a.) from bottom: \( e_p = 109.6 \text{ mm} \)

**Concrete**
- Normal weight concrete (NWC) C20/25
- Density (NWC, reinforced) \( \rho_{c,wet} = 26 \text{ kN/m}^3 \)
- Density (NWC) \( \rho_{c,dry} = 25 \text{ kN/m}^3 \)
- Cylinder strength \( f_{ck} = 20 \text{ Mpa} \)
- Mean value modulus of elasticity (short-term) \( E_{cm} = 30000 \text{ Mpa} \)

*Values may vary for a specific project depending on the amount of steel reinforcement.*

**Composite slab**
- Empirical factor for design shear resistance \( m = 214.69 \text{ N/mm}^2 \)
- Empirical factor for design shear resistance \( k = 0.01 \text{ N/mm}^2 \)
- Design value of longitudinal shear strength \( \tau_{u,Rd} = 0.08 \text{ N/mm}^2 \)

*Values of the Comflor 210 [8]*
- Distance centroid deck (e.n.a.) to top of slab \( d_p = 162.3 \text{ mm} \)

**Steel**
- Yield strength of the deck \( f_{yp} = 350 \text{ Mpa} \)
- Yield strength of the reinforcing steel \( f_{sk} = 500 \text{ Mpa} \)
- Modulus of elasticity \( E_s = 210000 \text{ Mpa} \)

*NEN-EN 1993-1-3 Table 3.1a and b*
- Density structural steel \( \rho_s = 7850 \text{ kg/m}^3 \)

**Partial factors for resistance**
- Structural steel \( \gamma_{M0} = 1.0 \)
- Concrete \( \gamma_c = 1.5 \)
- Reinforcement \( \gamma_s = 1.15 \)
- Longitudinal shear \( \gamma_{VS} = 1.25 \)

*NEN-EN 1993-1-3 p.12, NEN-EN 1992-1-1 NA Table 2.1 and NEN-EN 1994-1-1 p.24*

**Design values of material strength**
- Design yield strength of the deck \( f_{yp,d} = \frac{f_{yp}}{\gamma_{M0}} = 350 \text{ Mpa} \)
- Design value of the concrete compressive strength \( f_{cd} = \frac{f_{ck}}{\gamma_c} = 13.33 \text{ Mpa} \)
- Design yield strength of the reinforcement \( f_{sd} = \frac{f_{sk}}{\gamma_s} = 435 \text{ Mpa} \)
B.1.2. THE CONSTRUCTION STAGE

**Actions**

**Permanent actions**

A concrete layer $h_c$ of 60 mm is assumed. The area and weight of the wet concrete is then:

$$A_c = (b_u \cdot h_p + b_d \cdot h_c) \cdot \frac{1000}{b_d} = (50 \cdot 210 + 260 \cdot 60) \cdot \frac{1000}{260} = 100385 \text{ mm}^2 / \text{m}$$

$$G_c = \frac{A_c}{1000^2} \cdot \rho_{c,wet} = \frac{100385}{1000^2} \cdot 26 = 2.610 \text{ kN/m}^2$$

Weight of the steel deck is:

$$G_p = \frac{A_p}{1000^2} \cdot \rho_a \cdot g/1000 = \frac{3397}{1000^2} \cdot 7800 \cdot 9.81/1000 = 0.260 \text{ kN/m}^2$$

**Variable actions**

The construction loads are defined according to the NEN-EN 1991-1-6 and an overview is given in table A.1. Outside the working area acts a load of 0.75 kN/m$^2$. Inside the working area acts a load equal to 10% of the self-weight of the wet concrete limited by an under and upper value of 0.75 and 1.5 kN/m$^2$. This gives:

$$0.75 \leq 10\% \cdot G_c \leq 1.5 \Rightarrow 0.1 \cdot 2.610 = 0.261 < 0.75 \Rightarrow 0.75 \text{ kN/m}^2$$

**Serviceability limit state (SLS)**

**Partial factors for actions**

| Partial factor for permanent actions | $\gamma_g$ | 1.0 | - |
| Partial factor for variable actions | $\gamma_q$ | 1.0 | - |

**Permanent actions**

Weight steel deck

$$g_p = \frac{A_p}{1000^2} \cdot \rho_a \cdot g/1000 = \frac{3397}{1000^2} \cdot 7800 \cdot 9.81/1000 = 0.260 \text{ kN/m}^2$$

**Variable actions**

Weight concrete

$$g_c = 2.61 \text{ kN/m}^2$$

Total

$$g_k = 2.87 \text{ kN/m}^2$$

**Combination of actions at SLS**

$$q_d = \gamma_g g_k + \gamma_q q_k = 1.0 \cdot 2.87 + 1.0 \cdot 0 = 2.87 \text{ kN/m}^2$$

**Ponding effect**

The total load and deflection during construction are determined with an iterative calculation that includes the accumulated weight of the concrete. The following expressions, derived in section A.4, are used (illustrated in figure B.1):

$$\delta_{s,i} = \frac{5}{384} \frac{q_i L^4}{E I_{ef}} \text{ (mm)}$$

$$a_i = \frac{1}{120} \frac{q_i L^5}{E I_{ef}} - a_{i-1} \text{ (mm$^2$)}$$

$$h_{p,i} = \frac{a_i}{L} \text{ (mm)}$$

$$q_{p,i} = \left( \frac{h_{p,i}}{1000} \cdot \rho_c \cdot g \right) / 1000 \text{ (kN/m$^2$)}$$

$$q_{i+1} = q_i + q_{p,i} \text{ (kN/m)}$$

With $i = 0, 1, 2, ..$ and where:

- $a_i$ is the ponding area;
$h_{p,i}$ is the average height of the accumulated concrete over span $L$;

$q_{p,i}$ is the load increment due to ponding;

$g$ is the gravitational acceleration: $9.81 \text{ m/s}^2$.

![Figure B.1: Iterative process to determine the ponding effect](image)

**Step 0:**

\[
q_0 = 2.87 \text{ kN/m} \\
\delta_{s,0} = \frac{5}{384} \frac{2.87 \cdot 7200^4}{210000 \cdot 1786 \cdot 10^4} = 26.8 \text{ mm} \\
a_0 = \frac{1}{120} \frac{2.87 \cdot 7200^2}{210000 \cdot 1786 \cdot 10^4} = 123385 \text{ mm}^2 \\
h_{p,0} = \frac{7200}{123385} = 17.14 \text{ mm} \\
q_{p,0} = \left( \frac{17.14}{1000} \cdot 2400 \cdot 9.81 \right) / 1000 = 0.403 \text{ kN/m}^2
\]

**Step 1:**

\[
q_1 = 2.87 + 0.403 = 3.27 \text{ kN/m} \\
\delta_{s,1} = \frac{5}{384} \frac{3.27 \cdot 7200^4}{210000 \cdot 1786 \cdot 10^4} = 30.5 \text{ mm} \\
a_1 = \frac{1}{120} \frac{3.27 \cdot 7200^2}{210000 \cdot 1786 \cdot 10^4} = 17197 \text{ mm}^2 \\
h_{p,1} = \frac{7200}{17197} = 2.39 \text{ mm} \\
q_{p,1} = \left( \frac{2.39}{1000} \cdot 2400 \cdot 9.81 \right) / 1000 = 0.056 \text{ kN/m}^2
\]
Step 2:

\[
q_2 = 3.33 \text{ kN/m} \\
\delta_{s,2} = 31.1 \text{ mm} \\
\sigma_2 = 2579 \text{ mm}^2 \\
h_{p,2} = 0.36 \text{ mm} \\
q_{p,2} = 0.008 \text{ kN/m}^2
\]

Step 3:

\[
q_3 = 3.34 \text{ kN/m} \\
\delta_{s,3} = 31.2 \text{ mm}
\]

Here are 3 iterations supposed to be sufficient, because the difference \(\Delta q_p = q_{p,i} - q_{p,i-1}\) is very small and a next iteration step will not result in a significant load increment.

**Deflection with ponding**

The maximum deflection of the deck during the construction stage is: \(\delta_{s,max} = L/180 = 7200/180 = 40 \text{ mm}\). The occurring deflections including the ponding effect should not exceed this maximum the deflection.

\[
UC = \frac{\delta_s}{\delta_{s,max}} = \frac{31.2}{40} = 0.78 < 1.0
\]

The deck provides sufficient bending stiffness to limit the deflections.

**Ultimate limit state (ULS)**

**Concrete weight**

The additional weight of the extra concrete and the construction loads should be taken into account:

\[
g_{c,wet} = q_3 - g_p = 3.34 - 0.26 = 3.08 \text{ kN/m}^2
\]

**Partial factors for actions**

| Partial factor for permanent actions | \(\gamma_g\) | 1.2 | - |
| Partial factor for variable actions | \(\gamma_q\) | 1.5 | - |

**Permanent actions**

| Weight steel deck | \(g_p\) | 0.26 \text{ kN/m}^2 |
| Weight concrete | \(g_c\) | 3.08 \text{ kN/m}^2 |
| Total | \(g_k\) | 3.34 \text{ kN/m}^2 |

**Variable actions**

| Construction load | \(g_l\) | 0.75 \text{ kN/m}^2 |
| Total | \(q_k\) | 0.75 \text{ kN/m}^2 |

**Combination of actions at ULS**

\[
q_d = \gamma_g g_k + \gamma_q q_k = 1.2 \cdot 3.34 + 1.5 \cdot 0.75 = 5.2 \text{ kN/m}^2
\]

**Design moment and shear force**

The maximum design moment and shear force per meter width of deck are:

\[
M_{Ed} = \frac{1}{8} q_d L^2 = \frac{1}{8} \cdot 5.2 \cdot 7.2^2 = 33.7 \text{ kNm/m} \\
V_{Ed} = \frac{1}{2} q_d L = \frac{1}{2} \cdot 5.2 \cdot 7.2 = 17.7 \text{ kNm/m}
\]
**Bending resistance**

The following criteria should be met:

\[
UC = \frac{M_{Ed}}{M_{el,Rd}} = \frac{33.7}{48.0} = 0.70 < 1
\]

Where \( M_{el,Rd} \) is the elastic moment resistance of the deck. The elastic moment resistance of the deck during the construction stage is therefore sufficient.

**Shear resistance**

The shear resistance according to NEN-EN 1993-1-3 6.1.5. is:

\[
V_{b,Rd} = \frac{h_w \sin \varphi \cdot t \cdot f_{bv}}{\gamma_M 0} \ [N/web]
\]

Where:

- \( f_{bv} \) is the shear buckling strength determined with \( \tilde{\lambda}_w \) see figure B.2;
- \( h_w \) is the web height between the flanges: note that only the effective web height is considered;
- \( \varphi \) is the slope of the web relative to the flanges.

For webs without web stiffening at the support:

\[
\tilde{\lambda}_w = 0.346 \cdot \frac{s_w}{t} \sqrt{\frac{f_{yb}}{E}}
\]

\[
= 0.346 \cdot \frac{210}{1.21} \sqrt{\frac{550}{210000}} = 3.07
\]

\[
f_{bv} = 0.67 \cdot \frac{f_{yb}}{\tilde{\lambda}_w^2}
\]

\[
= 0.67 \cdot \frac{550}{3.07^2} = 39.02 \text{ Mpa}
\]

\[
V_{b,Rd} = \frac{2 \cdot 60}{\sin(90)} \cdot 1.21 \cdot 39.02 \cdot 1.0
\]

\[
= 5665 \cdot (260/2) = 43.6 \text{ kN/m}
\]

The following criteria should be met:

\[
UC = \frac{V_{Ed}}{V_{b,Rd}} = \frac{17.7}{43.6} = 0.41 < 1
\]

The shear resistance of the deck during the construction stage is therefore sufficient.

<table>
<thead>
<tr>
<th>Relative web slenderness</th>
<th>Web without stiffening at the support</th>
<th>Web with stiffening at the support</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>( \tilde{\lambda}_w \leq 0.83 )</td>
<td>( 0.58 f_{ys} )</td>
<td>( 0.58 f_{ys} )</td>
<td></td>
</tr>
<tr>
<td>( 0.83 &lt; \tilde{\lambda}_w &lt; 1.40 )</td>
<td>( 0.48 f_{ys} / \tilde{\lambda}_w )</td>
<td>( 0.48 f_{ys} / \tilde{\lambda}_w )</td>
<td></td>
</tr>
<tr>
<td>( \tilde{\lambda}_w \geq 1.40 )</td>
<td>( 0.67 f_{ys} / \tilde{\lambda}_w^2 )</td>
<td>( 0.48 f_{ys} / \tilde{\lambda}_w )</td>
<td></td>
</tr>
</tbody>
</table>

*Stiffening at the support, such as cleats, arranged to prevent distortion of the web and designed to resist the support reaction.

Figure B.2: Shear buckling strength \( f_{bv} \) [5, p.45]
B.1.3. The Service Life

**Actions**

**Permanent actions**

The dry weight of the concrete is taken into account.

\[ g_{c,\text{dry}} = g_{c,\text{wet}} \cdot \frac{\rho_{c,\text{dry}}}{\rho_{c,\text{wet}}} = 3.08 \cdot \frac{24}{26} = 2.85 \text{ kN/m}^2 \]

The weight of the reinforcement is determined by assuming tensile reinforcement of round ø20 mm in each rib and a top reinforcement mesh of round ø8 mm at a distance of 200 mm in both directions:

\[
g_s = \left( \pi \left( \frac{\text{ø}_{\text{bot}}}{2} \right)^2 \cdot \frac{1000}{b_d} + 2 \cdot \left( \pi \left( \frac{\text{ø}_{\text{top}}}{2} \right)^2 \cdot \frac{1000}{200} \right) \right) / 1000^2 \cdot \rho_s \cdot g / 1000
\]

\[
= \left( \pi \left( \frac{20}{2} \right)^2 \cdot \frac{1000}{260} + 2 \cdot \left( \pi \left( \frac{8}{2} \right)^2 \cdot \frac{1000}{200} \right) \right) / 1000^2 \cdot 7800 \cdot 9.81 / 1000 = 0.13 \text{ kN/m}^2
\]

**Variable actions**

The NEN-EN 1991-1-1 gives imposed floor loads, that depend on the function of the building. A uniform load of 4.0 kN/m\(^2\) and a concentrated load of 7.0 kN are valid values for most building types.

**Serviceability limit state (SLS)**

**Partial factors for actions**

<table>
<thead>
<tr>
<th>Partial factor for permanent actions</th>
<th>( \gamma_g )</th>
<th>1.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Partial factor for variable actions</td>
<td>( \gamma_q )</td>
<td>1.0</td>
</tr>
</tbody>
</table>

**Permanent actions**

- Weight steel deck \( g_p : 0.26 \text{ kN/m}^2 \)
- Weight reinforcing steel \( g_s : 0.13 \text{ kN/m}^2 \)
- Weight concrete \( g_c : 2.85 \text{ kN/m}^2 \)
- Total \( g_k : 3.24 \text{ kN/m}^2 \)

**Variable actions**

- Imposed floor load (uniform distributed load) \( q_k : 4.0 \text{ kN/m}^2 \)
- Imposed floor load (concentrated load) \( Q_k : 7.0 \text{ kN} \)

**Combination of actions at SLS**

Load combination 1: \( q_d = \gamma_g g_k + \gamma_q q_k = 3.24 + 4.0 = 7.24 \text{ kN/m}^2 \) (uniform load)

**Bending stiffness**

The bending stiffness for positive bending is determined according NEN-EN 1994-1-1. Factor \( n \) is used to take the contribution of the concrete into account:

\[
n = \frac{E_a}{\frac{1}{2} \left( E_{cm} + \frac{1}{3} E_{cm} \right)} = \frac{210000}{\frac{1}{2} \left( 300000 + \frac{1}{3} 30000 \right)} = 10.5
\]

Note that the effects of creep and shrinkage are taken into account in \( n \) (time dependent).

**Uncracked cross-section**

The height of the compression zone is determined with:

\[
x_u = \frac{b \frac{1}{2} h_c^2 + b_{om} h_p \left( h - \frac{1}{2} h_p \right) + n A_p d_p}{b h_c + b_{om} h_p + n A_p e}
\]

Where:
\( b_{om} \) is the average rib width per meter slab width;
\( d_p \) is the distance from the centroid of the deck to the top of the slab.

Having the following values:

\[
b_{om} = b_u \cdot \left( \frac{1000}{b_d} \right) = 50 \cdot \left( \frac{1000}{260} \right) = 192.3 \text{ mm}
\]

\[
d_p = h - e = 270 - 107.7 = 162.3 \text{ mm}
\]

\( h_c = 60 \text{ mm} \)

Therefore the height of the compression zone is:

\[
x_u = \frac{1000 \cdot 60^2 + 192.3 \cdot 210 \left( 270 - \frac{1}{2} \cdot 210 \right) + 10.5 \cdot 2141 \cdot 162.3}{1000 \cdot 60 + 160 \cdot 210 + 10.5 \cdot 2141} = 98.58 \text{ mm}
\]

The second moment of area of the uncracked cross-section is:

\[
I_{c,u} = \frac{bh_c^3}{12} + \frac{bh_c}{n} \left( x_u - \frac{1}{2} h_c \right)^2 + \frac{b_{om} h_p^3}{12n} + \frac{b_{om} h_p}{n} \left( n - x_u - \frac{1}{2} h_p \right)^2 + A_{pe} \left( d_p - x_u \right)^2 + I_{eff}
\]

\[
= \frac{1000 \cdot 60^3}{12 \cdot 10.5} + \frac{1000 \cdot 60}{10.5} \left( \frac{98.58 - \frac{1}{2} \cdot 60}{2} \right)^2 + \frac{192.3 \cdot 210^3}{12 \cdot 10.5} + \frac{192.3 \cdot 210}{10.5} \left( 270 - 98.58 - \frac{1}{2} \cdot 210 \right)^2 + 2141 \left( 162.3 - 98.58 \right)^2 + 1786 \cdot 10^4 = 86249696 \text{ mm}^4 / \text{m}
\]

**Cracked cross-section**

The height of the compression zone is determined with:

\[
x_c = \frac{n A_{pe}}{b} \left( \sqrt{1 + \frac{2bd_p}{n A_{pe}}} - 1 \right) = \frac{10.5 \cdot 2141}{1000} \left( \sqrt{1 + \frac{2 \cdot 1000 \cdot 162.3}{10.5 \cdot 2141}} - 1 \right) = 65.85 \text{ mm}
\]

The second moment of area of the cracked cross-section is:

\[
I_{c,c} = \frac{bx_c^3}{3n} + A_{pe} \left( d_p - x_c \right)^2 + I_{eff}
\]

\[
= \frac{1000 \cdot 65.85^3}{3 \cdot 10.5} + 2141 \left( 162.3 - 65.85 \right)^2 + 1786 \cdot 10^4 = 46847188 \text{ mm}^4 / \text{m}
\]

**Average bending stiffness**

The average bending stiffness of the slab is:

\[
I_{c,m} = \frac{1}{2} \left( I_{c,u} + I_{c,c} \right) = \frac{1}{2} (86249696 + 46847188) = 66548442 \text{ mm}^4 / \text{m}
\]

**Bending stiffness (short-term loading)**

The modular ratio for short-term loading:

\[
n_0 = \frac{E_a}{E_{cm}} = \frac{210000}{30000} = 7
\]
The short-term bending stiffness of the composite slab:

\[
x_u = \frac{1000 \cdot 60^2 + 192.3 \cdot 210 (270 - \frac{1}{2} \cdot 210) + 7 \cdot 2141 \cdot 162.3}{1000 \cdot 60 + 192.3 \cdot 210 + 7 \cdot 2141} = 94.44 \text{ mm}
\]

\[
I_{c,u} = \frac{1000 \cdot 60^3}{12 \cdot 7} + \frac{1000 \cdot 60}{7} \left( \frac{94.44 - \frac{1}{2} \cdot 60}{12 \cdot 7} \right)^2 + \frac{192.3 \cdot 210^3}{12 \cdot 7} + \frac{192.3 \cdot 210}{7} \left( \frac{270 - 94.44 - \frac{1}{2} \cdot 210}{12 \cdot 7} \right)^2 + 2141 (162.3 - 94.44)^2 + 1786 \cdot 10^4 = 93936221 \text{ mm}^4/m
\]

\[
x_c = \frac{7 \cdot 2141}{1000} \left( \sqrt{1 + 2 \cdot 1000 \cdot 162.3 \cdot 7 \cdot 2141} - 1 \right) = 56.35 \text{ mm}
\]

\[
I_{c,c} = \frac{1000 \cdot 56.35^3}{3 \cdot 7} + 2141 (162.3 - 56.35)^2 + 1786 \cdot 10^4 = 50419779 \text{ mm}^4/m
\]

\[
I_{c,m} = \frac{1}{2} (I_{c,u} + I_{c,c}) = \frac{1}{2} (93936221 + 50419779) = 72178000 \text{ mm}^4/m
\]

**Deflections**

The criteria for the maximum deflections are stated in NEN-EN 1990 NA A.1.4.3.

The total deflection due to the permanent and variable loads is (time dependent):

\[
w_{tot} = \frac{5}{384} \frac{q_k L^4}{E I_{c,m}} = \frac{5}{384} \frac{3.24 \cdot 4.0 \cdot 7200^4}{210000 \cdot 66548442} = 18.1 \text{ mm} \leq L/250 = 28.8 \text{ mm}
\]

The bending stiffness of the steel-concrete composite slab is sufficient to limit the total deflections. Note that the deflections during construction are not included.

The time independent deflection of the permanent load is:

\[
w_2 = \frac{5}{384} \frac{q_k L^4}{E I_{c,m}} = \frac{5}{384} \frac{3.24 \cdot 7200^4}{210000 \cdot 72178000} = 7.5 \text{ mm}
\]

The additional deflection due to the variable loads is:

\[
w_{addl} = w_{tot} - w_2 = 18.1 - 7.5 = 10.6 \text{ mm} \leq L/500 = 14.4 \text{ mm}
\]

The bending stiffness of the steel-concrete composite slab is sufficient to prevent to large additional deflections.

**Vibrations**

The criteria for vibrations are stated in NEN-EN 1990 NA A.1.4.4 (3).

The required eigenfrequency depends on the function of a building. Dwelling and offices require a eigenfrequency of at least be 3 Hz and gyms or dancing halls at least 5 Hz. The eigenfrequency should be determined with a quasi-permanent load-combination according to NEN-EN 1990 6.16b for a short-term loading situation. Note that verification for vibrations for loads over 5 kN/m² or 150 kN is not required.

Quasi-permanent load-combination:

\[
q_d = g_k + \Psi_2 q_k = 3.24 + 0.3 \cdot 4 = 4.44 \text{ kN/m}
\]

Where \( \Psi_2 \) is 0.3 found in table A.1.1.

Maximum deflection of the composite slab under a quasi-permanent load:

\[
\delta = \frac{5}{384} \frac{q_d L^4}{E I_{c,m}} = \frac{5}{384} \frac{4.44 \cdot 7200^4}{210000 \cdot 72178000} = 10.25 \text{ mm}
\]

NEN 6702 gives a formula for the eigenfrequency:

\[
f_e = \sqrt{\frac{a}{\delta}}
\]

Where:
\( a \) is a value for the type of dynamic system: 0.315 m/s\(^2\) for a single span beam with uniform load;

\( \delta \) is the maximum deflection of the composite slab under a quasi-permanent load in meter.

This gives that: 
\[
f_e = \sqrt{\frac{0.315}{0.01065}} = 5.54 \text{ Hz} > 5 \text{ Hz}
\]
and therefore is the performance of the design sufficient regarding the vibrations.

**Cracking of the concrete**

The steel-concrete composite slab is designed simply supported and therefore only anti-crack reinforcement is required. NEN-EN 1994-1-1 section 9.8.1 prescribes that, the cross-sectional area of the reinforcement above the ribs (\( A_s \)) of the deck should not be less than 0.4% of the cross-sectional area of the concrete above the ribs, for unpropped constructions.

\[
A_s \geq 0.4\% \cdot bh_c = 0.004 \cdot 1000 \cdot 60 = 240 \text{ mm}^2/m
\]

\( \phi 8\)-200 mm provides sufficient reinforcement: 
\[
A_s = 251 \text{ mm}^2/m > 0.4\% \cdot bh_c
\]

**Fire resistance**

The fire resistance of a composite slab can be determined with the method described in NEN-EN 1994-1-2. The scope of the method is defined in table B.1, where the rib geometry is illustrated in figure B.3. From table B.1 is clear that the method is not applicable for deep deck composite slabs, because the width of the top flange \( l_3 \) and the height of the profile \( h_2 \) are exceeding the scope.

<table>
<thead>
<tr>
<th>Field of application</th>
<th>Design value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( 80.0 \leq l_1 \leq 155.0 \text{ mm} )</td>
<td>50 mm</td>
</tr>
<tr>
<td>( 32.0 \leq l_2 \leq 132.0 \text{ mm} )</td>
<td>50 mm</td>
</tr>
<tr>
<td>( 40.0 \leq l_3 \leq 115.0 \text{ mm} )</td>
<td>\textbf{210 mm}</td>
</tr>
<tr>
<td>( 50.0 \leq h_1 \leq 125.0 \text{ mm} )</td>
<td>60 mm</td>
</tr>
<tr>
<td>( 50.0 \leq h_2 \leq 100.0 \text{ mm} )</td>
<td>\textbf{210 mm}</td>
</tr>
</tbody>
</table>

Table B.1: Field of application of the EC4-2 method to determine the fire resistance of unprotected composite slabs

Figure B.3: Definition of the rib geometry for steel-concrete composite slabs [11, p.79]
**Ultimate limit state (ULS)**

### Partial factors for actions

| Partial factor for permanent actions | \( \gamma_g \) | 1.2 - |
| Partial factor for variable actions | \( \gamma_q \) | 1.5 - |

### Permanent actions

| Weight steel deck | \( g_p \) | 0.26 kN/m² |
| Weight reinforcing steel | \( g_s \) | 0.13 kN/m² |
| Weight concrete | \( g_c \) | 2.85 kN/m² |
| Total | \( g_k \) | 3.24 kN/m² |

### Variable actions

| Imposed floor load (uniform distributed load) | \( q_k \) | 4.0 kN/m² |
| Imposed floor load (concentrated load at 100 mm square) | \( Q_k \) | 7.0 kN |

### Combination of actions at SLS

**Load combination 1:**

\( q_d = \gamma_g g_p + \gamma_q q_k = 1.2 \cdot 3.24 + 1.5 \cdot 4.0 = 9.9 \text{ kN/m}^2 \) (uniform load)

**Load combination 2:**

\( \gamma_g g_k + \gamma_q Q_k = (1.2 \cdot 3.89 = 3.89 \text{ kN/m}^2) + (1.5 \cdot 7.0 = 10.5 \text{ kN}) \) (uniform + point load)

### Design moment and shear force

The maximum design moment and shear force are determined by the most severe load combination:

\[
M_{Ed,1} = \frac{1}{8} q_d L^2 = \frac{1}{8} \cdot 9.9 \cdot 7.2^2 = 64.2 \text{ kNm/m}
\]

\[
M_{Ed,2} = \frac{1}{8} q_k L^2 + \frac{1}{4} Q_k L = \frac{1}{8} \cdot 3.89 \cdot 7.2^2 + \frac{1}{4} \cdot 10.5 \cdot 7.2 = 44.1 \text{ kNm/m (if point load is applied at midspan)}
\]

\[
V_{Ed,1} = \frac{1}{2} q_d L = \frac{1}{2} \cdot 10.09 \cdot 7.2 = 35.7 \text{ kNm/m}
\]

\[
V_{Ed,2} = \frac{1}{2} \cdot (g_k L + Q_k) = \frac{1}{2} \cdot (3.89 \cdot 7.2 + 10.5) = 19.3 \text{ kNm/m}
\]

Here is assumed that the point load is distributed over 1.0 meter width. The maximum design moment and shear force per meter width of deck are:

\[
M_{Ed} = \max(M_{Ed,1} ; M_{Ed,2}) = 64.2 \text{ kNm/m}
\]

\[
V_{Ed} = \max(V_{Ed,1} ; V_{Ed,2}) = 35.7 \text{ kNm/m}
\]

### Bending resistance

#### Location of the plastic neutral axis (p.n.a.)

Maximum compressive design force of the concrete above the sheeting:

\[
N_{c,f} = 0.85 f_{cd} h_c b = 0.85 \cdot 13.33 \cdot 60 \cdot 1000 = 680000 \text{ N/m} \]

The maximum tensile resistance of the deck:

\[
N_p = A_{p e f_y p, d} = 2141 \cdot 550 = 1177423 \text{ N/m} \]

Since \( N_{c,f} < N_p \) lies the neutral axis in the deck. Therefore, the sagging bending resistance should be determined according to the stress distribution as shown in figure B.4.

### Full shear connection

\( z \) and \( M_{pr} \) are determined according to NEN-EN 1994-1-1 with the following expressions:

\[
z = h - 0.5 h_c - e_p + (e_p - e) \frac{N_{c,f}}{A_{p e f_y p, d}}
\]

\[
M_{pr} = 1.25 M_{pa} \left( 1 - \frac{N_{c,f}}{A_{p e f_y p, d}} \right) \leq M_{pa}
\]

\[
M_{pl,Rd} = N_{c,f} z + M_{pr}
\]
Where:

\( z \)  
the internal lever arm;

\( M_{pr} \)  
reduced plastic resistance moment of the deck;

\( M_{pa} \)  
design value of the plastic resistance moment of the effective cross-section of the deck;

\( M_{pl,Rd} \)  
design value of the plastic resistance moment of the composite slab with full shear connection.

Giving:

\[
\begin{align*}
  z &= 270 - 0.5 \cdot 60 - 109.6 + (109.6 - 107.7) \frac{680000}{2141550} = 131.5 \text{ mm} \\
  M_{pa} &= 52.5 \text{ kNm/m} \\
  M_{pr} &= 1.25 \cdot 52.5 \cdot 10^6 \left( 1 - \frac{680000}{2141 \cdot 550} \right) = 27715586 \text{ Nmm/m} = 27.72 \text{ kNm/m} \\
  M_{pl,Rd} &= 680000 \cdot 131.5 + 27715586 = 117149942 \text{ Nmm/m} = 117.1 \text{ kNm/m}
\end{align*}
\]

The following criteria should be met:

\[
UC = \frac{M_{Ed}}{M_{pl,Rd}} = \frac{64.2}{117.1} = 0.55 < 1
\]

The moment resistance of the steel-concrete composite slab for full shear connection is sufficient.

### Longitudinal shear resistance: m-k method

The m-k method is described in NEN-EN 1994-1-1. It should be shown that the vertical shear resistance \( V_{l,Rd} \) exceeds the design shear force \( V_{Ed} \). The design shear resistance is determined with the following expression:

\[
V_{l,Rd} = \frac{bd_p}{\gamma_{VS}} \left( \frac{m A_p}{b L_s} + k \right)
\]

Where:

\( b, d_p \)  
are in mm;

\( A_p \)  
nominal cross-section of the sheeting in mm\(^2\);

\( m, k \)  
design values for the empirical factors in N/mm\(^2\);

\( \gamma_{VS} \)  
partial safety factor for ULS.

Shear length for a single span slab with uniform load over the entire length:

\[
L_s = 0.25L = 0.25 \cdot 7200 = 1800 \text{ mm}
\]
The $m$ and $k$ values for the Comflor 210 floor are used:

\[ m = 214.69 \text{ N/mm}^2 \]
\[ k = 0.01 \text{ N/mm}^2 \]

The design shear resistance:

\[ V_{l,Rd} = \frac{1000 \cdot 162.3}{1.25} \left( \frac{214.69 \cdot 3397 + 0.01}{1000 \cdot 1800} \right) = 53915 \text{ N/m} = 53.91 \text{ kN/m} \]

The design shear resistance should exceed the design vertical shear.

\[ UC = \frac{V_{Ed}}{V_{l,Rd}} = \frac{35.7}{53.9} = 0.66 < 1 \]

The design resistance for longitudinal shear is sufficient.

**Shear force**

**Without tensile shear reinforcement**

The verification of the vertical shear resistance is based on NEN-EN 1992-1-1 equation 6.2b:

\[ V_{Rd,c} = (\nu_{\text{min}} + k \sigma_{cp}) b_{w} d \]
\[ \nu_{\text{min}} = 0.035 k^{3/2} f_{ck}^{1/2} \]

Where:

\[ k = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \text{ with } d \text{ in mm;} \]
\[ b_{w} \text{ is the average width of the concrete ribs.} \]

Note that the NEN-EN 1992-1-1 takes only the part of the concrete ribs into account and the possible contribution of the steel deck is neglected.

\[ d = d_{p} = 162.3 \text{ mm} \]
\[ b_{w} = 50 \text{ mm} \]
\[ k = 1 + \sqrt{\frac{200}{162.3}} = 2.11 > 2.0 \]
\[ \nu_{\text{min}} = 0.035 \cdot 2^{3/2} \cdot 20^{1/2} = 0.443 \text{ N/mm}^2 \]

The design is assumed to be simply supported and therefore the beneficial effect of the hogging bending moment is neglected: $\sigma_{cp} = 0$. Hence,

\[ V_{Rd,c} = 0.443 \cdot 50 \cdot 162.3 = 3.59 \text{ kN/rib} = 13.8 \text{ kN/m} \]

The design shear resistance should exceed the design vertical shear.

\[ UC = \frac{V_{Ed}}{V_{Rd,c}} = \frac{36.3}{13.8} = 2.59 > 1 \]

Tensile reinforcement is required to provide sufficient shear resistance.

**With tensile shear reinforcement**

The verification of the vertical shear resistance is based on NEN-EN 1992-1-1 equation 6.2a.

\[ V_{Rd,c} = \left[ C_{Rd,c} k \left( 100 \rho_{l} f_{ck} \right)^{1/3} + k_{1} \sigma_{cp} \right] b_{w} d \]

Where:

\[ C_{Rd,c} \text{ value recommended as 0.18}/\gamma_{c}; \]
\[ \rho_{l} = \frac{A_{s}}{b_{w} d} \leq 0.02; \]
$A_{sl}$ is the area of the tensile reinforcement;

$b_{w}$ is the smallest width of the cross-section in the tensile area.

Assumed is that the tensile reinforcement of ø20 mm is placed 30 mm from the bottom flange of the deck.

\[
d = d_s = h - 30 = 270 - 30 = 240 \text{ mm}
\]

\[
b_{w} = 50 \text{ mm}
\]

\[
k = 1 + \sqrt{\frac{200}{240}} = 1.91 \leq 2.0
\]

\[
C_{Bd,c} = \frac{0.18}{1.5} = 0.12
\]

\[
A_{sl} = \frac{\pi}{2} = \frac{\pi \cdot 20^2}{2} = 314 \text{ mm}^2
\]

\[
\rho_1 = \frac{314}{50 \cdot 240} = 0.026 > 0.02
\]

\[
V_{Rd,c} = [0.12 \cdot 1.91 (100 \cdot 0.02 \cdot 20)^{1/3} + 0] \cdot 50240 = 9420 \text{ N/rib} = 36.2 \text{ kN/m}
\]

The design shear resistance should exceed the design vertical shear.

\[
UJC = \frac{V_{Ed}}{V_{Rd,c}} = \frac{35.7}{36.2} = 0.99 < 1
\]

The vertical shear resistance is just enough to provide sufficient resistance. Note that a bigger bar diameter would not improve the vertical shear resistance, because $\rho_1$ is limited by 0.02.

**Punching shear**

Punching shear resistance of a slab is calculated with the following expression (NEN-EN 1992-1-1 section 6.4.4):

\[
\nu_{Rd,c} = C_{Rd,c} \nu_{min} \geq \nu_{min} [\text{N/mm}^2]
\]

Where:

\[
d
\]

is the average effective height of the reinforcement;

\[
\rho_1 = \sqrt{\rho_{lx,x} \cdot \rho_{ly,y}} \leq 0.02;
\]

\[
A_{sl}
\]

is the area of the tensile reinforcement;

\[
b_{w}
\]

is the smallest width of the cross-section in the tensile area.

Round ø8 mm at c.t.c. distance 200 mm is applied above the deck to prevent the cracking of the concrete in SLS. Assumed is that this reinforcement in both the x- and y-direction is equal. Hence, the effective reinforcement ratio is:

\[
A_s = \frac{\pi}{2} = \frac{\pi \cdot 8^2}{2} = 50.3 \text{ mm}^2 = 251 \text{ mm}^2 / \text{m}
\]

\[
\rho_{lx} = \rho_{ly} = \frac{A_s}{bh_c} = \frac{251}{1000 \cdot 60} = 0.004
\]

\[
\rho_1 = 0.004 < 0.02
\]

The perimeter $C_p$ and the area within the perimeter $O_p$ is determined according to figure B.5:

\[
C_p = 2 \pi h_c + 2 b_p + 2(a_p + 2d_p - 2h_c) = 2 \pi h_c + 2(100 + 2 \cdot 162.3 - 2 \cdot 60) = 1186 \text{ mm}
\]

\[
O_p = (2d_p + a_p)(2h_c + b_p) + (\pi - 4)h_c^2 = (2 \cdot 162.3 + 100)(2 \cdot 60 + 100) + (\pi - 4)60^2 = 90322 \text{ mm}^2
\]

According to the NEN-EN 1991-1-1 NA table 6.2 is the load surface of the concentrated load a square of 100 mm: $a_p = b_p = 100 \text{ mm}$.

\[
k = 1 + \sqrt{\frac{200}{162.3}} = 2.11 > 2.0
\]

\[
\nu_{min} = 0.035 \cdot 2^{3/2}20^{1/2} = 0.443 \text{ N/mm}^2
\]

\[
\nu_{Rd,c} = 0.12 \cdot (100 \cdot 0.004 \cdot 20)^{1/3} = 0.48 \text{ N/mm}^2
\]
The punching shear resistance is:

\[ d = \frac{1}{2} (d_x + d_y) = \frac{1}{2} (29 + 37) = 33 \text{ mm} \]

\[ V_{p,Rd} = v_{Rd,C} C_p d = 0.48 \cdot 1186 \cdot 33 = 18786 \text{ N} = 18.7 \text{ kN} \]

The load that should be resisted is equal to:

\[ V_{p,Ed} = O_{p,g} \gamma_g + Q_{k,q} = (30322/1000^2) \cdot 3.24 \cdot 1.2 + 7.5 \cdot 1.5 = 11.4 \text{ kN} \]

The punching shear resistance should exceed the vertical shear:

\[ UC = \frac{V_{p,Ed}}{V_{p,Rd}} = \frac{11.4}{18.7} = 0.61 < 1.0 \]

**B.1.4. CONCLUSION**

The proposed deck design and composite slab are verified during the construction stage and service life. In table B.2 an overview is given of the most important floor properties. Besides that, notes are made regarding assumptions and calculation methods. Finally, an overview is given of the different verification criteria.

**THE CONSTRUCTION STAGE**

Notes are made regarding the assumptions and calculation methods, that are used to verify the deck during construction. The overview of the verification criteria for the steel deck is given in table B.3.

- assumed is that a certain part of the cross-section of the deck is effective;
- embossments and stiffeners are not yet designed for;
- deflection during construction is governing.
### Table B.2: Properties of steel deck design and composite floor slab

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>h</td>
<td>270</td>
<td>mm</td>
<td>floor height</td>
</tr>
<tr>
<td>L</td>
<td>7.2</td>
<td>m</td>
<td>unpropped span capacity</td>
</tr>
<tr>
<td>G</td>
<td>348</td>
<td>kg/m²</td>
<td>self-weight of the composite floor slab</td>
</tr>
<tr>
<td>A_p</td>
<td>26.5</td>
<td>kg/m²</td>
<td>amount of steel deck</td>
</tr>
<tr>
<td>A_s</td>
<td>13.3</td>
<td>kg/m²</td>
<td>amount of reinforcement steel</td>
</tr>
<tr>
<td>V_c</td>
<td>116</td>
<td>L/m²</td>
<td>amount of concrete</td>
</tr>
<tr>
<td>G_{deck}</td>
<td>42.6</td>
<td>kg/unit</td>
<td>weight per deck panel</td>
</tr>
</tbody>
</table>

### Table B.3: Verification criteria for the steel deck during the construction stage in SLS and ULS

<table>
<thead>
<tr>
<th>Deck</th>
<th>SLS:</th>
<th>ULS:</th>
</tr>
</thead>
<tbody>
<tr>
<td>w_0</td>
<td>0.78</td>
<td>&lt; 1</td>
</tr>
<tr>
<td>M^+</td>
<td>0.70</td>
<td>&lt; 1</td>
</tr>
<tr>
<td>V</td>
<td>0.41</td>
<td>&lt; 1</td>
</tr>
</tbody>
</table>

### Table B.4: Verification criteria for the composite slab during service life in SLS and ULS

<table>
<thead>
<tr>
<th>Composite slab</th>
<th>SLS:</th>
<th>ULS:</th>
</tr>
</thead>
<tbody>
<tr>
<td>w_{addl}</td>
<td>0.74</td>
<td>&lt; 1</td>
</tr>
<tr>
<td>w_{tot}</td>
<td>0.63</td>
<td>&lt; 1</td>
</tr>
<tr>
<td>f_e</td>
<td>0.90</td>
<td>&lt; 1</td>
</tr>
<tr>
<td>M^+</td>
<td>0.55</td>
<td>&lt; 1</td>
</tr>
<tr>
<td>V</td>
<td>0.99</td>
<td>&lt; 1</td>
</tr>
<tr>
<td>F_{p}</td>
<td>0.61</td>
<td>&lt; 1</td>
</tr>
<tr>
<td>V_l</td>
<td>0.66</td>
<td>&lt; 1</td>
</tr>
</tbody>
</table>

### THE SERVICE LIFE

Notes are made regarding the assumptions and calculation methods used to verify the composite slab during service life. The overview of the verification criteria for the composite slab is given in table B.4.

- longitudinal shear resistance is based on test values of the Comflor 210, which is a conservative assumption, because the new design has a longer shear span, more ribs per meter width, and a thicker deck;
- the method to calculate the fire resistance is not applicable and therefore the fire resistance is not yet determined;
- situation where the composite slab is continues over multiple spans is not yet considered.

### B.1.5. DIFFERENT DESIGNS

Different designs are compared in table B.5. The comparison is made for ‘minimum floor height’ \((h_p=210\text{ mm})\) and different steel sheet thicknesses, \(t\) is 1.0, 1.25 or 1.5 mm. The governing criteria are indicated. Here is \(\delta_0\) the deflection of the deck during construction in SLS and \(V_{Rd,c}\) the vertical shear resistance of the composite slab during service life in ULS.
Different variants are verified for each design according to the method described in the previous example calculation. The deflection during construction is the governing criteria for all designs and the shear resistance of the composite slab is the governing design criteria for the design with small ribs.

<table>
<thead>
<tr>
<th></th>
<th>Design 1: small ribs</th>
<th>Design 2: wide ribs</th>
</tr>
</thead>
<tbody>
<tr>
<td>( t ) (mm)</td>
<td>1.0 1.25 1.5</td>
<td>1.0 1.25 1.5</td>
</tr>
<tr>
<td>( h_p ) (mm)</td>
<td>210 210 210</td>
<td>210 210 210</td>
</tr>
<tr>
<td>( b_0 ) (mm)</td>
<td>150 210 280</td>
<td>60 60 60</td>
</tr>
<tr>
<td>( b_u ) (mm)</td>
<td>40 50 70</td>
<td>150 210 265</td>
</tr>
<tr>
<td>( b_d ) (mm)</td>
<td>190 260 350</td>
<td>210 270 325</td>
</tr>
<tr>
<td>( b_c ) (mm)</td>
<td>0 0 0</td>
<td>20 20 37</td>
</tr>
<tr>
<td>( A_p ) (mm²)</td>
<td>3284 3397 3504</td>
<td>3145 3361 3616</td>
</tr>
<tr>
<td>( A_{pe} ) (mm²)</td>
<td>2223 2141 2086</td>
<td>2331 2554 2808</td>
</tr>
<tr>
<td>( I_{eff} ) (cm⁴)</td>
<td>1812 1786 1785</td>
<td>2002 2198 2379</td>
</tr>
<tr>
<td>( G ) (kg/m²)</td>
<td>360 348 351</td>
<td>399 445 474</td>
</tr>
</tbody>
</table>

Governing criteria: \( \delta_0 \), \( V_{Rd,c} \)

Table B.5: Variant study of two types of deck designs
B.2. Trapezoidal Deck Design

In this section two decks from the previous section are designed as trapezoidal shaped elements. For the verification is only the deflection \( \delta_0 \) during construction considered. The slant angle \( \alpha \) is designed at 80°. The trapezoidal deck design are redesigned if the deflections during construction are exceeding the maximum.

B.2.1. Design Concept 1: Small Ribs

The different variants of design concept 1 are investigated and the results are given in table B.6.

The design with small ribs, of the previous section, is redesigned with a web angle of 80°. The deflection criteria of this design are not met, because the amount of effective steel is decreased due to the smaller width of the top flange. Besides that, the amount of concrete is increased due the trapezoidal shapes of the ribs.

Design variants with, \( t = 1.5 \text{ mm} \), \( h_p = 240 \text{ mm} \), and \( b_d = 200 \text{ mm} \) are used to verify the effect of the sheet thickness, the profile height and the deck element width, respectively.

<table>
<thead>
<tr>
<th></th>
<th>( \alpha = 90^\circ )</th>
<th>( \alpha = 80^\circ )</th>
<th>( t )</th>
<th>( h_p )</th>
<th>( b_d )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( t ) (mm)</td>
<td>1.25</td>
<td>1.25</td>
<td>1.5</td>
<td>1.25</td>
<td>1.25</td>
</tr>
<tr>
<td>( h_p ) (mm)</td>
<td>210</td>
<td>210</td>
<td>210</td>
<td>240</td>
<td>210</td>
</tr>
<tr>
<td>( b_0 ) (mm)</td>
<td>210</td>
<td>146</td>
<td>146</td>
<td>137</td>
<td>96</td>
</tr>
<tr>
<td>( b_u ) (mm)</td>
<td>50</td>
<td>40</td>
<td>40</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>( b_d ) (mm)</td>
<td>260</td>
<td>260</td>
<td>260</td>
<td>260</td>
<td>200</td>
</tr>
<tr>
<td>( \alpha ) (°)</td>
<td>90</td>
<td>80</td>
<td>80</td>
<td>80</td>
<td>80</td>
</tr>
<tr>
<td>( A ) (mm(^2))</td>
<td>3397</td>
<td>3036</td>
<td>3664</td>
<td>3232</td>
<td>3320</td>
</tr>
<tr>
<td>( A_{pe} ) (mm(^2))</td>
<td>2141</td>
<td>2065</td>
<td>2492</td>
<td>2065</td>
<td>2337</td>
</tr>
<tr>
<td>( I_{eff} ) (cm(^4))</td>
<td>1786</td>
<td>1680</td>
<td>2027</td>
<td>2165</td>
<td>1896</td>
</tr>
<tr>
<td>( \delta_0 ) (mm)</td>
<td>40.0</td>
<td>49.4</td>
<td>40.1</td>
<td>39.5</td>
<td>45.6</td>
</tr>
</tbody>
</table>

Table B.6: Different variants of design concept 1: small ribs

B.2.2. Design Concept 2: Wide Ribs with Fill Element

The different variants of design concept 2 are investigated and the results are given in table B.7.

The design with wide ribs, of the previous section, is redesigned with a web angle of 80°. The deflection criteria of this design are not met for the same reasons as given above.

Variants \( h_p = 245 \text{ mm} \), \( t = 1.25 \text{ mm} \), \( t = 1.25 \text{ mm} \) and \( h_p = 220 \text{ mm} \), and \( t = 1.5 \text{ mm} \) are used to verify the effect of the profile height, the sheet thickness, and deck element width.
Table B.7: Different variants of design concept 2: wide ribs with fill element

<table>
<thead>
<tr>
<th>$\alpha$ (°)</th>
<th>$t$ (mm)</th>
<th>$h_p$ (mm)</th>
<th>$b_u$ (mm)</th>
<th>$b_d$ (mm)</th>
<th>$A$ (mm$^2$)</th>
<th>$A_{pe}$ (mm$^2$)</th>
<th>$I_{eff}$ (cm$^4$)</th>
<th>$\delta_0$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>90°</td>
<td>1.0</td>
<td>210</td>
<td>60</td>
<td>150</td>
<td>3145</td>
<td>2331</td>
<td>2002</td>
<td>40.0</td>
</tr>
<tr>
<td>80°</td>
<td>1.0</td>
<td>210</td>
<td>60</td>
<td>150</td>
<td>3114</td>
<td>2010</td>
<td>1625</td>
<td>56.9</td>
</tr>
<tr>
<td>80°</td>
<td>1.0</td>
<td>245</td>
<td>60</td>
<td>64</td>
<td>3586</td>
<td>1953</td>
<td>2195</td>
<td>39.7</td>
</tr>
<tr>
<td>80°</td>
<td>1.25</td>
<td>210</td>
<td>60</td>
<td>76</td>
<td>3683</td>
<td>2533</td>
<td>2049</td>
<td>42.5</td>
</tr>
<tr>
<td>80°</td>
<td>1.25</td>
<td>210</td>
<td>60</td>
<td>72</td>
<td>4327</td>
<td>2513</td>
<td>2246</td>
<td>38.4</td>
</tr>
<tr>
<td>80°</td>
<td>1.5</td>
<td>210</td>
<td>60</td>
<td>76</td>
<td>4327</td>
<td>3057</td>
<td>2472</td>
<td>34.7</td>
</tr>
</tbody>
</table>

B.3. EFFECTIVE CROSS-SECTION

B.3.1. INTRODUCTION

In previous calculations is assumed that a certain part of the cross-section is effective (figure B.6a). Stiffeners were not designed for. The goal of this section is to design and calculate if the assumed amount of effective steel is realistic. The effective part of the cross-section is determined according to the ‘effective width’ method as given in NEN-EN 1993-1 part 3 and part 5.

![Effective part of the cross-section](image)

Figure B.6: Effective part of the cross-section

B.3.2. NO STIFFENERS

Firstly, the effective steel area of design with small ribs without stiffeners is calculated according to the effective width method. The geometry of this design is given in table B.6. A brief overview is here given:

- $t_0 = 1.46$ design thickness deck in mm;
- $b_u = 40$ width bottom flange in mm;
- $h_p = 210$ height of the profiled steel sheeting in mm;
- $b_d = 260$ width of single deck panel in mm;
- $\alpha = 80^\circ$ angle of web to flange;
- $b_0$ width of the top flange;
- $s_w$ slant height.
The cross-sectional properties are determined for a symmetrical half of the deck element according to the method described in NEN-EN 1993-1-3 Annex C1. The part that is considered is illustrated in figure B.7.

- cross-section is divided into $n$ parts. Numbered from 1 to $n$.
- nodes are inserted between the parts. Numbered from 0 to $n$.
- part $i$ is then defined by node $i-1$ and $i$.

![Figure B.7: Labelling of nodes and parts for half a deck element](image)

Area of cross-section part:

$$dA_i = t_i \sqrt{+ (x_i - x_{i-1})^2 + (y_i - y_{i-1})^2}$$  \hspace{1cm} (B.1)

Cross-section area:

$$A = \sum_{i=1}^{n} dA_i$$  \hspace{1cm} (B.2)

First moment of area with respect to x-axis and the coordinate for gravity:

$$S_{x0} = \sum_{i=1}^{n} \left( y_i + y_{i-1} \right) \cdot \frac{dA_i}{2}$$

$$y_{gc} = \frac{S_{x0}}{A}$$  \hspace{1cm} (B.3)

This gives the following cross-sectional properties for half a deck element:

- $A = 476 \text{ mm}^2$;
- $S = 55063 \text{ mm}^3$;
- $y_{gc} = 115.6 \text{ mm}$.

Therefore, the amount of steel per meter width is:

$$A_p = A \cdot \frac{1000}{b_d/2} = 476 \cdot \frac{1000}{(260/2)} = 3664 \text{ mm}^2 / \text{m}$$
Reduction of the compression flange

Eurocode 3 part 1-5 provides expressions to determine the effective width for an internal element under compression or bending, where a $\psi$-factor describes the internal stress distribution [13, p.21]. The effective area of the element under compression becomes:

$$A_{\text{eff}} = \rho A$$

Where $\rho$ is the reduction factor for buckling.

$$\rho = 1.0 \quad \text{for } \bar{\lambda}_p \leq 0.5 + \sqrt{0.085 - 0.055\Psi}$$
$$\rho = \left(\bar{\lambda}_p - 0.055(3 + \Psi)\right) / \bar{\lambda}_p^2 \leq 1.0 \quad \text{for } \bar{\lambda}_p \geq 0.5 + \sqrt{0.085 - 0.055\Psi} \quad (B.4)$$

$$\epsilon = \sqrt{\frac{235}{f_y [\text{MPa}]}}$$
$$\bar{\lambda}_p = \frac{\bar{b}/t}{28.4\epsilon \sqrt{k_{\sigma}}} \quad (B.5)$$

According to NEN-EN 1993-1-5 table 4.1:

- $k_{\sigma} = 4$ is the buckling factor;
- $\Psi = 1$ is a factor for the internal stress distribution;
- $\bar{b} = b_0$ is the width considered.

Where:

$$\epsilon = \sqrt{\frac{235}{350}} = 0.819$$
$$\bar{\lambda}_p = \frac{145.94/1.46}{28.4 \cdot 0.819 \cdot \sqrt{4}} = 2.148$$
$$\bar{\lambda}_p \geq 0.5 + \sqrt{0.085 - 0.055\Psi} = 0.5 + \sqrt{0.085 - 0.055 \cdot 1} = 0.673$$
$$\rho = \left(\bar{\lambda}_p - 0.055(3 + \Psi)\right) / \bar{\lambda}_p^2 = (2.148 - 0.055(3 + 1)) / 2.148^2 = 0.418 \leq 1.0$$

And thus:

$$b_{\text{eff}} = \rho \bar{b} = 0.418 \cdot 145.94 = 60.99 \text{ mm}$$
$$b_{e1} = 0.5 b_{\text{eff}} = 0.5 \cdot 60.99 = 30.50 \text{ mm}$$
$$b_{e2} = 0.5 b_{\text{eff}} = 0.5 \cdot 60.99 = 30.50 \text{ mm}$$

New position of the neutral axis

The reduced area of the compression flange leads to a downward shift of the neutral axis (n.a.). The new position of the n.a. is determined with the expressions B.1, B.2, and B.3, taking the reduced width of the compression flange into account.

$$y_{g1} = \frac{S_{x1}}{A} = \frac{28030}{281} = 99.8 \text{ mm}$$
Here is assumed that a certain part of the web is not effective, as previously illustrated in figure B.6a. The height of the tension zone becomes \( e_{t,1} = y_{gc,1} = 99.8 \) and the height of the compression zone is equal to \( e_{c,1} = h_p - y_{gc,1} = 210 - 99.8 = 110.2 \) mm.

**Reduction of the web under bending**

The factor for the internal stress distribution for the web under bending is:

\[
\psi = -\frac{b_t}{b_c} = -\frac{99.8}{110.2} = -0.906
\]

Where \( b_t = e_{t,1} \) and \( b_c = e_{c,1} \). According to NEN-EN 1993-1-5 table 4.1 is for \( 0 > \Psi > -1 \):

\[
k_\sigma = 7.81 - 6.29\Psi + 9.78\Psi^2 = 7.81 - 6.29 \cdot -0.906 + 9.78 \cdot (-0.906)^2 = 21.53
\]

Where:

\[
\tilde{\lambda}_p = \frac{\bar{b}/t}{28.4\sqrt{K_\sigma}} = \frac{210/1.46}{28.4 \cdot 0.819 \cdot 21.43} = 1.332
\]

\[
\tilde{\lambda}_p \geq 0.5 + 0.085 - 0.055\Psi = 0.5 + 0.085 - 0.055 \cdot -0.906 = 0.867
\]

\[
\rho = (\tilde{\lambda}_p - 0.055(3 + \Psi)) / \tilde{\lambda}_p^2 = (1.332 - 0.055(3 + (-0.906))) / 1.332^2 = 0.686 \leq 1.0
\]

With \( \bar{b} = h_p = 210 \) mm. And thus:

\[
b_{t,1} = 0.4\rho b_{t} = 0.5 \cdot 39.65 = 19.83 \text{ mm}
\]

\[
b_{c,2} = 0.6\rho b_{c} = 0.5 \cdot 39.65 = 23.79 \text{ mm}
\]

**Figure B.9: Internal elements under bending [13, p.21]**

**Reduced cross-section properties**

The reduced area of the compression flange and the web under bending leads to a further downward shift of the neutral axis. This shift is again determined with the expressions B.1, B.2, and B.3 for a symmetrical half of the deck element:

\[
A_{eff} = 215.4 \text{ mm}^2
\]

\[
S_{x,eff} = 16770.0 \text{ mm}^3
\]

\[
y_{gc,2} = \frac{S_{x,eff}}{A_{eff}} = \frac{16770.0}{215.4} = 77.9 \text{ mm}
\]

\[
e_{c,2} = h_p - y_{gc,2} = 210 - 77.9 = 132.1 \text{ mm}
\]

\[
e_{t,2} = 77.9 \text{ mm}
\]

The second moment of area is calculated with respect to the original x-axis \( I_{x,0,eff} \) and with respect to the new x-axis through the gravity centre \( I_{x,eff} \) (NEN-EN 1993-1-3 Annex C1 and figure B.7):

\[
I_{x,0,eff} = \sum_{i=1}^{n} [(y_{i})^2 + (y_{i-1})^2 + y_{i} \cdot y_{i-1}] \cdot \frac{dA_i}{3}
\]

\[
I_{x,eff} = I_{x,0} - A_{eff} y_{gc,2}^2
\]
This gives:

\[ I_{x,0,\text{eff}} = 303.1 \cdot 10^4 \text{ mm}^4 \]
\[ I_{x,\text{eff}} = 303.1 \cdot 10^4 - 215.4 \cdot 77.9^2 = 172.5 \cdot 10^4 \text{ mm}^4 \]

The section properties of the reduced cross-section per meter with are:

\[ A_{pe} = 215.4 \cdot \frac{1000}{(b_d/2)} = 1657 \text{ mm}^2/\text{m} \]
\[ I_{\text{eff}} = 172.5 \cdot 10^4 \cdot \frac{1000}{(b_d/2)} = 1327.1 \cdot 10^4 \text{ mm}^4/\text{m} \]

**Conclusion**

In table B.8 a comparison is made between the cross-section properties of the assumed effective steel area and the calculated effective area. It is clear that without stiffeners the amount of effective steel is significant lower than initially assumed. For this reason, the bending stiffness is lower and the deflections of the deck during construction (\(\delta_0\)) are exceeding the maximum allowed deflections.

<table>
<thead>
<tr>
<th>assumption 'effective width method'</th>
</tr>
</thead>
<tbody>
<tr>
<td>( A_p ) (mm(^2)) : 3664</td>
</tr>
<tr>
<td>( A_{pe} ) (mm(^2)) : 2492</td>
</tr>
<tr>
<td>( I_{\text{eff}} ) (cm(^4)) : 2027</td>
</tr>
<tr>
<td>( \delta_0 ) (mm) : 40.0</td>
</tr>
</tbody>
</table>

Table B.8: Effective section properties of design with small ribs if no stiffeners are designed

![Figure B.10: Effective area of the cross-section](image)

**B.3.3. Stiffeners**

**Two symmetric flange stiffeners**

NEN-EN 1993-1-3 5.5.3.4.2 (3) gives an expression to determine the elastic critical buckling stress of a flange stiffener.

\[ \sigma_{cr,s} = \frac{4.2 k_w E}{A_s \sqrt{\frac{I_s r^3}{8 b_1^3(3 b_e - 4 b_1)}}} \]

With:

\[ b_e = 2b_{p,1} + b_{p,2} + 2b_t \]
\[ b_1 = b_{p,1} + 0.5b_r \]

Where:
$b_{p,1}, b_{p,2}$ are illustrated in figure B.11;

$b_r$ is the overall width of a stiffener;

$A_s, I_s$ are the cross-section area and the second moment of area of the stiffener cross-section according to figure B.11;

$k_w$ is a coefficient.

A first assumption of the flange stiffener dimensions is made and illustrated in figure B.12.

- $b_{p,1} = 30$ mm;
- $b_r = 16$ mm;
- $h_s = 5.5$ mm.

\[ b_{p,2} = b_h - 2(b_{p,1} + b_r) = 145.94 - 2(30 + 16) = 53.94 \text{ mm} \]
\[ b_s = 2 \sqrt{(b_r/2)^2 + 5.5^2} = 2 \sqrt{(16/2)^2 + 5.5^2} = 19.42 \text{ mm} \]
\[ b_e = 2 \cdot 30 + 53.94 + 2 \cdot 19.42 = 152.78 \text{ mm} \]
\[ b_1 = 30 + 0.5 \cdot 16 = 38 \text{ mm} \]
B.3. EFFECTIVE CROSS-SECTION

\[ \bar{\lambda}_p = \frac{30/1.46}{28.4 \cdot 0.819 \cdot \sqrt{4}} = 0.441 < 0.5 + \sqrt{0.085 - 0.055} \Psi = 0.673 \]
\[ \Rightarrow \rho = 1.0 \]
\[ \Rightarrow b_{1,\text{eff}} = \rho \bar{b} = 1.0 \cdot 30 = 30 \text{ mm} \]

For part \( b_{p,2} \):
\[ \bar{b} = b_{p,2} = 53.94 \text{ mm}; \]
\[ k_\sigma = 4; \]
\[ \Psi = 1. \]
\[ \bar{\lambda}_p = \frac{53.94/1.46}{28.4 \cdot 0.819 \cdot \sqrt{4}} = 0.794 > 0.5 + \sqrt{0.085 - 0.055} \Psi = 0.673 \]
\[ \Rightarrow \rho = \frac{0.794 - 0.055(3 + 1)}{0.794} = 0.911 \]
\[ > 1.0 \]
\[ \Rightarrow b_{2,\text{eff}} = \rho \bar{b} = 0.911 \cdot 53.94 = 49.12 \text{ mm} \]

The cross-section area of the stiffener is:
\[ A_s = (0.5 b_{1,\text{eff}} + 0.5 b_{2,\text{eff}} + b_s) t_0 = (0.5 \cdot 30 + 0.5 \cdot 49.12 + 19.42) \cdot 1.46 = 86.11 \text{ mm}^2 \]

The second moment of area of the stiffener is determined with the method given by NEN-EN 1993-1-3 Annex C1 (equations B.1, B.2, B.3, B.6).

\[ A = 79.55 \text{ mm}^2; \]
\[ S = 359.6 \text{ mm}^3; \]
\[ y_{gc} = 4.52 \text{ mm}; \]
\[ I_{s,0} = 1834.7 \text{ mm}^4; \]
\[ I_s = 209.4 \text{ mm}^4. \]

**Coefficient \( k_w \)**

According to NEN-EN 1993-1-3 5.5.3.4.2 (5) and (7).

\[ k_w = \begin{cases} k_{w0} & \text{if } \frac{l_b}{s_w} \geq 2 \\ k_{w0} - (k_{w0} - 1) \left( \frac{l_b}{s_w} - \left( \frac{l_b}{s_w} \right)^2 \right) & \text{if } \frac{l_b}{s_w} < 2 \end{cases} \]

Here is \( s_w \) the slant height. \( k_{w0} \) is determined with the compression flange wave buckling length \( l_b \). For two intermediate stiffeners:

\[ l_b = 3.65 \sqrt{I_b t^3} = 3.65 \sqrt{209.4 \cdot 38^2 (3 \cdot 152.78 - 4 \cdot 38) / 1.46^3} = 269.61 \text{ mm} \]
\[ k_{w0} = \sqrt{\frac{(2b_e + s_w)(3b_e - 4b_1)}{b_1(4b_e - 6b_1) + s_w(3b_e - 4b_1)}} = \sqrt{\frac{(2 \cdot 152.78 + 213.34)(3 \cdot 152.78 - 4 \cdot 38)}{38(4 \cdot 152.78 - 6 \cdot 38) + 213.24(3 \cdot 152.78 - 4 \cdot 38)}} = 1.410 \]
\[ \Rightarrow k_w = \frac{1.410 - (1.411 - 0.1) \left( \frac{2 \cdot 269.61}{213.24} - \left( \frac{269.61}{213.24} \right)^2 \right)}{213.24} = 1.029 \]

**Elastic critical buckling stress**

\[ \sigma_{\text{cr,s}} = \frac{4.2 \cdot 1.029 \cdot 210000}{86.11 \sqrt{209.4 \cdot 1.46^3 / 8 \cdot 38^2 (3 \cdot 152.78 - 4 \cdot 38)}} = 143.01 \text{ N/mm}^2 \]
Reduced effective area of the stiffener

\[ A_{s,\text{red}} = \kappa_d A_s \frac{f_{yb}}{\sigma_{\text{com,ser}}} \leq A_s \]

Determine factor \( \kappa_d \) with NEN-EN1993-1-3 5.5.3.1 (7).

\[
\begin{align*}
\kappa_d &= 1.0 \quad \text{if } \bar{\lambda}_d \leq 0.65 \\
\kappa_d &= 1.47 - 0.723 \bar{\lambda}_d \quad \text{if } 0.65 < \bar{\lambda}_d < 1.38 \\
\kappa_d &= 0.66 \bar{\lambda}_d \quad \text{if } \bar{\lambda}_d \geq 1.38
\end{align*}
\]  (B.7)

Where:

\[ \bar{\lambda}_d = \sqrt{\frac{f_{yb}}{\sigma_{cr,s}}} \]

for unstiffened webs: \( \sigma_{cr,s} \rightarrow \kappa_d \)

for stiffened webs: \( \sigma_{cr,\text{com}} \rightarrow \kappa_d \)

First assume that webs are unstiffened.

\[ \bar{\lambda}_d = \sqrt{\frac{350}{143.01}} = 1.564 \]

\[ \Rightarrow \kappa_d = 0.66/1.564 = 0.422 \]

The reduced effective area and the reduced effective thickness are:

\[ A_{s,\text{red}} = \kappa_d A_s = 0.422 \cdot 86.11 = 36.33 \, \text{mm}^2 \]

\[ t_{\text{red}} = t \cdot \frac{A_{s,\text{red}}}{A_s} = 1.46 \cdot \frac{36.33}{86.11} = 0.62 \, \text{mm} \]

Reduced cross-section properties

The reduced cross-section properties are determined with the same method used in the previous section. The reduced effective thickness of the stiffener and the increased effective widths are taken into account.

\[
\begin{align*}
A_{\text{eff}} &= 228.1 \, \text{mm}^2 \\
S_{x,\text{eff}} &= 19419.1 \, \text{mm}^3 \\
y_{gc,2} &= \frac{S_{x,\text{eff}}}{A_{\text{eff}}} = \frac{19419.1}{228.1} = 85.1 \, \text{mm} \\
e_{c,2} &= h_p - y_{gc,2} = 210 - 85.1 = 124.9 \, \text{mm} \\
e_{t,2} &= 85.1 \, \text{mm} \\
I_{x,0,\text{eff}} &= 358.3 \cdot 10^4 \, \text{mm}^4 \\
I_{x,\text{eff}} &= 358.3 \cdot 10^4 - 228.1 \cdot 10^4 = 193.0 \cdot 10^4 \, \text{mm}^4
\end{align*}
\]

The section properties of the reduced cross-section per meter with are:

\[
\begin{align*}
A_{pe} &= 228.1 \cdot \frac{1000}{(B_d/2)} = 1755 \, \text{mm}^2/\text{m} \\
I_{\text{eff}} &= 193.0 \cdot 10^4 \cdot \frac{1000}{(B_d/2)} = 1485 \cdot 10^4 \, \text{mm}^4/\text{m}
\end{align*}
\]

Conclusion

Two symmetric flange stiffeners are designed in design with small ribs. In table B.9 a comparison is made of the cross-section properties for the assumed effective steel area and the calculated effective area. The amount of effective steel is improved, because of the two symmetric flange stiffeners. However, the bending stiffness of the deck is still insufficient to limit the deflections of the deck during construction (\( \delta_0 \)).
### Effective Cross-section

<table>
<thead>
<tr>
<th>assumption</th>
<th>'effective width method'</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_p$ (mm²)</td>
<td>3664 3664</td>
</tr>
<tr>
<td>$A_{pe}$ (mm²)</td>
<td>2492 1755</td>
</tr>
<tr>
<td>$I_{eff}$ (cm⁴)</td>
<td>2027 1485</td>
</tr>
<tr>
<td>$\delta_0$ (mm)</td>
<td>40.08 58.06</td>
</tr>
</tbody>
</table>

Design from section B.2 as described in table B.5

Table B.9: Effective section properties of design with small ribs if only two symmetric flange stiffeners are designed

![Figure B.13: Effective area of the cross-section](image)

#### Flange and Web Stiffeners

**Two symmetrical flange stiffeners**

The reduction factor $\kappa_d$, to determine the reduced effective area of the flange stiffener, is effected by the presence of a web or multiple web stiffeners.

\[
\hat{\lambda}_d = \sqrt{\frac{f_{yb}}{\sigma_{cr,mod}}} \\
\beta_s = \frac{\sigma_{cr,s}}{\sqrt{1 + \beta_s^4 \frac{\sigma_{cr,s}}{\sigma_{cr,sa}}}} \\
\beta_s = \begin{cases} 
1, & \text{for a profile in axial compression} \\
1 - (h_a + 0.5 h_{ha})/e_c, & \text{for a profile in bending}
\end{cases}
\]

Where $\sigma_{cr,sa}$ is the elastic critical stress for a single web stiffener or the stiffener closer to the compression flange in webs with two stiffeners.

**Webs with up to two intermediate stiffeners**

The effective cross-section of the web:

- $s_{eff,1}$ strip adjacent to the compression flange;
- $A_{s,red}$ reduced effective area of (each) web stiffener;
- $s_{eff,n}$ strip adjacent to centroidal axis;
- $e_t$ part of the web in tension.

Effective area of stiffener:

\[
A_{s} = t_0 (s_{eff,2} + s_{eff,3} + s_{la}), \text{ for a single stiffener or the stiffener closed to the compression flange;}
\]

\[
A_{sb} = t_0 (s_{eff,4} + s_{eff,5} + s_{lb}), \text{ for a second stiffener.}
\]

Dimensions are shown in figure B.14.

The position of the neutral axis is initially determined with the effective cross-section of the flanges and the gross-section of the webs.
When only designing one web stiffener near the compression flange.

\[ h_a = 61 \text{ mm}; \]
\[ h_{sa} = 10 \text{ mm}; \]
\[ s_{sa} = \sqrt{8^2 + 8^2} = 14.14 \text{ mm}. \]

The basic effective width is:

\[
 s_{eff,0} = 0.76 \cdot \sqrt{E/(\gamma_M \sigma_{com,Ed})} = 0.76 \cdot 1.46 \sqrt{210000/(1.0 \cdot 350)} = 27.18 \text{ mm} \quad (B.8)
\]

Where \( \sigma_{com,Ed} \) is the stress in the compression flange when the cross-section resistance is reached: \( \sigma_{com,Ed} = f_yb/\gamma_M = 350 \text{ MPa} \). If the web is not fully effective \( (s_{eff,0} < s_w) \):

\[
 s_{eff,1} = s_{eff,0} = 27.18 \text{ mm}
\]
\[
 s_{eff,2} = (1 + 0.5h_a/e_c)s_{eff,0} = (1 + 0.5 \cdot 61/1106)27.18 = 35.00 \text{ mm}
\]
\[
 s_{eff,3} = [1 + 0.5(h_a + h_{sa})/e_c]s_{eff,0} = [1 + 0.5(61 + 10)/106]27.18 = 36.29 \text{ mm}
\]
\[
 s_{eff,n} = 1.5s_{eff,0} = 1.5 \cdot 27.18 = 40.77 \text{ mm}
\]

**Elastic critical buckling stress**

\[
 \sigma_{cr,sa} = \frac{1.05 k_f E \sqrt{I_s / t^3 s_1}}{A_{sa} s_2 (s_1 - s_2)}
\]

\[
 A_{sa} = t(s_{eff,2} + s_{eff,3} + s_{sa}) = 1.46(35.00 + 36.29 + 14.14) = 124.73 \text{ mm}
\]

\[
 s_1 = \begin{cases} 
 0.9 (s_a + s_{sa} + s_c), & \text{for a single stiffener} \\
 s_a + s_{sa} + s_b + 0.5 (s_{sb} + s_c), & \text{for a stiffener closer to the compression flange if two stiffeners}
\end{cases}
\]

\[
 s_2 = s_1 - s_a - 0.5s_{sa}
\]

From figure **B.14** follows:

\[
 s_a = \frac{h_a}{\sin \varphi} = \frac{61}{\sin(80)} = 61.94 \text{ mm}
\]
\[
 s_c = \frac{(h_p - h_a - h_{sa})}{\sin \varphi} = \frac{(210 - 61 - 10)}{\sin(80)} = 141.14 \text{ mm}
\]
\[
 s_1 = 0.9 (s_a + s_{sa} + s_c) = 0.9 (61.94 + 14.14 + 141.14) = 195.50 \text{ mm}
\]
\[
 s_2 = s_1 - s_a - 0.5s_{sa} = 195.50 - 61.94 - 0.5 \cdot 19.44 = 126.49 \text{ mm}
\]
Coefficient \( k_f \) is taken equal to 1.0 corresponding to a pin-joined condition.

The second moment of area \( I_s \) of the stiffener with the fold \( s_{sa} \) is determined about its own centroidal axis parallel to the web where two adjacent strips of \( s_{eff,1} \) are taken into account.

\[
I_s = 1418.7 \text{ mm}^4
\]

The elastic critical stress for a single web stiffener:

\[
\sigma_{cr,sa} = \frac{1.05 \cdot 1.0 \cdot 210000 \cdot \sqrt{1418.7 \cdot 1.46^4 \cdot 195.50}}{124.73 \cdot 126.49 \cdot (195.50 - 126.49)} = 188.15 \text{ N/mm}^2
\]

The reduction factor \( \kappa_d \)

Where \( \kappa_d \) should be determined with the modified elastic critical stress \( \sigma_{cr,mod} \), because the flanges are also stiffened. NEN-EN 1993-1-3 5.5.3.4.4 gives the interaction between the distorsional buckling of the flange and web stiffeners.

\[
\sigma_{cr,mod} = \sqrt{\frac{\sigma_{cr,s}}{1 + \beta_s \left( \frac{\sigma_{cr,s}}{\sigma_{cr,sa}} \right)^4}} = \sqrt{\frac{160.46}{1 + 0.377 \left( \frac{160.46}{160.04} \right)^4}} = 160.04 \text{ N/mm}^2
\]

Where:

\[
\sigma_{cr,s} = 160.46 \text{ MPa}, \text{ the elastic critical stress for an intermediate flange stiffener};
\]

\[
\beta_s = 1 - \frac{(h_a + 0.5h_{sa})}{e} = 1 - \frac{(61 + 0.5 \cdot 10)}{106} = 0.377, \text{ for a profile in bending}.
\]

\( \kappa_d \) is determined with expressions B.7, where \( \sigma_{cr,s} \) is substituted by \( \sigma_{cr,mod} \).

\[
\bar{\lambda}_d = \sqrt{\frac{f_{yb}}{\sigma_{cr,mod}}} = \sqrt{\frac{350}{160.04}} = 1.479
\]

\[
\Rightarrow \kappa_d = \frac{0.66}{1.479} = 0.446
\]

The reduced effective area of the flange and web stiffener

The reduced effective area \( A_{sa,red} \) and reduced thickness \( t_{sa,red} \) for a single web stiffener in compression is:

\[
A_{sa,red} = \frac{\kappa_d A_{sa}}{1 - (h_a + 0.5h_{sa})/e} = \frac{0.446 \cdot 124.73}{1 - (61 + 0.5 \cdot 10)/106} = 124.73 \text{ mm}^2 \leq A_{sa}
\]

\[
t_{sa,red} = t \cdot \frac{A_{sa,red}}{A_{sa}} = 1.46 \cdot \frac{124.73}{84.61} = 1.46 \text{ mm}
\]

The reduced effective area \( A_{s,red} \) and reduced thickness \( t_{s,red} \) of the flange stiffener:

\[
A_{s,red} = \kappa_d A_s = 0.446 \cdot 84.61 = 37.76 \text{ mm}^2
\]

\[
t_{s,red} = t \cdot \frac{A_{s,red}}{A_s} = 1.46 \cdot \frac{37.76}{84.61} = 0.65 \text{ mm}
\]

Reduced effective cross-section properties

The reduced cross-section properties are determined with the same method used in the previous sections. The reduced effective thickness of the flange and web stiffener and the increased effective widths are taken
into account.

\[ A_{eff} = 366.4 \text{ mm}^2 \]
\[ S_{s,eff} = 40003.2 \text{ mm}^3 \]
\[ y_{gc,2} = \frac{S_{s,eff}}{A_{eff}} = \frac{40003.2}{366.4} = 109.2 \text{ mm} \]
\[ e_{c,2} = h_p - y_{gc,2} = 210 - 109.2 = 100.8 \text{ mm} \]
\[ e_{t,2} = 109.2 \text{ mm} \]
\[ I_{s,0,e} = 672.2 \cdot 10^4 \text{ mm}^4 \]
\[ I_{s,eff} = 672.2 \cdot 10^4 - 366.4 \cdot 109.2^2 = 235.5 \cdot 10^4 \text{ mm}^4 \]

Due to the web stiffener the width of a single deck panel is increased.

\[ b_d = 2(0.5(b_0 + b_u) + \frac{h_p - h_{ia}}{\tan(a)} + \frac{h_{ia}}{\tan(45)}) = 2(0.5(145.94 + 40) + \frac{210 - 10}{\tan(80)} + \frac{10}{\tan(45)}) = 276.47 \text{ mm} \]

The section properties of the reduced cross-section per meter with are:

\[ A_{pe} = 366.4 \cdot \frac{1000}{(b_d/2)} = 2650.7 \text{ mm}^2 / \text{m} \]
\[ I_{eff} = 235.5 \cdot 10^4 \cdot \frac{1000}{(b_d/2)} = 1703.7 \cdot 10^4 \text{ mm}^4 / \text{m} \]

**Conclusion**

In the design with small ribs is designed for two symmetric flange stiffeners and a single web stiffener. In table B.10 a comparison is made of the cross-section properties of the assumed effective steel area and the calculated effective area. The deck width of a single panel \(b_d\) is slightly increased due to the web stiffeners, and therefore the amount of steel \(A_p\) is slightly decreased. Compared to the previous design the amount of effective steel is improved. The deck design with top flange and web stiffeners provides insufficient bending stiffness to limit the deflections of the deck during construction \((\delta_0 \geq L/180)\).

<table>
<thead>
<tr>
<th>(b_d) (mm)</th>
<th>(e_c) (mm)</th>
<th>(e_{t}) (mm)</th>
<th>(A_p) (mm(^2))</th>
<th>(A_{pe}) (mm(^2))</th>
<th>(I_{eff}) (cm(^4))</th>
<th>(\delta_0) (mm)</th>
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</thead>
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<tr>
<td>260</td>
<td>95.5</td>
<td>114.5</td>
<td>3664</td>
<td>2492</td>
<td>2027</td>
<td>40.08</td>
</tr>
<tr>
<td>276.5</td>
<td>100.8</td>
<td>109.2</td>
<td>3524</td>
<td>2608</td>
<td>1681</td>
<td>50.6</td>
</tr>
</tbody>
</table>

Table B.10: Effective steel area of design with small ribs with two symmetric flange stiffener and a single web stiffener

**B.3.4. DIFFERENT DESIGNS**

The 'effective width method' is performed for the two design concepts for three situations: no stiffeners, flange stiffeners, and flange and web stiffeners. In tables B.11 and B.12 are the results presented. The total amount of steel \(A_p\), total amount of effective steel \(A_{pe}\), the effective second moment of area \(I_{eff}\) and the deflection during construction \(\delta_0\) are given.

Note that the amount of effective steel for both designs is decreasing if web stiffeners are applied. This, due to the increase in panel width \(b_d\) as discussed above.
B.4. Redesign of the Decks

Table B.13 gives an overview of a variant study of the two different designs. In this case are the trapezoidal deck shape, stiffeners and embossments included. The decks are verified during construction and the composite slab during service life according to the calculation procedures given in section B.1 and B.3. The different parameters, sheet thickness $t$, deck height $h_p$, and panel width $b_d$ are varied. The different variants presented in this table are evaluated according to verification prescribe by the Eurocode and the design specification summarised in table 3.1.

Effective width of design concept 1: small ribs

<table>
<thead>
<tr>
<th>Assumption</th>
<th>No stiffeners</th>
<th>Flange stiffeners</th>
<th>Flange and web stiffeners</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_p$ (mm$^2$)</td>
<td>3664</td>
<td>3664</td>
<td>3664</td>
</tr>
<tr>
<td>$A_{pe}$ (mm$^2$)</td>
<td>2492</td>
<td>1657</td>
<td>1755</td>
</tr>
<tr>
<td>$I_{eff}$ (cm$^4$)</td>
<td>2027</td>
<td>1327</td>
<td>1485</td>
</tr>
<tr>
<td>$\delta_0$ (mm)</td>
<td>40.1</td>
<td>66.8</td>
<td>58.1</td>
</tr>
</tbody>
</table>

Design from section B.2 as described in table B.6

Table B.11: Effective steel area of design concept 1

Effective width of design concept 2: wide ribs with fill element

<table>
<thead>
<tr>
<th>Assumption</th>
<th>No stiffeners</th>
<th>Flange stiffeners</th>
<th>Flange and web stiffeners</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_p$ (mm$^2$)</td>
<td>3683</td>
<td>3683</td>
<td>3821</td>
</tr>
<tr>
<td>$A_{pe}$ (mm$^2$)</td>
<td>2513</td>
<td>1672</td>
<td>1666</td>
</tr>
<tr>
<td>$I_{eff}$ (cm$^4$)</td>
<td>2246</td>
<td>1527</td>
<td>1502</td>
</tr>
<tr>
<td>$\delta_0$ (mm)</td>
<td>38.4</td>
<td>60.8</td>
<td>62.1</td>
</tr>
</tbody>
</table>

Design from section B.2 as described in table B.7

Table B.12: Effective steel area of design concept 2

Figure B.15: Effective part of the cross-section
### Deck dimensions

<table>
<thead>
<tr>
<th></th>
<th>Small ribs</th>
<th>Wide ribs</th>
<th>CF210</th>
<th>CF225</th>
</tr>
</thead>
<tbody>
<tr>
<td>( t ) (mm)</td>
<td>1</td>
<td>1.25</td>
<td>1.5</td>
<td>1.25</td>
</tr>
<tr>
<td>( t_0 ) (mm)</td>
<td>0.96</td>
<td>1.21</td>
<td>1.46</td>
<td>0.96</td>
</tr>
<tr>
<td>( h_p ) (mm)</td>
<td>330</td>
<td>250</td>
<td>220</td>
<td>260</td>
</tr>
<tr>
<td>( h_c ) (mm)</td>
<td>266</td>
<td>216</td>
<td>216</td>
<td>256</td>
</tr>
<tr>
<td>( \alpha ) (°)</td>
<td>80</td>
<td>80</td>
<td>80</td>
<td>78</td>
</tr>
<tr>
<td>( b_0 ) (mm)</td>
<td>94</td>
<td>72</td>
<td>82</td>
<td>60</td>
</tr>
<tr>
<td>( b_a ) (mm)</td>
<td>40</td>
<td>40</td>
<td>40</td>
<td>88</td>
</tr>
<tr>
<td>( b_c ) (mm)</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>23</td>
</tr>
</tbody>
</table>

### Deck properties

<table>
<thead>
<tr>
<th></th>
<th>Small ribs</th>
<th>Wide ribs</th>
<th>CF210</th>
<th>CF225</th>
</tr>
</thead>
<tbody>
<tr>
<td>( A_p ) (mm²/m)</td>
<td>3093</td>
<td>3769</td>
<td>4209</td>
<td>2876</td>
</tr>
<tr>
<td>( A_{pe} ) (mm²/m)</td>
<td>1297</td>
<td>2430</td>
<td>3286</td>
<td>1576</td>
</tr>
<tr>
<td>( I_{eff} ) (cm⁴/m)</td>
<td>2798</td>
<td>2525</td>
<td>2350</td>
<td>1986</td>
</tr>
<tr>
<td>( M_{el,Rd} ) (kNm/m)</td>
<td>53.7</td>
<td>70.6</td>
<td>73.0</td>
<td>52.9</td>
</tr>
</tbody>
</table>

### Floor properties

<table>
<thead>
<tr>
<th></th>
<th>Small ribs</th>
<th>Wide ribs</th>
<th>CF210</th>
<th>CF225</th>
</tr>
</thead>
<tbody>
<tr>
<td>( h ) (mm)</td>
<td>390</td>
<td>310</td>
<td>280</td>
<td>320</td>
</tr>
<tr>
<td>( L ) (m)</td>
<td>7.2</td>
<td>7.2</td>
<td>7.2</td>
<td>7.2</td>
</tr>
<tr>
<td>( G ) (kg/m²)</td>
<td>561</td>
<td>506</td>
<td>465</td>
<td>389</td>
</tr>
<tr>
<td>( G_p ) (kg/m²)</td>
<td>23.8</td>
<td>29.4</td>
<td>32.8</td>
<td>22.4</td>
</tr>
<tr>
<td>( G_s ) (kg/m²)</td>
<td>6.2</td>
<td>6.8</td>
<td>6.8</td>
<td>13.5</td>
</tr>
<tr>
<td>( V_c ) (L/m²)</td>
<td>192</td>
<td>167</td>
<td>148</td>
<td>118</td>
</tr>
<tr>
<td>( G_{deck} ) (kg/unit)</td>
<td>24</td>
<td>46</td>
<td>51</td>
<td>41</td>
</tr>
<tr>
<td>( G_{ponding} ) (kg/m²)</td>
<td>71</td>
<td>70</td>
<td>69</td>
<td>71</td>
</tr>
</tbody>
</table>

### Criteria

**During construction**

<table>
<thead>
<tr>
<th></th>
<th>SLS:</th>
<th>ULS:</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \delta_0 &lt;1 )</td>
<td>0.99</td>
<td>1.00</td>
</tr>
<tr>
<td>( M^+ &lt;1 )</td>
<td>1.19</td>
<td>0.82</td>
</tr>
<tr>
<td>( V &lt;1 )</td>
<td>4.23</td>
<td>0.90</td>
</tr>
</tbody>
</table>

**During service life**

<table>
<thead>
<tr>
<th></th>
<th>SLS:</th>
<th>ULS:</th>
</tr>
</thead>
<tbody>
<tr>
<td>( w_3 &lt;1 )</td>
<td>0.34</td>
<td>0.49</td>
</tr>
<tr>
<td>( w_{tot} &lt;1 )</td>
<td>0.40</td>
<td>0.55</td>
</tr>
<tr>
<td>( f_r &lt;1 )</td>
<td>0.75</td>
<td>0.87</td>
</tr>
<tr>
<td>( A_s &lt;1 )</td>
<td>0.95</td>
<td>0.95</td>
</tr>
</tbody>
</table>

**Notes:**

1. Self-weight does not include the ponding effect
2. Unity check per failure criteria

Table B.13: Variant study for the two designs compared with existing deep decks
Installation with a Double Cast

In this appendix the effect of a double cast on the deflection of a deck with small ribs and a steel thickness of 1.25 mm is studied. Two situations are considered: first cast up to centroid of the deck and first cast up to top of the deck.

**Trial section**

A deck profile from the previous section with small ribs is reevaluated. The weight of a single deck panel is 42 kg. Other properties of the deck are:

<table>
<thead>
<tr>
<th>Deck</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of profile</td>
<td>$h_p$ : 220 mm</td>
</tr>
<tr>
<td>Width of deck panel (rib distance c.t.c.)</td>
<td>$b_{ld}$ : 218 mm</td>
</tr>
<tr>
<td>Width bottom flange</td>
<td>$b_u$ : 50 mm</td>
</tr>
<tr>
<td>Width top flange</td>
<td>$b_0$ : 112 mm</td>
</tr>
<tr>
<td>Angle of the web</td>
<td>$\alpha$ : 80 °</td>
</tr>
<tr>
<td>Thickness of the profile</td>
<td>$t$ : 1.25 mm</td>
</tr>
<tr>
<td>Effective thickness of the profile</td>
<td>$t_0$ : 1.21 mm</td>
</tr>
<tr>
<td>Distance from centroid of the profile to the bottom</td>
<td>$d_p$ : 105 mm</td>
</tr>
<tr>
<td>Cross-section area of the profile</td>
<td>$A_p$ : 3397 mm$^2$/m</td>
</tr>
<tr>
<td>Effective cross-section area of the profile</td>
<td>$A_{pe}$ : 2141 mm$^2$/m</td>
</tr>
<tr>
<td>Second moment of area of the profile</td>
<td>$I_p$ : 2027 cm$^4$/m</td>
</tr>
</tbody>
</table>

**Tension reinforcement**

| Diameter of tension reinforcement | $\varnothing$ : 16 mm |
| Area of tension reinforcement    | $A_s$ : 925 mm$^2$/m |
| Second moment of area of tension reinforcement | $I_s$ : 14798 mm$^4$/m |

**Concrete**

| Cross-section area of the concrete | $A_c$ : 139000 mm$^2$/m |

The amount of ribs per meter width:

$$n_p = \frac{1000}{b_{ld}} = \frac{1000}{218} = 4.6 \text{ ribs/m}$$

**C.1. First cast up to centroid of deck**

The first cast is up to the height of the centroid of the deck: $h_f = 105$ mm.

Average concrete width per rib

$$b_{om} : 59 \text{ mm}$$
**Deflection of the Deck**

The amount of concrete during the first cast is:

\[ A_c = b_{om} h_f n_p = 59 \cdot 105 \cdot 4.6 = 28497 \text{ mm}^2 \]

**Loads during first cast**

- Weight of the concrete \( (\rho_{c,wet} = 25 \text{ kN/m}^3) \)
  \[ G_c : 0.71 \text{ kN/m}^2 \]
- Weight of the deck \( G_p : 0.29 \text{ kN/m}^2 \)
- Weight of the tension reinforcement \( G_s : 0.07 \text{ kN/m}^2 \)
- Weight during the first cast (construction loads not incl.) \( g_k : 1.07 \text{ kN/m}^2 \)

The construction loads are not taken into account during this first cast, because the deflections during the second cast will be governing.

Deflection of the deck:

\[ \delta_1 = \frac{5}{384} \frac{g_k L^4}{E_s I_{eff}} = \frac{5}{384} \frac{1.07 \cdot 7200^4}{2.1 \cdot 10^5 \cdot 2027 \cdot 10^4} = 8.8 \text{ mm} \]

The effect of ponding is taken into account with the method described in section A.4. Note that the ponding will only take place in the ribs.

\[ \text{Area} = \frac{1}{120} \frac{g_k L^5}{E_s I_{eff}} = \frac{1}{120} \frac{1.07 \cdot 7200^5}{2.1 \cdot 10^5 \cdot 2027 \cdot 10^4} = 40531 \text{ mm}^2 \]

\[ h_{ponding} = \frac{\text{Area}}{L} = \frac{40531}{7200} = 5.6 \text{ mm} \]

\[ q_{ponding} = \frac{(h_{ponding} b_{om} n_p)}{1000^2 \cdot \rho_{wet}} = (5.6 \cdot 59 \cdot 4.6)/1000^2 \cdot 25 = 0.04 \text{ kN/m}^2 \]

\[ \delta_1 = \frac{5}{384} (\frac{g_k + q_{ponding}}{E_s I_{eff}})^4 = \frac{5}{384} \frac{(1.07 + 0.04) \cdot 7200^4}{2.1 \cdot 10^5 \cdot 2027 \cdot 10^4} = 9.1 \text{ mm} \]

**Bending Stiffness**

The second moment of area of the steel deck with concrete ribs is determined. Some assumptions are made:

- short-term loading: \( E_{cm} = 30000 \text{ N/mm}^2 \) for C20/25 (no creep and shrinkage);
- contribution of the concrete calculated with factor: \( n = \frac{E_s}{E_{cm}} = \frac{21000}{30000} = 7; \)
- the cross-section is cracked and the concrete in tension is neglected.

The height of the compression zone \( x_c \) assumed that all concrete is in tension (from the bottom fiber):

\[ x_{c,0} = \frac{A_{pe} d_p + A_s \frac{3}{2} \delta}{A_{pe} + A_s} = \frac{2519 \cdot 105 + 925 \cdot \frac{3}{2} \cdot 16}{2519 + 925} = 83.5 \text{ mm} \]

The concrete is cast up to 105 mm and thus is a part in compression. Therefore the height of the compression zone is iteratively calculated:

\[ x_{c,n} = \frac{b_{om}(h_f - x_{c,n-1})(h_f - \frac{1}{2}(h_f - x_{c,n-1})) n_p + n(A_{pe} d_p + A_s \frac{3}{2} \delta)}{b_{om}(h_f - x_{c,n-1}) n_p + n(A_{pe} + A_s)} \]

\[ = \frac{59(105 - x_{c,n-1})(105 - \frac{1}{2}(105 - x_{c,n-1}))4.6 + 7(2519 \cdot 105 + 925 \cdot \frac{3}{2} \cdot 16)}{59(105 - x_{c,n-1})4.6 + 7(2519 + 925)} \]

Where \( n = 0 \). For 10 iterations the compression height is: \( n_{c,10} = 85.4 \text{ mm} \).
The second moment of area of the cracked cross-section:

\[
I_{cc} = \frac{b_{om}(h_f - x_c)^3}{12n}n_p + \frac{b_{om}(h_f - x_c)}{n}(d_p - x_c)\left(\frac{d_p - x_c}{2}\right)^2n_p + I_{eff} + A_{pe}(d_p - x_c)^2 + I_s + A_s\left(x_c - \frac{3}{2}\right)^2
\]

\[
= \frac{59(105 - 85.4)^3}{12 \cdot 7} \cdot 4.6 + \frac{59(105 - 85.4)}{7}\left(\frac{105 - 85.4}{2}\right)^2 4.6 + 2027 \cdot 10^4 + 2519(105 - 85.4)^2 + 14798 + 925\left(85.4 - \frac{3}{2}\right)^2 = 2483.7 \cdot 10^4 \text{ mm}^4 / \text{m}
\]

**Deflection of the partially steel-concrete composite slab**

The amount of concrete during the second cast is:

\[
A_c = 139000 - 28497 = 110503 \text{ mm}^2
\]

**Loads during first cast**

- Self-weight of the slab (\(\rho_c = 23.5 \text{ kN/m}^3\))
  \(G_c : 1.07 \text{ kN/m}^2\)
- Weight of the reinforced wet concrete (\(\rho_{c,wet} = 26 \text{ kN/m}^3\))
  \(G_c : 2.87 \text{ kN/m}^2\)
- Construction loads
  \(G_p : 0.75 \text{ kN/m}^2\)
- Weight during the second cast
  \(g_k : 4.86 \text{ kN/m}^2\)

Deflection of the deck:

\[
\delta_1 = \frac{5 \cdot g_k L^4}{384 E_s I_{c,c}} = \frac{5 \cdot 4.86 \cdot 7200^4}{384 \cdot 2.1 \cdot 10^5 \cdot 2483.7 \cdot 10^4} = 32.6 \text{ mm}
\]

The effect of ponding is taken into account with the method described in section A.4.

\[
\text{Area} = \frac{1}{120 E_s I_{c,c}} = \frac{1}{120 \cdot 2.1 \cdot 10^5 \cdot 2483.7 \cdot 10^4} = 150245 \text{ mm}^2
\]

\[
h_{\text{ponding}} = \frac{\text{Area}}{L} = \frac{150245}{7200} = 20.9 \text{ mm}
\]

\[
q_{\text{ponding}} = \frac{h_{\text{ponding}} \cdot b}{1000^2 \cdot \rho_{\text{wet}}} = \frac{(20.9 \cdot 1000)}{1000^2 \cdot 25} = 0.52 \text{ kN/m}^2
\]

\[
\delta_1 = \frac{5 \cdot (g_k + q_{\text{ponding}}) L^4}{384 E_s I_{c,c}} = \frac{5 \cdot (4.86 + 0.52) \cdot 7200^4}{384 \cdot 2.1 \cdot 10^5 \cdot 2483.7 \cdot 10^4} = 36.1 \text{ mm}
\]

The total deflection during construction (incl. ponding):

\[
\delta_0 = \delta_1 + \delta_2 = 9.1 + 36.1 = 45.2 \text{ mm}
\]

**C.2. First cast up to height of deck**

The first cast is up to the height of the top of the deck: \(h_f = h_p = 220 \text{ mm}\).

- Average concrete width per rib
  \(b_{om} : 78 \text{ mm}\)

**Deflection of the deck**

The amount of concrete during the first cast is:

\[
A_c = b_{om} h_p n_p = 78 \cdot 220 \cdot 4.6 = 78936 \text{ mm}^2
\]

**Loads during first cast**

- Weight of the concrete (\(\rho_{c,wet} = 25 \text{ kN/m}^3\))
  \(G_c : 1.97 \text{ kN/m}^2\)
- Weight of the deck
  \(G_p : 0.29 \text{ kN/m}^2\)
- Weight of the tension reinforcement
  \(G_s : 0.07 \text{ kN/m}^2\)
- Weight during the first cast (construction loads not incl.)
  \(g_k : 2.32 \text{ kN/m}^2\)
Deflection of the deck:
\[
\delta_1 = \frac{5}{384} \frac{g_k L^4}{E_s I_{eff}} = \frac{5}{384} \frac{2.32 \cdot 7200^4}{2.1 \cdot 10^5 \cdot 2027 \cdot 10^4} = 19.1 \text{ mm}
\]

The effect of ponding is taken into account with the method described in section A.4.
\[\text{Area}_{\text{ponding}} = \frac{1}{120} \frac{g_k L^5}{E_s I_{eff}} = \frac{1}{120} \frac{2.32 \cdot 7200^5}{2.1 \cdot 10^5 \cdot 2027 \cdot 10^4} = 87881 \text{ mm}^2\]
\[\text{h}_{\text{ponding}} = \frac{\text{Area}_{\text{ponding}}}{L} = \frac{87881}{7200} = 12.2 \text{ mm}\]
\[\text{q}_{\text{ponding}} = \frac{(\text{h}_{\text{ponding}} - \text{h}_c) / \text{h}_c}{1000^2 \cdot \rho_{\text{wet}}} = (12.2 \cdot 1000 / 1000^2 \cdot 25 = 0.31 \text{kN/m}^2\]
\[\delta_1 = \frac{5}{384} \frac{(g_k + q_{\text{ponding}}) L^4}{E_s I_{eff}} = \frac{5}{384} \frac{(2.32 + 0.31) \cdot 7200^4}{2.1 \cdot 10^5 \cdot 2027 \cdot 10^4} = 21.6 \text{ mm}\]

**Bending stiffness**

The second moment of area of the partially steel-concrete composite slab is determined.

The concrete is cast up to 220 mm and thus is a part in compression. Therefore the height of the compression zone \(x_c\) is iteratively calculated:
\[x_{c,n} = \frac{b_{om}(h_p - x_{c,n-1})(h_p - \frac{1}{2}(h_p - x_{c,n-1}))n_p + n(A_{pe}d_p + A_s \frac{3}{2} \rho)}{b_{om}(h_p - x_{c,n-1})n_p + n(A_{pe} + A_s)}
= \frac{78(220 - x_{c,n-1})(220 + \frac{1}{2}(220 - x_{c,n-1}))4.6 + 7(2519 \cdot 105 + 925 \cdot \frac{3}{2} \cdot 16)}{78(220 - x_{c,n-1})4.6 + 7(2519 + 925)}\]

Where \(n = 0\). For a number of iterations the compression height is: \(x_{c,n=\infty} = 161 \text{ mm}\). The height of the concrete in compression is: \(h_c = h_p - x_c = 220 - 161 = 59 \text{ mm}\).

The second moment of area of the cracked cross-section:
\[I_{c,e} = \frac{b_{om}h_c^3}{12n} n_p + \frac{b_{om}h_c}{n} \left( \frac{h_c}{2} \right)^2 n_p + I_{eff} + A_{pe}(x_c - d_p)^2 + I_s + A_s \left( x_c - \frac{3}{2} \right)^2\]
\[= \frac{78 \cdot 59^3}{12 \cdot 7} \cdot 4.6 + \frac{78 \cdot 59}{7} \cdot \left( \frac{59}{2} \right)^2 4.6 + 2027 \cdot 10^4\]
\[+ 2519(161 - 105)^2 + 14798 + 925(161 - \frac{3}{16})^2 = 4905.5 \cdot 10^4 \text{ mm}^4 / \text{m}\]

**Deflection of the partially steel-concrete composite slab**

The amount of concrete during the second cast is:
\[A_c = 60 \cdot 1000 = 60000 \text{ mm}^2\]

**Loads during first cast**
- Self-weight of the slab (\(\rho_c = 23.5 \text{ kN/m}^3\))
- Weight of the reinforced wet concrete (\(\rho_{c,wet} = 26 \text{ kN/m}^3\))
- Construction loads
- Weight during the second cast

Deflection of the deck:
\[\delta_1 = \frac{5}{384} \frac{g_k L^4}{E_l I_{c,e}} = \frac{5}{384} \frac{4.81 \cdot 7200^4}{2.1 \cdot 10^5 \cdot 4905.5 \cdot 10^4} = 16.3 \text{ mm}\]
The effect of ponding is taken into account with the method described in section A.4.

\[ \text{Area} = \frac{1}{120} \frac{g_k L^5}{E_s I_{c,c}} = \frac{1}{120} \frac{4.81 \cdot 7200^5}{2.1 \cdot 10^5 \cdot 4905.5 \cdot 10^4} = 75288 \text{ mm}^2 \]

\[ h_{\text{ponding}} = \frac{\text{Area}}{L} = \frac{75288}{7200} = 10.5 \text{ mm} \]

\[ q_{\text{ponding}} = \frac{(h_{\text{ponding}} \cdot b)}{1000^2} \cdot \rho_{\text{wet}} = \frac{(10.5 \cdot 1000)}{1000^2} \cdot 25 = 0.26 \text{kN/m}^2 \]

\[ \delta_1 = \frac{5}{384} \frac{(g_k + q_{\text{ponding}}) L^4}{E_s I_{c,c}} = \frac{5}{384} \frac{(4.81 + 0.26) \cdot 7200^4}{2.1 \cdot 10^5 \cdot 4905.5 \cdot 10^4} = 17.2 \text{ mm} \]

The total deflection during construction (incl. ponding):

\[ \delta_0 = \delta_1 + \delta_2 = 21.6 + 17.2 = 38.8 \text{ mm} \]
D.1. CROSS-SECTIONAL PROPERTIES OF THE WEB STIFFENERS

In this section the cross-sectional properties of the web stiffeners are calculated. The stiffeners have a width of 17 mm, are designed to be 3 mm in depth, and have a c.t.c. distance of 220 mm. The width of the unembossed part is therefore 203 mm. The deck is 1.21 mm thick.

D.1.1. THE GROSS CROSS-SECTION

The gross cross-sectional properties are:

\[ A_s = 1.21(203 + 17 + 2 \cdot 3) = 273.4 \text{ mm}^2 \]

\[ y_{gc} = \frac{1.21 \cdot 17 \cdot 3 + 2 \cdot 1.21 \cdot 3 \cdot 1.5}{273.4} = 0.27 \text{ mm} \]

\[ I_s = \frac{1}{12} 203 \cdot 1.21^3 + 203 \cdot 1.21 \cdot 0.27^2 + \frac{1}{12} 17 \cdot 1.21^3 + 17 \cdot 1.21 \cdot (3 - 0.27)^2 + 2 \cdot \frac{1}{12} 1.21 \cdot 3^3 + 2 \cdot 1.21 \cdot 3 \cdot (1.5 - 0.27)^2 = 220.1 \text{ mm}^4 \]

Here is \( A_s \) the area and \( I_s \) the second moment of area of the stiffener.

D.1.2. THE EFFECTIVE CROSS-SECTION

The effective area of a stiffener is obtained with Eurocode 3 part 3 and 5. Firstly, the spring stiffness \( K \) of the equivalent system is determined (figure D.1) [5, p.26-27]:

\[ \delta = \frac{u b_1^2 b_2^2}{3(b_1 + b_2)} \cdot \frac{12(1 - v^2)}{E l^3} = \frac{1 \cdot 110^2 110^2}{3(110 + 110)} \cdot \frac{12(1 - 0.3^2)}{2.1 \cdot 10^3 \cdot 1.21^3} = 6.511 \text{ mm} \]

Here is \( u = 1 \) the unit load and \( v = 0.3 \) the Poisson's ratio in the elastic range. The width \( b_1 \) and \( b_2 \) are taken equal to half of the c.t.c. distance of the transverse web stiffeners. The spring stiffness:

\[ K = \frac{u}{\delta} = \frac{1}{6.511} = 0.153 \text{ N/mm} \]

The effective section of the transverse stiffener is determined in 4 steps as shown in figure D.2 [5, p.31].
Assumed is $K = \infty$. With this assumption the effective width of an internal plane element under uniform compression is determined ($k_r = 4$ and $\Psi = 1$) [13, p.18-21]:

$$\hat{\lambda}_p = \frac{\hat{b} / t}{28.4 \sqrt{k_r \sigma}} = \frac{203 / 1.21}{28.4 \sqrt{235 / 350}} = 3.605$$

$$\rho = \frac{\hat{\lambda}_p - 0.055(3 + \Psi)}{\hat{\lambda}_p^2} = \frac{3.605 - 0.055(3 + 1)}{3.605^2} = 0.260$$

$$b_{\text{eff}} = \rho \hat{b} = 0.260 \cdot 203 = 52.9 \text{ mm}$$

$$b_{1,e2} = b_{2,e1} = 0.5 \cdot b_{\text{eff}} = 0.5 \cdot 52.9 = 26.4 \text{ mm}$$

The effective area and second moment of area of the stiffener are:

$$A_s = t(b_{1,e2} + b_{2,e1} + b_s) = 1.21(26.4 + 26.4 + 23) = 91.8 \text{ mm}^2$$

$$I_s = 149.5 \text{ mm}^4$$

Where $b_s = 2 \cdot 3 + 17 = 23 \text{ mm}$ is the length of the stiffener. $A_s$ and $I_s$ are obtained in a similar way as for the gross cross-section.

**Step 2**

The elastic critical buckling stress and the relative slenderness are [5, p.32]:

$$\sigma_{cr,s} = \frac{2 \cdot \sqrt{K E I_s}}{A_s} = \frac{2 \cdot \sqrt{0.153 \cdot 2.1 \cdot 10^5 \cdot 149.5}}{91.8} = 47.8 \text{ N/mm}^2$$

$$\hat{\lambda}_d = \sqrt{\frac{f_{yb}}{\sigma_{cr,s}}} = \sqrt{\frac{350}{47.8}} = 2.705$$

The reduction factor for distortional buckling [5, p.27]:

$$\kappa_d = \begin{cases} 
1.0, & \text{if } \hat{\lambda}_d \leq 0.65 \\
1.47 - 0.723 \hat{\lambda}_d, & \text{if } 0.65 < \hat{\lambda}_d < 1.38 \\
0.66 \hat{\lambda}_d, & \text{if } \hat{\lambda}_d \geq 1.38 
\end{cases}$$

$$\kappa_d = \frac{0.66}{2.705} = 0.244$$

**Step 3**

Repeat with $\kappa_d \cdot \frac{f_{yb}}{T_{\text{m0}}} = 0.244 \cdot (350 / 1) = 85.4 \text{ N/mm}^2$ [5, p.32].
Step 1: Effective cross-section for infinite stiff K

Step 2: Elastic critical stress for the effective area of the stiffener from step 1

Step 3: Iteration of step 1 with a reduced stress

Step 4: reduced effective area of the stiffener with $b_{1,2}$ and $b_{2,01}$ and $t_{red}$

Figure D.2: Plane element with an intermediate stiffener: 4 steps to determine the effective area of the stiffener [5, p.31-33]
Repeat with $\lambda_p = 1.605$ — $\rho = 0.537$ — $b_{eff} = 109.1$ mm — $b_{1,e2} = b_{2,e1} = 54.5$ mm — $A_s = 159.8$ mm$^2$ — $I_s = 173.9$ mm$^4$ — $\sigma_{cr,s} = 29.6$ N/mm$^2$ — $\lambda_d = 3.436$ — $\kappa_d = 0.192$

Repeat with $\kappa_d \cdot \frac{f_{yb}}{\gamma M_0} = 0.192 \cdot (350/1) = 67.2$ N/mm$^2$.

$n = 3$: $\lambda_p = 1.579$ — $\rho = 0.544$ — $b_{eff} = 110.6$ mm — $b_{1,e2} = b_{2,e1} = 55.3$ mm — $A_s = 161.6$ mm$^2$ — $I_s = 174.3$ mm$^4$ — $\sigma_{cr,s} = 29.3$ N/mm$^2$ — $\lambda_d = 3.454$ — $\kappa_d = 0.191$

**STEP 4**

Reduced effective area of the stiffener [5, p.32]:

$$A_{s,red} = \kappa_d A_s \frac{f_{yb}/\gamma M_0}{\sigma_{com,Ed,n}} = 0.191 \cdot 161.6 \frac{350/1}{67.2} = 160.8 \text{ mm}^2 \leq A_s$$

$$t_{red} = \frac{A_{s,red}}{A_s} = 1.21 \cdot \frac{160.8}{161.6} = 1.20 \text{ mm}$$

Here is $\sigma_{com,Ed,n}$ the compressive stress at centre line of the stiffener calculated on basis of the effective cross-section with $\kappa_d$ from the previous iteration ($n = 2$): $\sigma_{com,Ed,2} = \kappa_d f_{yb}/\gamma M_0 = 0.192 \cdot 350/1 = 67.2$ N/mm$^2$.

The effective section properties of the stiffener should be determined with $b_{1,e2} = 55.3$ mm, $b_{2,e1} = 55.3$ mm and $t_{red} = 1.20$ mm.

**D.2. DERIVATION OF THE DIFFERENTIAL EQUATION**

The differential equation (DE) follows from the equilibrium of a small part of the bar with a initial deflection. The equilibrium of the internal forces provide a 4th order DE. The derivation is found in the reader of Hartsuijker and Wellman [37, p.132-148]. Figure D.3 illustrates a part $\Delta x$ of the deformed bar including the forces that act on $\Delta x$.

Note that the normal force is constant over the length and that there acts no uniform force on the bar, respectively $q_x = 0$ and $q_z = 0$.

Normal force $N$ and shear force $V$ are replaced by the resultant force $S$. $S$ is dissolved in a vertical- and horizontal component, respectively $S_z$ and $S_x$. In this manner the rotation of $N$ and $V$ over the bar length is included in the force equilibrium (figure D.3).

Force equilibrium:

$$\Sigma F_x = 0: \quad -S_x + (S_z + \Delta S_z) = \Delta S_x = 0$$
$$\Sigma F_z = 0: \quad -S_z + (S_x + \Delta S_x) = \Delta S_z = 0$$
$$\Sigma T_y = 0: \quad -M + S_x \Delta w - S_z \Delta x + (M + \Delta M) = S_x \Delta w - S_z \Delta x + \Delta M = 0$$

Divide by $\Delta x$ gives:

$$\frac{dM}{dx} + S_z \frac{dw}{dx} - S_z = 0, \text{ or in a brief notation: } M' + S_z w' - S_z = 0$$

Differentiate to $x$:

$$M'' + S_z w'' + S_x' w' - S_z' = 0$$

Substitution of $\Delta S_x = S_x' = 0$ and $\Delta S_z = S_z' = 0$ gives:

$$M'' + S_z w'' = 0.$$
Figure D.3: Small part of the bar $\Delta x$ (left) and resultant force $S$ (right)

Is the bending stiffness of the bar $EI$, than holds for the bending moment $M = -EIw''$. Substitution gives:

$$(-EIw''')'' + S_xw'' = 0$$
$$EIw''' - S_xw'' = 0$$

Note that this expression is valid for a prismatic bar where $EI$ is independent of $x$. This 4th order DE in displacement $w$ describes the geometric non-linear behaviour of a prismatic bar. The normal force is a constant compression force over the length of the bar: $S_x = -F$. This gives the buckling equation:

$$EIw''' + \frac{F}{EI}w'' = 0$$  \hspace{1cm} (D.1)

**D.3. MAPLE CALCULATION**

First are the parameters $l_1$, $l_2$, $L$, $EI_1$, $EI_2$, $C_{\theta,1}$, $C_{\theta,2}$, and $K$ defined.

```maple
C1:=0: C2:=0:
```

The solutions for $w_1$, $w_2$, and $w_3$ are given with the relations for $\phi$, $M$, and $S_z$.

```maple
w1:=_C1+_C2*x+_C3*sin(sqrt(F/EI2)*x)+_C4*cos(sqrt(F/EI2)*x);
w2:=_C5+_C6*x+_C7*sin(sqrt(F/EI1)*x)+_C8*cos(sqrt(F/EI1)*x);
w3:=_C9+_C10*x+_C11*sin(sqrt(F/EI2)*x)+_C12*cos(sqrt(F/EI2)*x);
phi1:=diff(-w1,x): M1:=-EI2*diff(w1,x$2); Sz1:=diff(M1,x)-F*diff(w1,x);
phi2:=diff(-w2,x): M2:=-EI1*diff(w2,x$2); Sz2:=diff(M2,x)-F*diff(w2,x);
phi3:=diff(-w3,x): M3:=-EI2*diff(w3,x$2); Sz3:=diff(M3,x)-F*diff(w3,x);
```

The 12 equations are defined.

```maple
x:=(0): eq1:=w1=0: eq2:=M1=-C1*phi1:
x:=(l1): eq3:=w1-w2=0: eq4:=phi1-phi2=0: eq5:=M1-M2=0: eq6:=Sz1+w1*K-Sz2=0:
x:=(l2): eq7:=w2-w3=0: eq8:=phi2-phi3=0: eq9:=M2-M3=0: eq10:=Sz2+w2*K-Sz3=0:
x:=(L): eq11:=w3=0: eq12:=M3=-C2*phi2:
```

This system of equations is written in matrix notation and the determinant is plotted for different values of $F$.

```maple
with(LinearAlgebra): sys:=[eq1,eq2,eq3,eq4,eq5,eq6,eq7,eq8,eq9,eq10,eq11,eq12]: var:=[C1,C2,C3,C4,C5,C6,C7,C8,C9,C10,C11,C12]: (A,b):=GenerateMatrix(sys,var):
q:=Determinant(A): with(plots): P:=plot(q,F=0..10000): display(P);
```

The plot, shown in figure D.4, is the function of the determinant $A$ for $F$. The buckling load $N_{cr}$ is the lowest value of $F$ for which the function is 0. The buckling load $N_{cr}$ is 8323 in this plot.
Figure D.4: Plot of $\det(A)$ for different values of $F$
**Temperature distributions**

<table>
<thead>
<tr>
<th>Staaltemperatuur</th>
<th>$f_{eq}/f_{p}$</th>
<th>$f_{eq}/f_{pm}$</th>
<th>$E_{eq}/E_{p}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\theta$[^1][°C]</td>
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<td>warmgewalst en koudgevormd</td>
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</tr>
<tr>
<td>1100</td>
<td>0,02</td>
<td>0,01</td>
<td>0,02</td>
</tr>
</tbody>
</table>

Figure E.1: Reduction for steel at elevated temperatures (maximum stress $f_{eq}$).

[^1]: $\theta$ represents temperature in °C.
Figure E.2: Temperature distribution in concrete slabs (height \( h = 200 \text{ mm} \)) for R30-R240 [14, p.68]

Figure E.3: Temperature distribution for a concrete beam \( h \times b = 300 \times 160 \) for R60 (left) and R90 (right) [14, p.69-70]
BENDING STIFFNESS DURING FIRE

F.1. $MN\kappa$-DIAGRAM

The bending stiffness of the concrete top flange depends on the occurring moment, normal force and the radius of curvature. Four distinctive points are depicted in a $MN\kappa$-diagram (figure F.1):

- cracking of the concrete in tension $M_r$;
- yielding of the steel $M_y$;
- crushing of the concrete in compression in the outer fibre $M_{c,pl}$;
- failure of the concrete in compression $M_{Rd}$.

![Figure F.1: $MN\kappa$-diagram](image)

The bending stiffness is determined with the relation: $EI = \frac{M}{k}$. The $MN\kappa$-diagram is made for one normal force. Recalculation of the diagram is required if the magnitude of this normal force changes. Note from figure F.1 that for a higher load $M$ the bending stiffness $EI$ reduces. This, because the amount of crack formation in the cross-section increases for higher loads.

Assumptions that are used:

- concrete in tension is neglected;
- steel and concrete strains are proportional with respect to the neutral axis;
• bi-linear $\sigma - \varepsilon$-diagram for steel and concrete (figure E2).

![Stress-strain-diagram for concrete (left) and steel (right)](image)

Figure E2: stress-strain-diagram for concrete (left) and steel (right)

The four distinctive points are here described: $M_r$, $M_{c,pl}$, $M_y$, and $M_{Rd}$ (figure E3).

**Cracking of the concrete in tension**

• tension strength concrete: $\sigma_c \leq f_{ctm}$;
• $f_{cd} < E_c \cdot \varepsilon_c$, where $\varepsilon_c \leq 1.75\%$;
• $E_c = \frac{f_{cd}}{\varepsilon_c} = \frac{f_{cd}}{1.75\%}$ (for concrete class up to C50/60);
• steel B500: $E_s = 200000 \text{ N/mm}^2$ and $f_{sd} = f_{sk} = 500 \text{ N/mm}^2$;
• $\varepsilon_{c2} = \frac{f_{ctm}}{E_c}$.

Strain- and stress distribution, and internal forces to calculate $M_r$ are illustrated in figure E3a.

**Crushing of the concrete in compression in the outer fibre**

• $\varepsilon_{s1} \leq \varepsilon_{sy}$ and $\varepsilon_{s2} \leq \varepsilon_{sy}$;
• $\varepsilon_{c,pl} = \varepsilon_c = \varepsilon_{c3} = 1.75\%$ and $\sigma_c = f_{cd} = 20 \text{ N/mm}^2$;
• $E_c = \frac{f_{cd}}{\varepsilon_{c3}}$;
• $\varepsilon_{sy} = \frac{f_{yd}}{E_s} = \frac{f_{sk}}{E_s} = 2.5\%$.

Strain- and stress distribution, and internal forces to calculate $M_{c,pl}$ are illustrated in figure E3b.

**Yielding of the steel**

• $\sigma_{s2} = f_{yd}$;
• $\sigma_{s1} \leq f_{yd}$ and $1.75\% < \varepsilon_c < 3.5\%$;
• yielding of the steel for $\varepsilon_{s2} = \varepsilon_{sy} = 2.5\%$.

Strain- and stress distribution, and internal forces to calculate $M_y$ are illustrated in figure E3c.

**Failure of the concrete in compression**

• $\varepsilon_c = \varepsilon_{cu3} = 3.5\%$;
• $\varepsilon_{s2} < \varepsilon_{ud} = 0.9 \varepsilon_{uk} = 0.9 \cdot 25\% = 22.5\%$.

Strain- and stress distribution, and internal forces to calculate $M_{Rd}$ are illustrated in figure E3d.
(a) Cracking of the concrete in tension: $M_r$

(b) Crushing of the concrete in the outer fibre: $M_{c,pl}$

(c) Yielding of the steel: $M_y$

(d) Failure of the concrete in compression: $M_{Rd}$

Figure E3: Strain- and stress distribution, and internal forces to calculate $M_r$, $M_{c,pl}$, $M_y$, and $M_{Rd}$
F.2. BENDING STIFFNESS

In this section the sagging- and hogging bending stiffness of the concrete top flange is calculated.

F.2.1. SAGGING MOMENT

Design of the cross-section for the sagging region:

- height of the concrete in the ribs: \( x = 35 \text{ mm} \);
- top reinforcement: standard mesh \( \phi 8 - 200 \rightarrow A_{s1} = 72 \text{ mm}^2 \);
- bottom reinforcement: \( 3\phi 12 \rightarrow A_{s2} = 339 \text{ mm}^2 \).

The cross-section in figure 9.12b is reduced for fire and simplified to determine the properties. The dimensions are:

- \( h_1 = h_c - f_{R60} = 60 - 22 = 38 \text{ mm} \);
- \( h_2 = x = 35 \text{ mm} \);
- \( h = h_1 + h_2 = 38 + 35 = 73 \text{ mm} \);
- \( b_1 = b_d = 286 \text{ mm} \);
- \( b_2 = b_d - b_0 - 2f_{R60} - \frac{2x}{\tan(\alpha)} = 286 - 60 - 2\cdot22 - \frac{2\cdot35}{\tan(80)} = 170 \text{ mm} \);
- \( d_1 = c_d + \frac{b_1}{2} = 25 + \frac{286}{2} = 29 \text{ mm} \);
- \( d_2 = h_c = 60 \text{ mm} \).

Here are \( d_1 \) and \( d_2 \) the effective depths of the top- and bottom reinforcement, respectively.

The neutral axis of the cross-section is calculated.

\[
e_1 = \frac{\frac{1}{2}b_1h_1^2 + b_2h_2(h - \frac{h_2}{2}) + n(A_{s1}d_1 + A_{s2}d_2)}{b_1h_1 + b_2h_2 + n(A_{s1} + A_{s2})} = \frac{\frac{1}{2}286\cdot38^2 + 170\cdot35(73 - \frac{35}{2}) + 10.7(72\cdot29 + 339\cdot60)}{286\cdot38 + 170\cdot35 + 10.7(72 + 339)} = 36.6 \text{ mm}
\]

\[
e_2 = h - e_1 = 73 - 36.3 = 36.4 \text{ mm}
\]

The design of the cross-section to resist the sagging moment \( M_{Ed}^{+} \) is illustrated in figure F.4. The stress-strain relations and internal forces are shown on the right.

The height of the compression zone \( x_u \) is determined with the following relations.

Force equilibrium:

\[\Sigma F_h = 0 : \quad N_c + N_{s1} - N_{s2} - N_{Ed} = 0\]

The compression force:

\[N_c = abx_u f_{cd}\]

Figure F4: Sagging moment at cross-section AA in figure 9.14
The force in the reinforcement depends on the stress-strain relations.

Force in the top reinforcement:

\[ \epsilon_{s1} = \frac{x_u - \alpha_1}{x_u} \cdot \epsilon_{cu3} \]
\[ \sigma_{s1} = \min(\epsilon_{s1} \cdot E_s; f_{sd}) \]
\[ N_{s1} = A_{s1} \cdot \sigma_{s1} \]

Force in the bottom reinforcement:

\[ \epsilon_{s2} = \frac{d_2 - x_u}{x_u} \cdot \epsilon_{cu3} \]
\[ \sigma_{s2} = \min(\epsilon_{s2} \cdot E_s; f_{sd}) \]
\[ N_{s2} = A_{s2} \cdot \sigma_{s2} \]

The height of the compression zone \( x_u \) is obtained from these expressions. The set of equations above is solved in Maple.

\[
x_u = 28.0 \text{ mm}; \\
N_c = 180 \text{ kN}; \\
N_{s1} = -1.8 \text{ kN}; \\
N_{s2} = 135 \text{ kN}.
\]

Note that the top reinforcement mesh is as well compression as in tension. For simplicity is the height of the compression zone is therefore recalculated neglecting the contribution of the top reinforcement (\( N_{s1} = 0 \)). Giving:

\[
x_u = 27.9 \text{ mm}; \\
N_c = 180 \text{ kN}; \\
N_{s2} = 136 \text{ kN}.
\]

The strains are:

\[ \epsilon_{cu3} = 3.5\% \to \text{failure of the concrete in compression} \]
\[ \epsilon_{s2} = 2.013\% < \epsilon_{sy} = 2.5\% \to \text{tension reinforcement is not yielding} \]

The radius of curvature:

\[ \kappa_{Rd}^+ = \frac{\epsilon_{cu3} + \epsilon_{s2}}{d_2} \]
\[ = \frac{3.5 + 2.013}{60} = 0.0919 \text{ m}^{-1} \]

Bending moment resistance:

\[ M_{Rd}^+ = N_c(e_1 - \beta x_u) + N_{s2}(d_2 - e_1) \]
\[ = (180(36.6 - 0.39 \cdot 27.9) + 136(60 - 36.6)) \cdot 10^{-3} = 7.814 \text{ kNm} \]

Bending stiffness:

\[ EI_{Rd}^+ = \frac{M_{Rd}^+}{\kappa_{Rd}^+} = \frac{7.814}{0.0919} = 85.0 \text{ kNm}^2 \]

**F.2.2. HOGGING MOMENT**

Design of the cross-section for the hogging region:

- height of the concrete in the ribs: \( x = 70 \text{ mm} \);
- hogging moment reinforcement: standard mesh \( \varnothing 8 - 200 \text{ and} \varnothing 12 \to A_{s1} = 72 + 339 = 411 \text{ mm}^2 \);
The cross-section in figure 9.12b is reduced for fire and simplified to determine the properties. The dimensions are:

- \( h = h_c + x - f_{R60} = 60 + 70 - 22 = 108 \text{ mm}; \)
- \( b = b_d - b_0 - 2f_{R90} - \frac{x}{\tan(\alpha)} = 286 - 60 - 2 \cdot 22 - \frac{70}{\tan(80)} = 170 \text{ mm}; \)
- \( d_1 = h - c_d + \frac{y}{2} = 108 - 25 + \frac{22}{2} = 79 \text{ mm}; \)
- \( d_2 = h - h_c = 108 - 60 = 48 \text{ mm}. \)

Determine the neutral axis of the cross-section:

- \( e_1 = \frac{\frac{1}{2}bh^2 + n(A_{s1}d_1 + A_{s2}d_2)}{bh + n(A_{s1} + A_{s2})} = \frac{1}{2} 170 \cdot 108^2 + 10.7 \cdot (411 \cdot 29 + 339 \cdot 60)}{170 \cdot 108 + 10.7 \cdot (411 + 339)} = 49.0 \text{ mm} \)
- \( e_2 = h - h_1 = 108 - 49.0 = 59.0 \text{ mm} \)

The design of the cross-section to resist the hogging moment \( M_{Ed} \) is illustrated in figure F5. The stress-strain relations and internal forces are shown on the right.

The height of the compression zone \( x_u \) is determined with the following relations.

Force equilibrium gives:

- \( \Sigma F_h = 0: \ N_c + N_{s2} - N_{s1} - N_{Ed} = 0 \)

The compression force:

- \( N_c = abx_u f_{Ed} \)

The force in the reinforcement depends on the stress-strain relations.

Force in the top reinforcement:

- \( \epsilon_{s1} = \frac{d_1 - x_u}{x_u} \cdot \epsilon_{cu3} \)
- \( \sigma_{s1} = \min(\epsilon_{s1} \cdot E_s; f_{sd}) \)
- \( N_{s1} = A_{s1} \cdot \sigma_{s1} \)

Force in the bottom reinforcement:

- \( \epsilon_{s2} = \frac{x_u - d_2}{x_u} \cdot \epsilon_{cu3} \)
- \( \sigma_{s2} = \min(\epsilon_{s2} \cdot E_s; f_{sd}) \)
- \( N_{s2} = A_{s2} \cdot \sigma_{s2} \)

The height of the compression zone \( x_u \) is obtained from these expressions. The set of equations above is solved in Maple.

Figure F5: Hogging moment at cross-section BB in figure 9.14
\[ x_u = 51.0 \text{ mm}; \]
\[ N_C = 194 \text{ kN}; \]
\[ N_{s1} = 158 \text{ kN}; \]
\[ N_{s2} = 6.9 \text{ kN}. \]

The strains are:
\[ \epsilon_{cu3} = 3.5\%e \rightarrow \text{failure of the concrete in compression} \]
\[ \epsilon_{s1} = 1.926\%e < \epsilon_{sy} = 2.5\%e \rightarrow \text{tension reinforcement is not yielding} \]

The radius of curvature:
\[ \kappa_{Rd} = \frac{\epsilon_{cu3} + \epsilon_{s1}}{d_1} = \frac{3.5 + 1.926}{60} = 0.0687 \text{ m}^{-1} \]

Bending moment resistance:
\[ M_{Rd} = N_C(e_2 - \beta x_u) + N_{s1}(d_1 - e_2) + N_{s2}(e_2 - d_2) \]
\[ = (194(59.0 - 0.39 \cdot 51.0) + 158(79 - 59.0) + 6.9(59.0 - 48) \cdot 10^{-3} = 10.854 \text{ kNm} \]

Bending stiffness:
\[ EI_{Rd} = \frac{M_{Rd}}{\kappa_{Rd}} = \frac{10.854}{0.0687} = 158.0 \text{ kNm}^2 \]
G.1. CROSS-SECTIONAL PROPERTIES

Eurocode 3 part 1 appendix C1 gives a method to determine the cross-section properties for thin-walled open cross-sections [5, p.121-122]. This method is used to obtain the cross-sectional properties of the steel deck.

Area of the cross-section parts:

\[ dA_n = \left[ t_n \cdot \sqrt{(y_n - y_{n-1})^2 + (z_n - z_{n-1})^2} \right] \]

Cross-section area:

\[ A_{gr} = \sum_{n=2}^{11} dA_n \]

Effective cross-section area and corresponding first moment of area with respect to the y-axis:

\[ A_{eff} = \left( \sum_{n=2}^{11} 0 + \sum_{n=12}^{15} \right) dA_n \]
\[ S_{eff} = \left( \sum_{n=2}^{11} 0 + \sum_{n=12}^{15} \right) (z_n + z_{n-1}) \cdot \frac{dA_n}{2} \]

Coordinate for gravity centre:

\[ y_{gc} = \frac{S_{eff}}{A_{eff}} \]

Second moment of area with respect to original y-axis \((y = 0)\) and new y-axis through gravity centre \((y = y_{gc})\):

\[ I_{y,0} = \left( \sum_{n=2}^{11} 0 + \sum_{n=12}^{15} \right) \left[ (z_n)^2 + (z_{n-1})^2 + z_n \cdot z_{n-1} \right] \cdot \frac{dA_n}{3} \]
\[ I_{y,eff} = I_{y,0} - A_{eff} \cdot (y_{gc})^2 \]
Cross-section properties of the deck per meter width:

\[ A_p = A_{gr} \cdot \frac{1000}{b_d/2} \text{[mm}^2\text{/m]} \]

\[ A_{pe} = A_{eff} \cdot \frac{1000}{b_d/2} \text{[mm}^2\text{/m]} \]

\[ I_{eff} = I_{y,eff} \cdot \frac{1000}{b_d/2} \cdot 10^4 \text{mm}^4\text{/m]} \]

\[ e_t = y_{gc} \text{[mm]} \]

\[ e_c = h_w - y_{gc} \text{[mm]} \]

Where:

- \( b_d \) is the width of the deck panel in mm;
- \( A_{gr} \) is the gross cross-section of the deck in mm\(^2\) per meter width;
- \( A_{pe} \) is the estimated effective cross-section of the deck in mm\(^2\) per meter width;
- \( I_{eff} \) is the estimated effective second moment of area of the deck in \( \cdot 10^4 \text{mm}^4\) per meter width;
- \( e_t \) is the height of the tension zone;
- \( e_c \) is the height of the compression zone.

The elastic section modulus is:

\[ W_{el} = \frac{I_{eff}}{\max(e_c; e_t)} \cdot 10^3 \text{mm}^3\text{/m]} \]

The elastic moment resistance of the deck:

\[ M_{el,Rd} = W_{el} \cdot f_{yb,d} \text{[kNm/m]} \]

### G.2. The Effective Cross-Section

Design rules for compression elements with edge or intermediate stiffeners are given in Eurocode 3 part 3 (EC3-3). Here is a spring modelled to allow for the effect of the stiffener. The width and thickness of the compression elements are reduced to take the effect of local buckling into account. In this section is the reduced area calculated for the intermediate flange stiffener and edge stiffener in the top flange, and the longitudinal web stiffener.

#### G.2.1. Intermediate Stiffener

Design rules for intermediate flange stiffeners are given in article 5.5.3.4.2 of Eurocode 3 part 3 [5, p.34-36].

Dimensions of the intermediate stiffener in the top flange are designed in chapter 8. Where:

- \( b_0 \) is the width of the top flange: 60 mm;
- \( t \) is the thickness of the deck: 1.21 mm;
- \( b_r \) is the width of the base of the intermediate stiffener in the top flange: 25 mm;
- \( b_{rr} \) is the width of the crown of the intermediate stiffener in the top flange: 20 mm;
- \( h_s \) is the height of the intermediate stiffener in the top flange: 6 mm.

The elastic critical buckling stress for one central flange stiffener is obtained from:

\[
\sigma_{cr,s} = \frac{4.2k_w E}{A_s} \cdot \sqrt{\frac{I_s t^3}{4b_p^2(2b_p+3b_s)}}
\]

\( b_p \) and \( b_s \) are illustrated in figure G.1.
The effective cross-section

Cross-section to calculate $A_s$

Cross-section to calculate $I_s$

Figure G.1: Compression flange with one intermediate stiffener [5, p.34]

$$b_p = \frac{b_0 - b_r}{2} = \frac{60 - 25}{2} = 17.5 \text{ mm}$$

$$b_s = b_{rr} + \sqrt{h^2 + \left(\frac{b_r - b_{rr}}{2}\right)^2} = 20 + \sqrt{6^2 + \left(\frac{25 - 20}{2}\right)^2} = 33 \text{ mm}$$

The effective width $b_{eff}$ is calculated with the reduction factor $\rho$ according to Eurocode 3 part 5 (EC3-5) for an internal element under uniform compression ($k_\sigma = 4$ and $\Psi = 1$) [13, p.18-21].

$$\epsilon = \sqrt{\frac{235}{f_y [\text{N/mm}^2]}} = \sqrt{\frac{235}{350}} = 0.819$$

$$\lambda_p = \frac{b_{pt}}{28.4 \epsilon \sqrt{k_\sigma}} = \frac{17.5 \cdot 1.21}{28.4 \cdot 0.819 \sqrt{4}} = 0.311$$

if $\lambda_p \leq 0.5 + \sqrt{0.085 - 0.055 \Psi} \rightarrow \rho = 1.0$

$$\lambda_p = 0.311 < 0.5 + \sqrt{0.085 - 0.055 \cdot 1} = 0.673 \rightarrow \rho = 1.0$$

$$b_{eff} = \rho b_p = 1.0 \cdot 17.5 = 17.5 \text{ mm}$$

The cross-section area $A_s$ and the second moment of area $I_s$ of the stiffener are determined with the dimensions given in figure G.1.

$$A_s = 61.1 \text{ mm}^2 \text{ (with } 0.5b_{eff} = 8.75 \text{ mm})$$

$$I_s = 614.4 \text{ mm}^4 \text{ (with } 15t = 18.2 \leq b_p = 17.5 \text{ mm})$$

The value $k_w$, that takes the partial restrained by the webs into account, is obtained from $l_b$ and $k_{w0}$.

$$l_b = 3.07 \sqrt{\frac{I_s b_p^2 (2b_p + 3b_s)}{t^4}} = 3.07 \sqrt{\frac{614.4 \cdot 17.5^2 (2 \cdot 17.5 + 33)}{1.21^4}} = 188.6$$

$$b_t = 2 \cdot b_p + b_s = 2 \cdot 17.5 + 33 = 68 \text{ mm}$$

$$s_w = 231.4 \text{ mm (determined in chapter 10)}$$

$$k_{w0} = \sqrt{\frac{s_w + 2b_t}{s_w + 0.5b_t}} = \sqrt{\frac{231.4 + 2 \cdot 68}{231.4 + 0.5 \cdot 68}} = 1.177$$
The value $k_w$ is:

\[
\text{if } \frac{l_b}{s_w} = \frac{188.6}{231.4} = 0.82 < 2 \text{ then,}
\]

\[
k_w = k_{wo} - (k_{wo} - 1) \cdot \left[ \frac{2}{210000} - \left( \frac{188.6}{231.4} \right)^2 \right] = 1.178 - (1.178 - 1) \cdot \left[ \frac{2 \cdot 188.6}{231.4} - \left( \frac{188.6}{231.4} \right)^2 \right] = 1.0060
\]

Note that for a pin-joined connection the factor $k_w = 1.0$. The restrain of the webs is negligible.

The elastic critical stress is:

\[
\sigma_{cr,s} = \frac{4.2 \cdot 1.006 \cdot 210000}{61.1} \cdot \sqrt{\frac{614.4 \cdot 1.21^3}{4 \cdot 17.5^2 (2 \cdot 17.5 + 3.33)}} = 1182 \text{ N/mm}^2
\]

The reduced effective area of the stiffener $A_{s,red}$ is obtained from:

\[
A_{s,red} = \kappa_d A_s \frac{f_{yb}}{\sigma_{com,ser}} \leq A_s
\]

The reduction factor $\kappa_d$ is [5, p.27]:

Where:

\[
\lambda_d = \sqrt{\frac{f_{yb}}{\sigma_{cr,s}}} = \sqrt{\frac{350}{1182}} = 0.544 < 0.65
\]

\[
\Rightarrow \kappa_d = 1.0
\]

$\sigma_{com,ser}$ is the compressive stress at the centre line of the stiffener calculated on basis of the effective cross-section. The stiffener is fully effective, giving: $\sigma_{com,ser} = 350$ N/mm$^2$.

\[
A_{s,red} = 1.0 \cdot 61.1 \cdot \frac{350}{350} = 61.1 \text{ mm}^2
\]

The webs are also stiffened. The interaction of flange and web stiffeners is taken into account with $\lambda_d = \sqrt{\frac{f_{yb}}{\sigma_{cr,mod}}}$, where $\sigma_{cr,mod}$ is calculated in the following section.

If the effective section properties are determined, the reduced effective area $A_{s,red}$ should be represented by a reduced thickness $t_{red}$ for all elements in $A_s$. Here is $t_{red} = t = 1.21$ mm, because the stiffener is fully effective.

In a SLS calculation is allowed to calculate the effective section properties of the stiffener with the design thickness $t$.

### G.2.2. DECK WITH FLANGE AND WEB STIFFENERS

As mentioned above gives Eurocode 3 part 3 a modified elastic critical stress for sheeting with flange and web stiffeners [5, p.40].

\[
\sigma_{cr,mod} = \sqrt{\frac{\sigma_{cr,s}}{1 + \left( \beta_s \frac{\sigma_{cr,s}}{\sigma_{cr,sa}} \right)^4}}
\]

Where:

- $\sigma_{cr,s}$ is the elastic critical stress for an intermediate flange stiffener;
- $\sigma_{cr,sa}$ is the elastic critical stress of the stiffener closed to the compression flange.

For a profile in bending is $\beta_s = 1 - \frac{h_a + 0.5h_a}{e_c}$. 

---

The value $k_w$ is:

\[
\text{if } \frac{l_b}{s_w} = \frac{188.6}{231.4} = 0.82 < 2 \text{ then,}
\]

\[
k_w = k_{wo} - (k_{wo} - 1) \cdot \left[ \frac{2}{210000} - \left( \frac{188.6}{231.4} \right)^2 \right] = 1.178 - (1.178 - 1) \cdot \left[ \frac{2 \cdot 188.6}{231.4} - \left( \frac{188.6}{231.4} \right)^2 \right] = 1.0060
\]

Note that for a pin-joined connection the factor $k_w = 1.0$. The restrain of the webs is negligible.

The elastic critical stress is:

\[
\sigma_{cr,s} = \frac{4.2 \cdot 1.006 \cdot 210000}{61.1} \cdot \sqrt{\frac{614.4 \cdot 1.21^3}{4 \cdot 17.5^2 (2 \cdot 17.5 + 3.33)}} = 1182 \text{ N/mm}^2
\]

The reduced effective area of the stiffener $A_{s,red}$ is obtained from:

\[
A_{s,red} = \kappa_d A_s \frac{f_{yb}}{\sigma_{com,ser}} \leq A_s
\]

The reduction factor $\kappa_d$ is [5, p.27]:

Where:

\[
\lambda_d = \sqrt{\frac{f_{yb}}{\sigma_{cr,s}}} = \sqrt{\frac{350}{1182}} = 0.544 < 0.65
\]

\[
\Rightarrow \kappa_d = 1.0
\]

$\sigma_{com,ser}$ is the compressive stress at the centre line of the stiffener calculated on basis of the effective cross-section. The stiffener is fully effective, giving: $\sigma_{com,ser} = 350$ N/mm$^2$.

\[
A_{s,red} = 1.0 \cdot 61.1 \cdot \frac{350}{350} = 61.1 \text{ mm}^2
\]

The webs are also stiffened. The interaction of flange and web stiffeners is taken into account with $\lambda_d = \sqrt{\frac{f_{yb}}{\sigma_{cr,mod}}}$, where $\sigma_{cr,mod}$ is calculated in the following section.

If the effective section properties are determined, the reduced effective area $A_{s,red}$ should be represented by a reduced thickness $t_{red}$ for all elements in $A_s$. Here is $t_{red} = t = 1.21$ mm, because the stiffener is fully effective.

In a SLS calculation is allowed to calculate the effective section properties of the stiffener with the design thickness $t$.

### G.2.2. DECK WITH FLANGE AND WEB STIFFENERS

As mentioned above gives Eurocode 3 part 3 a modified elastic critical stress for sheeting with flange and web stiffeners [5, p.40].

\[
\sigma_{cr,mod} = \sqrt{\frac{\sigma_{cr,s}}{1 + \left( \beta_s \frac{\sigma_{cr,s}}{\sigma_{cr,sa}} \right)^4}}
\]

Where:

- $\sigma_{cr,s}$ is the elastic critical stress for an intermediate flange stiffener;
- $\sigma_{cr,sa}$ is the elastic critical stress of the stiffener closed to the compression flange.

For a profile in bending is $\beta_s = 1 - \frac{h_a + 0.5h_a}{e_c}$. 

---
G.2.3. EDGE STIFFENERS

Design rules for edge stiffeners of plane elements are given in article 5.5.3.2 of Eurocode 3 part 3 [5, p.28-31]. These design rules are applicable if the angle between the plane element and the stiffener is between 45° and 135°. Here is the angle 100°. Assumed is that, the plane element in figure G.2 is equal to the width of the top flange, \( b_p = b_0 = 60 \text{ mm} \), and that the length of the edge stiffener \( c \) is 20 mm. The effective width \( b_{e1} \) and \( b_{e2} \) are already calculated in the previous part, where is shown that the top flange is fully effective.

The initial effective cross-section is determined assuming that the stiffener gives full restraint \( K = \infty \). The effective width \( c_{eff} \) is obtained with reduction factor \( \rho \).

\[
c_{eff} = \rho b_{p,c}
\]

\( \rho \) is calculated according to EC3-5 with the buckling factor \( k_\sigma \) obtained from EC3-3.

\[
k_\sigma = \begin{cases} 0.5, & \text{if } \frac{b_{p,c}}{b_p} \leq 0.35 \\ 0.5 + 0.83 \sqrt{ \left( \frac{b_{p,c}}{b_p} - 0.35 \right)^2 }, & \text{if } 0.35 < \frac{b_{p,c}}{b_p} \leq 0.6 \end{cases}
\]

Where \( b_{p,c} \approx c \) is \( \frac{b_{p,c}}{b_p} = \frac{20}{60} = 0.33 \), giving \( k_\sigma = 0.5 \). Further assume that \( \Psi = 1 \).

\[
\lambda_p = \frac{ct}{28.4c \sqrt{k_\sigma}} = \frac{20 \cdot 1.21}{28.4 \cdot 0.819 \sqrt{0.5}} = 1.004
\]

if \( \lambda_p > 0.748 \) \( \rightarrow \) \( \rho = \frac{\lambda_p - 0.188}{\lambda_p^2} \)

\[
\lambda_p = 1.004 > 0.748 \quad \rightarrow \quad \rho = \frac{1.004 - 0.188}{1.004^2} = 0.809
\]

\[
c_{eff} = \rho c = 0.809 \cdot 20 = 16.2 \text{ mm}
\]

The effective cross-sectional area of the edge stiffener \( A_s \) is:

\[
A_s = t(b_{e2} + c_{eff}) = 1.21(8.75 + 16.2) = 30.2 \text{ mm}^2
\]

The edge stiffener is supported by the web of the adjacent deck panel and therefore is \( K = \infty \). Iterations therefore are not required and \( \kappa_d = 1.0 \).

In a SLS calculation the effective section properties of the stiffener are calculated with the design thickness \( t \).
**G.2.4. WEB STIFFENERS**

Design rules for web stiffeners are given in article 5.5.3.4.3 of Eurocode 3 part 3 [5, p.36-40].

Reduction to take account for local buckling are required for compression elements. In this section only the top longitudinal stiffener is taken into account, because the bottom longitudinal stiffener in the tension zone and is fully effective. Therefore, the web is considered as a stiffened web with a single stiffener in the compression zone as illustrated in the middle of figure G.3.

Dimensions of the web stiffener are designed in chapter 10. Where:

\[ h_a = 25 \text{ mm}; \]
\[ h_{sa} = 10 \text{ mm}; \]
\[ \varphi = \psi = 80^\circ. \]

\( h_a, h_{sa} \) and \( \varphi \) are illustrated in figure G.3. In chapter 10 is assumed that the angle of the longitudinal stiffeners is \( 45^\circ \). \( s_a \) and \( s_{sa} \) are:

\[ s_a = \frac{h_a}{\sin(\psi)} = \frac{25}{\sin(80^\circ)} = 25.4 \text{ mm} \]
\[ s_{sa} = \frac{h_{sa}}{\sin(45^\circ)} = \frac{10}{\sin(45^\circ)} = 14.1 \text{ mm} \]

The effective area of the stiffener is:

\[ A_{sa} = t(s_{eff,2} + s_{eff,3} + s_{sa}) \]

![Figure G.3: Effective cross-sections of webs of trapezoidal sheets [5, p.37]](image)

The initial location of the neutral axis is obtained using the effective section of the flange and the gross-section of the web. Note that the embossed area does not contribute to the bending stiffness and thus \( s_{eff,3} \) is 0. The geometry of half a deck is described in nodes in Maple, where the reductions of the flange are taken in account. The cross-sectional properties are calculated with the method from section G.1. The neutral axis is defined with:

\[ e_c = 96.7 \text{ mm (height of the compression zone)} \]
\[ e_t = 123.3 \text{ mm (height of the tension zone)} \]
The effective area of the longitudinal stiffener in compression is:

\[ s_{\text{eff},0} = 0.76 \cdot t_0 \cdot \sqrt{\frac{E}{f_y}} = 0.76 \cdot 1.21 \cdot \sqrt{\frac{210000}{350}} = 22.5 \, \text{mm} \]

The dimensions of \( s_{\text{eff},1} \) and \( s_{\text{eff},2} \) are:

\[ s_{\text{eff},1} = s_{\text{eff},0} = 22.5 \, \text{mm} \]

\[ s_{\text{eff},2} = \left( 1 + 0.5 \cdot \left( \frac{h_a}{e_c} \right) \right) \cdot s_{\text{eff},0} = \left( 1 + 0.5 \cdot \left( \frac{25}{96.7} \right) \right) \cdot 22.5 = 25.4 \, \text{mm} \]

Here is \( s_{\text{eff},1} + s_{\text{eff},2} \geq s_a \), and thus \( s_a \) is fully effective. The lengths are revised:

\[ s_{\text{eff},1} = \frac{s_a}{2 + 0.5h_a/e_c} = \frac{25.4}{2 + 0.5 \cdot 25/96.7} = 11.9 \, \text{mm} \]

\[ s_{\text{eff},2} = s_a \cdot \frac{(1 + 0.5h_a/e_c)}{2 + 0.5h_a/e_c} = 25.4 \cdot \frac{(1 + 0.5 \cdot 25/96.7)}{2 + 0.5 \cdot 25/96.7} = 13.5 \, \text{mm} \]

The effective area of the stiffener is:

\[ A_{sa} = 1.21(13.5 + 0 + 14.1) = 33.4 \, \text{mm}^2 \]

The elastic critical stress for the stiffener is obtained from:

\[ \sigma_{cr,sa} = \frac{1.05 \cdot k_f \cdot E}{A_s \cdot s_2(s_1 - s_2)} \sqrt{I_c \cdot r^3 \cdot s_i} \]

Here is:

\[ s_1 = 0.9(s_a + s_{sa} + s_c) = 0.9(25.4 + 14.1 + 191.8) = 208.2 \, \text{mm} \]

\[ s_2 = s_1 - s_a - 0.5 \cdot s_{sa} = 208.2 - 25.4 - 0.5 \cdot 14.1 = 175.8 \, \text{mm} \]

\( s_c \) is illustrated in figure G.3 and is calculated as \( s_c = s_w - s_a - s_{sa} = 231.4 - 25.4 - 14.1 = 191.8 \, \text{mm} \). \( k_f \) is a coefficient that takes the partial restrained of the stiffened web by the flanges into account. This is conservatively taken as 1.0. The second moment of area \( I_s \) of the stiffener is determined from the fold \( s_{sa} \) and two adjacent strips of \( s_{eff,1} \) with respect to the centroidal axis parallel to the web. This gives \( I_s = 568.4 \, \text{mm}^4 \).

The elastic critical stress for the stiffener is:

\[ \sigma_{cr,sa} = \frac{1.05 \cdot 1.0 \cdot 210000 \sqrt{568.4 \cdot 1.21^3 \cdot 208.2}}{33.4 \cdot 175.8(208.2 - 175.8)} = 530 \, \text{N/mm}^2 \]

The modified elastic critical stress is calculated with \( \beta_s = 1 - \frac{25 \cdot 0.5}{96.7} = 0.690 \).

\[ \sigma_{cr,mod} = \frac{\sigma_{cr,s}}{\sqrt{1 + \left[ \frac{\beta_s}{\sigma_{cr,sa}} \right]^4}} = \frac{1182}{\sqrt{1 + \left[ \frac{0.690}{530 \cdot 568.4} \right]^4}} = 737 \, \text{N/mm}^2 \]

The effective area of the longitudinal stiffener in compression is:

\[ A_{sa,red} = \frac{\kappa_d \cdot A_{sa}}{1 - (h_a + 0.5 \cdot h_{sa})/e_c} \text{ but } A_{sa,red} \leq A_{sa} \]

Here is the reduction factor \( \kappa_d \) obtained from:

\[ \lambda_d = \sqrt{\frac{f_y}{\sigma_{cr,mod}}} = \sqrt{\frac{350}{737}} = 0.68 < 1.38 \]

\[ \Rightarrow \kappa_d = 1.47 - 0.723 \cdot \lambda_d = 1.47 - 0.723 \cdot 0.68 = 0.97 \]
Giving:

\[
A_{s,\text{red}} = \frac{0.97 \cdot 33.4}{1 - (25 + 0.5 \cdot 10)/96.7} = 47.0 > 33.4 \text{ mm}^2
\]

Therefore, the web stiffener is fully effective.

However, the modified elastic critical stress should also be used to determine the effective area of the intermediate stiffener of the top flange. Use \( \kappa_d = 0.97 \). The effective area of the flange stiffener is therefore:

\[
A_{s,\text{red}} = 0.97 \cdot 61.1 \cdot \frac{350/1}{350} = 59.3 \text{ mm}^2
\]

If the effective section properties are determined, the reduced effective area \( A_{s,\text{red}} \) should be represented by a reduced thickness \( t_{\text{red}} \) for all elements in \( A_s \). Here is \( t_{\text{red}} = t \cdot \left(A_{s,\text{red}}/A_s\right) = 1.21 \cdot (59.3/61.1) = 1.17 \text{ mm} \).

Note that in a SLS calculation it is allowed to calculate the effective section properties of the stiffener with the design thickness \( t = 1.21 \text{ mm} \).

**G.3. Differential Equation for Deflection**

First are the parameters \( q_1, q_2, q_3, q_l, \gamma_g, \gamma_q, \rho_{cw}, L, L_x \) and \( EI \) defined.

```maple
> restart; q1:=0.35: q2:=2.36: q3:=4.79: q_l:=0: gamma_g:=1: gamma_q:=1: rho_cw:=25: L:=7200: L_x:=600:
> E:=210000: I:=20580000: EI:=E*I:
```

The load \( q \) is written in one function over the length of the span \( x \) with a Heaviside-function and illustrated in a plot (figure G.4).

```maple
> q:=gamma_g*q1+gamma_q*q_l+gamma_g*q2*(Heaviside(x-L_x)-Heaviside(x-(L-L_x)))+gamma_g*q3*((Heaviside(x-0)-Heaviside(x-L_x))+(Heaviside(x-(L-L_x))-Heaviside(x-(L)))):
> plot(q, x=0..L, y=0..(q1+q3));
```

Figure G.4: Plot of the load \( q \) (y-axis) over the span \( x \)

The differential equation and boundary conditions are given and the DE is solved.

```maple
> DE:=EI*diff(w(x),x$4)=q:
> RV:=w(0)=0, (D@@2)(w)(0)=0, w(L)=0, (D@@2)(w)(L)=0:
> dsolve(DV ,RV,w(x)): w:=rhs(%):
```

The deflection of the deck \( w \) is plotted over the length of the span \( x \) and the maximum deflection is obtained (figure G.5).

```maple
> plot(-w,x=0..L,title='w-deflection');
> w_max:=evalf(maximize((w),x=0..L));
```

Figure G.5: Plot of the deflection \( w \) (y-axis) over the span \( x \)
The effect of ponding is included as a uniform load of \( q_0 = 0.7 \cdot w_{\text{max}} \) over the whole length of the span and a new load \( q_d \) is obtained.

\[
q_0 := 0.7 \cdot w_{\text{max}} \cdot \rho_{\text{cw}} / 1000;
\]
\[
q_d := \gamma_g (q_0 + q_1) + \gamma_q q_1 + \gamma_g q_2 (\text{Heaviside}(x-L_x) - \text{Heaviside}(x-(L-L_x))) \\
+ \gamma_g q_3 ((\text{Heaviside}(x-0) - \text{Heaviside}(x-L_x)) + (\text{Heaviside}(x-(L-L_x)) - \text{Heaviside}(x-(L))));
\]
\[
\text{plot}(q_d, x=0..L, y=0..(q_1+q_3));
\]

The same DE and boundary conditions are given and solved for the new load \( q_d \). The relations for the moment \( M \) and shear force \( V \) are defined.

\[
\text{DE} := EI \cdot \text{diff}(w(x), x^4) = q;
\]
\[
\text{RV} := w(0) = 0, (D^{@2})(w)(0) = 0, w(L) = 0, (D^{@2})(w)(L) = 0;
\]
\[
\text{dsolve(DV, RV, w(x))}: w := \text{rhs}(%);
\]
\[
M := -EI \cdot \text{diff}(z, x); V := \text{diff}(M, x);
\]

The maximum deflection, moment and shear force are obtained from the solution of \( w(x) \).

\[
w_{\text{max}} := \text{evalf(maximize}(z, x=0..L)); M_{\text{max}} := \text{maximize}(M, x=0..L); V_{\text{max}} := \text{maximize}(V, x=0..L);
\]
H.1. SHEAR CAPACITY OF THE CONCRETE CROSS-SECTION

The minimum shear capacity of the full concrete cross-section is obtained with Eurocode 2 [6, p.84-87].

\[ V_{Rd} \geq V_{Rd,\text{min}} = \nu_{\text{min}} b_w d \]

Where:

- \( b_w \) is the smallest width of the concrete cross-section in tension: 130 mm;
- \( d \) is the effective depth: 251 mm.

\[ k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{251}} = 1.89 \leq 2.0 \]

\[ \nu_{\text{min}} = 0.035 k \frac{f_{ck}^3}{20} = 0.035 \cdot 1.89 \frac{20^3}{20} = 0.406 \text{ N/mm}^2 \]

\[ V_{Rd} = 0.406 \cdot 130 \cdot 251 \cdot 10^{-3} = 13.2 \text{ kN/rib} > V_{Ed,\text{max}} = 10.8 \text{ kN/rib} \]

Unity check:

\[ UC = \frac{V_{Ed,\text{max}}}{V_{Rd}} = \frac{10.8}{13.2} = 0.81 \]

The full cross-section is able to resist the maximum shear force at position of the support.