COMPOSITE BRIDGE DESIGN
FOR SMALL AND MEDIUM SPANS

Final Report

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COMPOSITE BRIDGE DESIGN FOR SMALL AND MEDIUM SPANS

Abstract

(1) The research project was undertaken to develop and test new composite bridge concepts that are easily buildable by the best possible application of construction materials and a high grade of prefabrication.

(2) One of the problems was the use of hybrid girders which lead to significant material cost savings and to a reduction of girder heights. Therefore the fatigue rules given in ENV3 Part 2 have been validated experimentally and by numerical studies with a special regard to the local yielding of the web.

(3) A further investigation were the use of dismountable shear connectors and headed studs with a diameter of 25 mm which are not considered in the codes until now. Studs with a diameter of 25 mm are favourable for bridges due to their higher shear resistance. This test series has included static and dynamic Push-Out tests.

(4) Studies on the behaviour of joints of composite bridges took the special behaviour of partial- and full prefabricated slabs into account. Reference bridges have been used to design the test set-up and cycle loadings were applied to simulate moving traffic loads. The investigations were performed in order to verify the opening of the joints, the fatigue behaviour of the shear studs and to study the transfer of vertical shear forces between the elements.

(5) The works on the fatigue tests on special joints of beams comprised connections in span and on support. Due to the fact that welding activities are expensive bolted connections and fillet welds instead of full penetration welds have been investigated. The best alternatives for an easy erection, stabilising of the steel beams during assembly and concreting as well as achieving a durable structure have been found.

(6) Particular experimental and numerical studies on steel and composite beams with stocky and slender webs reveal that the resistance to vertical shear for S460 according to the ENV3 Part 1–1-1 and 1-5 and the influence of the concrete slab on the plate buckling behaviour under sagging and hogging moments have to be verified and improved.

(7) The effects from the interaction with the concrete slab of the bridge have been evaluated. Comparisons between calculation methods and test results have shown that the simplified rules in ENV4 Part 2 are in a good accordance to more precise research approaches. A sophisticated software, called CASE, has been developed to determine the effects from creep and shrinkage on partial prefabricated slabs.

(8) An additional working item about the vibration behaviour of composite bridges has been performed. Measurements on several composite bridges have shown that usually no adverse comments of pedestrians crossing the bridge simultaneously with traffic occurred. Slender constructions out of HSS and HSC seem to result in vibration problems due to strong perceptions of vibration.

(9) The Design Guide include economic and durable concepts for composite bridges including detailed alternative technologies with background information and complete tender documents for reference bridges. A sophisticated software provide the practical engineer with a very fast and user-friendly tool by considering all possible variants for different designs.

(10) The results of the research project can be transferred directly into new and improved design rules for steel and composite structures by implementing them in ENV 1993 (Design of Steel Structures) and ENV 1994 (Design of Steel and Concrete Composite Structures).
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1 INTRODUCTION AND DESCRIPTION OF INVESTIGATIONS

1.1 Objectives and aims of the project

The usual span length for composite bridges built so far have been in the range of 40 m to 80 m. Bridges with a span lengths of 15 m to 30 m have been mostly built as concrete bridges due to the fact that the superstructure only represents a small part of the total construction works for the main contractor who deals with concrete foundations, -piers and -abutments and therefore tends to maintain the building technique he is accustomed to.

The European steel industry, consultant offices and research institutes have already developed proposals for improvements on competitive construction techniques for composite bridges. Although the materials and the production capacities are available they are not yet generally used by potential contractors.

Obstacles to the use of composite structures so far are:
- missing knowledge on how to design, fabricate and erect composite bridges
- restrictions due to administrative and certification procedures
- experiences with prefabricated superstructures (beams and different technologies for the slabs)

Therefore the main objective of this research programme with ProfilARBED and research institutes has been the developing of concepts that are easily buildable so that it is attractive for being executed by the main contractor without any problems on site. These concepts include the best possible application of materials and a minimum of disturbed traffic to achieve an economic and friendly environmental construction.

The composite bridge concepts being developed are covering the span range of about 15 m to 50 m to link the traditional span lengths of composite bridges. By that range it covers about 75 % of all span requirements for road bridges. The project work has result in the following:
- a guidance for the design of such composite bridges including the background of the design,
- complete tender documents including drawings with all necessary details and static analysis for building standardized demonstration bridges,
- development of a software for the pre-design of single span composite bridges.

The following aspects were identified in a pre-normative study as missing knowledge on how to design, fabricate and erect competitive composite bridges (Table 1-1):

Table 1-1: Principle aspects to be investigated in this project

<table>
<thead>
<tr>
<th>1)</th>
<th>Strength and fatigue of hybrid girders</th>
</tr>
</thead>
<tbody>
<tr>
<td>2)</td>
<td>Static and dynamic push out tests of studs $\varnothing$ 25 mm and dismountable shear connectors</td>
</tr>
<tr>
<td>3)</td>
<td>Behaviour of joints of slabs for full- and partial depth prefabricated elements</td>
</tr>
<tr>
<td>4)</td>
<td>Fatigue tests on special joints in span and on middle support of composite girders</td>
</tr>
<tr>
<td>5)</td>
<td>Plate buckling of stocky and slender webs</td>
</tr>
<tr>
<td>6)</td>
<td>Effects from the interaction with the concrete slab of the bridge</td>
</tr>
<tr>
<td>7)</td>
<td>Software for the design of a composite bridge using different concepts</td>
</tr>
</tbody>
</table>
The aims of the project will be met by the best possible application of construction materials and a high grade of factory prefabrication. A particular feature of the new concept is the sufficient robustness for execution, cost effectiveness and durability in use that shall be realised by the following means:

- Avoiding on site construction problems by applying light weight steel elements (e.g. hybrid girders) that can be easily mounted by mobile cranes without propping and do not need welding on site.

- Application of construction techniques with a minimum of disturbed traffic and a minimum of construction time on site. In addition to this dismountable shear connectors for reusable bridge construction elements shall be tested.

- Use of alternative levels of prefabrication of the concrete slab: completely cast on site, full- and partial depth prefabricated elements. The assembly techniques with full depth prefabricated elements also allow construction during winter time in cold regions.

By using high strength steels for the steel beams (rolled or welded girders) the weight of the elements and the fabrication costs (by reducing the amount of welding) will be reduced. In case of welded girders the concept of hybrid girders shall be used with high strength steels for the flanges only and lower strength steels (which are cheaper) for the webs.

For the development of prefabricated slab-elements the techniques for joining them to the steel beams and preparing the joints between the slab elements and the connection to the in-situ concrete layer shall be improved.

For optimising the total composite assembly the best possibilities of easy erection, stabilising of the steel beams during erection and concreting and achieving a durable structure meeting also esthetical needs shall be found.

The software will include simple and multi-span composite bridges consisting of prefabricated steel beams and concrete slabs. The slab can be solid or constructed using different kinds of prefabricated elements. The longitudinal and transverse reinforcement is considered not to be pre-stressed.

All these investigations and developments will result in competitive and friendly environmental composite bridge structures, which can be designed by unspecialised consulting offices and built by ordinary construction companies.
1.2 Working group

The working group consists of the following partners:

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1.3 Keywords

1. Composite bridge
2. Fatigue behaviour
3. Hybrid girders
4. Composite beams
5. Push-Out tests
6. Shear connectors
7. Partial prefabricated slabs
8. Full prefabricated slabs
9. Joints in the concrete slab
10. Connections
11. Cross girder
12. Plate buckling
13. Slender webs
14. Stocky webs
15. Creep and shrinkage effects
16. Vibration
### 1.4 Description of experimental and numerical investigations

The following tables summarise the experimental tests to be carried out:

*Table 1-2: Experimental tests to be carried out*

<table>
<thead>
<tr>
<th>Test arrangement</th>
<th>Description</th>
<th>Number of tests (offered / performed)</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>a) Tests concerning Serviceability and Fatigue</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><img src="image1" alt="Diagram" /></td>
<td>Hybrid Girder: 3-point bending tests to evaluate the fatigue resistance of a transverse welded attachment. Materials: Steel: S460 &amp; S235</td>
<td>6</td>
<td>CTICM</td>
</tr>
<tr>
<td><img src="image2" alt="Diagram" /></td>
<td>Hybrid Girder: 3-point bending tests to evaluate the fatigue resistance of fillet welds at the flange-web junction (at a specified distance from the support) when these webs are previously plastified by transverse loads. Materials: Steel: S460 &amp; S235</td>
<td>2</td>
<td>CTICM</td>
</tr>
<tr>
<td><img src="image3" alt="Diagram" /></td>
<td>Hybrid Girder: 3-point bending tests to examine the stabilisation of deformations under alternate loading $\Delta\sigma &lt; 1.5 f_y$. Materials: Steel: S460 &amp; S235</td>
<td>1</td>
<td>CTICM</td>
</tr>
</tbody>
</table>
### Static and dynamic Push-Out tests with dismountable shear connectors:

- studs welded on a steel plate bolted to the girder
- prestressed bolts M20 as shear connectors

**Materials:**
- Steel: S355
- Concrete: C30/37

<table>
<thead>
<tr>
<th>4+2</th>
<th>BUG Wuppertal</th>
</tr>
</thead>
</table>

### Static and dynamic Push-Out test with headed shear studs $\varnothing 25$ mm in a partial prefabricated slab

**Materials:**
- Steel: S355
- Concrete: C30/37

<table>
<thead>
<tr>
<th>3+3</th>
<th>BUG Wuppertal</th>
</tr>
</thead>
</table>

### Static and dynamic Push-Out test with headed shear studs $\varnothing 25$ mm in a full prefabricated slab with a shear connector channel filled with mortar

**Materials:**
- Steel: S355
- Concrete: C30/37

<table>
<thead>
<tr>
<th>3+3</th>
<th>BUG Wuppertal</th>
</tr>
</thead>
</table>

### Composite girders with:

- partial prefabricated slabs
- full prefabricated slab under hogging moment and cycle loadings measuring the:
  - opening of the joints
  - fatigue behaviour of the shear studs
  - vertical transfer between the elements

**Materials:**
- Steel: S460
- Concrete: min. C25/30

| 1 | Université de Liège |
| 1 | Luleå Technical University |
### Fatigue tests on special joints of composite beams

<p>| | | |</p>
<table>
<thead>
<tr>
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<tbody>
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<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- a) connection in span
  - cover plates with prestressed bolts
- b) connection on support
  - concrete cross girder with cap plate welded to the beam with different fillet welds

**Materials:**
- Steel: S460
- Concrete: C35/45

**Université de Liège**

### b) Experimental tests for stability phenomena

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
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<tbody>
<tr>
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<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- Plate buckling for stocky webs
  - 2 steel beams
  - 2 composite beams under hogging moment
  - 2 composite beams under sagging moment

**Materials:**
- Steel: S460

**Luleå Technical University**

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
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<tbody>
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<td></td>
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<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- Plate buckling for stocky webs
  - 2 steel beams
  - 2 composite beams under hogging moment
  - 2 composite beams under sagging moment

**Materials:**
- Steel: S460

**Luleå Technical University**

**Total tests:** 44 45
The following table shows the analysis about the effects from the interaction with the concrete slab of the bridge to be carried out:

**Table 1-3: Effects from the interaction with the concrete slab of the bridge to be carried out**

<table>
<thead>
<tr>
<th>Keywords</th>
<th>Locations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effects from the interaction with the concrete slab of the bridge:</td>
<td>RWTH Aachen</td>
</tr>
<tr>
<td>Cracking, slab, joints, creep, shrinkage, tension stiffening, crack width, concrete technology, prefabricated elements, in-situ concrete, CASE, vibration</td>
<td></td>
</tr>
</tbody>
</table>

The following table shows the numerical investigations to be carried out:

**Table 1-4: Numerical investigations (by BE- and FEM simulations) to be carried out**

<table>
<thead>
<tr>
<th>Keywords</th>
<th>Locations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hybrid girders:</td>
<td>RWTH Aachen</td>
</tr>
<tr>
<td>Boundary elements, fatigue, stress intensity factor, crack propagation, Paris-Forman, constants C and m, initial crack length, manual formula</td>
<td></td>
</tr>
<tr>
<td>Stability:</td>
<td></td>
</tr>
<tr>
<td>Eigenvalue-analyses, Eigenshape, Mohr-Coulomb-Law, steel strength, pre-imperfection, b/t-relation, ( \eta )-factor, shear buckling capacity</td>
<td>RWTH Aachen</td>
</tr>
</tbody>
</table>

The following table shows the development of software to be carried out:

**Table 1-5: Software Development to be carried out**

<table>
<thead>
<tr>
<th>Keywords</th>
<th>Locations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Composite bridge design – CBD - :</td>
<td>RWTH Aachen</td>
</tr>
<tr>
<td>Multi-span, loading history, solid slab, partial-/ full prefabricated slab, rolled and plate sections, hybrid girders, shear connectors, sequences of casting, crack-width-criteria</td>
<td></td>
</tr>
<tr>
<td>Creep and shrinkage effects – CASE - :</td>
<td>RWTH Aachen</td>
</tr>
<tr>
<td>Creep and shrinkage curve, deflection curve, bend curve, crack development, iteration, hogging moment, long-run analysis</td>
<td></td>
</tr>
</tbody>
</table>
## 1.5 Ways and Means

The distribution of activities and works between the partners is shown in the following Table:

*Table 1-6: Distribution of works between the partners*

<table>
<thead>
<tr>
<th>Keyword</th>
<th>Investigation</th>
<th>Partner who will do this work:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Experimental</td>
<td>Numerical / in theory</td>
</tr>
<tr>
<td>Co-ordination</td>
<td>×</td>
<td>RWTH Aachen</td>
</tr>
<tr>
<td>Hybrid girder</td>
<td>×</td>
<td>CTICM</td>
</tr>
<tr>
<td></td>
<td>×</td>
<td>RWTH Aachen</td>
</tr>
<tr>
<td>Push-Out tests</td>
<td>×</td>
<td>BUG Wuppertal</td>
</tr>
<tr>
<td>Prefabricated slabs</td>
<td>×</td>
<td>Luleå Technical University</td>
</tr>
<tr>
<td></td>
<td>×</td>
<td>Université de Liège, RWTH Aachen</td>
</tr>
<tr>
<td>Connections</td>
<td>×</td>
<td>Université de Liège, RWTH Aachen</td>
</tr>
<tr>
<td>Stability</td>
<td>×</td>
<td>Luleå Technical University</td>
</tr>
<tr>
<td></td>
<td>×</td>
<td>RWTH Aachen</td>
</tr>
<tr>
<td>Creep / shrinkage</td>
<td>×</td>
<td>RWTH Aachen</td>
</tr>
<tr>
<td>Software CBD, CASE</td>
<td>×</td>
<td>RWTH Aachen</td>
</tr>
<tr>
<td>Design Guide</td>
<td>×</td>
<td>all partners</td>
</tr>
</tbody>
</table>
2 STRENGTH AND FATIGUE OF HYBRID GIRDER

2.1 Experimental investigations

2.1.1 Introduction

A main contribution of CTICM in the present ECSC research project was to evaluate the possibility of using hybrid plate girders in the range of small and medium span bridges. The use of such girders leads to a significant material cost saving (compared to homogeneous girders) and to a reduction of girder heights as well. A main aspect being the validation of the applicability of fatigue rules in Eurocode 3 Part 2 to this type of girders, CTICM has carried out a fatigue test programme on 9 hybrid girders made with S235 steel for webs and S460 steel for flanges:

- **Six tests, denoted TH1, TH2, TH3, TM1, TM2 and TL1**, aimed to evaluate the fatigue resistance of a transverse welded attachment commonly used in steel and composite bridges and to examine the "in service" behaviour of the girder. In order to generate the necessary bending action and stress pattern, these tests were 3-point bending tests carried out on simply supported I hybrid girders (5 meter span) under concentrate cycling loading at mid-span.

- **Two tests, denoted TGAP and TNOGAP respectively**, were 3-point bending tests on simply supported I hybrid girders with 1,8 meter span. The aim of these tests was to evaluate the fatigue resistance of fillet welds at the flange-web junction under high shear stress, when previously plastified by transverse loads (e.g. during launching) and in presence of large contact default at this junction (2mm gap between the tension flange and the web).

- **One test, denoted TOC** (for "oligo-cyclic"), aimed at examining the stabilisation of load-deformations and load-stresses hysteresis loops under alternate loading, in order to clarify, for hybrid girders, the limit condition $\Delta \sigma_{\text{max}} = 1.5 f_y$ in Eurocode 3 Part 2.

2.1.2 Progress of the experimental programme during the research

During a preliminary period, CTICM achieved and managed a certain number of tasks for preparing the experimental programme: final definition of the different tests, detailing of specimens and fabrication sheets, procurement of steel plates from steel producers, fabrication of specimens by steel fabricators, consultation of different laboratories for the test execution, tensile tests for steel properties determination, load calculations for each test, …

So, tests TH1, TH2 and TM1 were finally carried out during the second semester of 1999. Tests TL1, TM2 and TH3 were then carried out during the first 6 months of 2000. So, the serial of 6 fatigue tests concerning the transverse welded attachment were completed in the middle of 2000.

Tests TGAP, TNOGAP and TOC were carried out during the second semester of 2000 and completed at mid-November 2000.

The fatigue test programme were completed with only one month delay compared to the initial scheduled time-table, this delay being mainly due to problems encountered with the capacity of the initial testing machine for the fatigue phase of one of the tests and experimental fatigue lives longer than expected.

Recorded data for all tests have been treated using EXCEL Application and all available information have been transmitted to RWTH (Aachen) on a CD-ROM. A detailed report on the 9 tests has been established and is available, and the significant results obtained from these tests are shortly described here after.
In the light of the results, CTICM has also participated to the elaboration of the Design Guide foreseen as a result of this research project, and has proposed recommendations concerning design and fatigue for hybrid girders.

### 2.1.3 Third parties involved for services allowances

The selected companies and laboratories from the consultations have been:

- **Procurement of steel plates** (free – Many thanks to GTS):
  
  - GTS Industries / Dillinger - F 59760 – Grande Synthe (France)

- **Tensile tests on steel coupons**:
  
  - Institut de Soudure - F 95942 – Roissy CDG (France)

- **Fabrication of specimens**:
  
  - Baudin-Châteauneuf - F 45110 – Châteauneuf-sur-Loire (France)

- **Testing laboratory**:
  
  - CEBTP - F 78470 – Saint-Rémy-les-Chevreuse (France)

### 2.1.4 Description of the fatigue tests

#### 2.1.4.1 Fatigue resistance of a transverse weld attachment – TH/M/L tests

**2.1.4.1.1 Aim of the tests**

Regarding fatigue, main details in small to medium span bridges are transverse welded attachments of stiffeners to flanges and web (see Fig. 2-1). In hybrid girders, their fatigue resistance may be affected by the plastification occurring in the web when the bending is over the elastic bending resistance. So, six tests have been defined to assess the fatigue resistance of such a detail. The tested parameter in these tests was in fact the height $H_p$ of the plastified zone in the web (see Fig. 2-2).

![Fig. 2-1: Test detail](#)

![Fig. 2-2: $H_p$ parameter](#)

Six identical simply supported I girders with a 5 meter span were tested. Each one was submitted to a cyclic concentrated load at mid-span in a 3-point bending test (see Fig. 2-3).
Lateral supports at 1/4, 1/2 and 3/4 of the span

Web 670 X 8 S 235
Flanges 180 X 15 S 460

Fig. 2-3 : Principle of tests for transverse attachment investigation

Three loading configurations were considered for this detail by keeping the same geometrical characteristics of the girders and varying only the average level of the applied stress range: cycles fully in the plastic range (TH tests, for High), cycles around the elastic limit (TM tests, for Medium) and cycles fully in the elastic range (TL test, for Low). From the fatigue point of view, TL test is considered as a reference test on an homogeneous girder. These 3 configurations led to three values of plastified height $H_p$ (see Table 2-1).

The test setup for these 6 tests is pictured in Photo 2-1.

Photo 2-1 : Overview of the test setup for TH/M/L tests
Lateral supports, by attached brackets near the compression flange, were defined to prevent the lateral torsional buckling (see Fig. 2-3). At the beginning of the first test (TH1), only the upper flange had been maintained laterally, but after one million cycles, the specimen showed a notable longitudinal torsional rotation of the section at mid-span and at one support (probably due to an imperfection in the introduction of the vertical loading). Because of that, both flanges were then laterally restrained. See Photo 2-2.

*Photo 2-2: Lateral support*

Measurements concerned vertical displacement (Fig. 2-5) at mid-span and the strain variation as indicated in Fig. 2-6.

![Fig. 2-5: Displacement transducers](image)

![Fig. 2-6: Positions of strain gauges](image)

### 2.1.4.1.2 Loading parameters

Each beam was first submitted to a preliminary loading phase during which 5000 cycles were performed between two load levels $P_{o,\text{min}}$ and $P_{o,\text{max}}$, and then to a static incremental loading from 0 to a $P_{\text{max}}$ level followed by an incremental unloading to 0, in order to check the global behaviour of the beam and the measuring devices. Then, the fatigue phase started, with a load cycling between two levels $P_{\text{min}}$ and $P_{\text{max}}$ until cracks were observed. At the end of fatigue phase, an ultimate loading phase was applied with a load increasing incrementally from 0 to the full collapse of the beam.

A constant normal stress range $\Delta \sigma = 100 \text{ MPa}$ was applied at the web-to-flange junctions for all tests during cycling, that corresponded to a constant load range $\Delta P = 220 \text{ kN}$.

Table 2-1 summarises the loading parameters for the six tests TH/M/L.
Table 2-1: Loading parameters for TH/M/L tests

<table>
<thead>
<tr>
<th>TESTS</th>
<th>TH1, TH2 and TH3</th>
<th>TM1 and TM2</th>
<th>TL1</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_{o,min}$ (kN)</td>
<td>200</td>
<td>200</td>
<td>100</td>
</tr>
<tr>
<td>$P_{o,max}$ (kN)</td>
<td>400</td>
<td>400</td>
<td>200</td>
</tr>
<tr>
<td>$P_{min}$ (kN)</td>
<td>680</td>
<td>520</td>
<td>250</td>
</tr>
<tr>
<td>$P_{max}$ (kN)</td>
<td>880</td>
<td>720</td>
<td>450</td>
</tr>
<tr>
<td>$P_{average}$ (kN)</td>
<td>780 ($\sigma = 1.25 f_{yw}$)</td>
<td>620 ($\sigma = 1.0 f_{yw}$)</td>
<td>350 ($\sigma = 0.56 f_{yw}$)</td>
</tr>
<tr>
<td>$\Delta \sigma$ (Mpa)</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>$H_p$ (mm)</td>
<td>105</td>
<td>60</td>
<td>0</td>
</tr>
</tbody>
</table>

2.1.4.2 Fatigue resistance of longitudinal fillet weld at web-to-flange junction – TGAP/TNOGAP tests

2.1.4.2.1 Aim of the tests

These two tests (called TGAP and TNOGAP, see below) aimed to get experimental results on fatigue resistance of fillet welds at the flange-to-web junction in high shear zone, in order to be compared with design rules. So, they consisted in three-point bending tests on simply supported I hybrid girders with only 1.8 meter span (to have dominant shear effects) under concentrate cycling loading at mid-span (see Fig. 2-7). In order to account for additional effects due to transverse loading occurring during launching operations, it was decided to submit both test girders to a transverse patch load in a critical section before performing fatigue tests themselves. Moreover, a large contact default (gap) has been introduced for one specimen (TGAP test) at the web-to-flange junction (nominally a 2mm gap between the tension flange and the web – see Fig. 2-8) to assess its influence on the fatigue resistance. The second specimen has no gap (TNOGAP test). The magnitude of the transverse load has been calculated just to plastify the fillet weld of the girder with gap. Each girder was therefore submitted to two loading phases: a transverse patch loading phase and a fatigue loading phase.

Fig. 2-7: Principle and dimensions for TGAP/TNOGAP fatigue tests
The 2mm gap, were introduced on the total length of the beam, between the web and the tension flange as indicated in Fig. 2-8.

![Diagram of the gap between the web and the tension flange]

**Fig. 2-8**: Gap between the web and the tension flange

### 2.1.4.2.2 Transverse path loading phase

Each girder was installed under the 2000kN press in the CEBTP Laboratory. Calculations led to apply a 400kN transverse patch load through a 100 mm bearing length. On the tension flange side, the gapped side for TGAP Test, the fillet weld throat is 4mm. At the opposite flange, a bearing length of 400 mm was chosen in order to spread widely the reaction; and the fillet weld throat is 6mm.

It was chosen to apply the transverse patch loading (to plastify the weld) enough far from local effects due to the jack load introduction at mid-span and from support reactions, that is at 500 mm from the support (400 mm from mid-span).
Measurements were set in the patch-loaded section and consisted in six 1D-gages and two 3D-gages (J1 to J12) for strains, and two displacement transducers (C1 and C2) for the shortening of the web.

For each test, a 200kN pre-load was applied and the full test load 400kN was applied twice.
2.1.4.2.3 Fatigue loading phase

After having been submitted to transversal patch loading as explained before, the girders were tested for fatigue loading.

As already mentioned, these tests were 3-point bending tests under concentrate cycling loading at mid-span.

Fig. 2-11 shows the principle and the main dimensions of these fatigue tests.

The investigated section was the patch-loaded section, where the combination of shear, bending and local plastification due to patch loading seemed to be critical.

Calculations led to a variation of the jack load equal to 1000kN in order to have a fatigue resistance governed by shear. Accounting for the ultimate static resistance of the girder (≈1225kN), the jack load cycled between 100kN and 1100kN.

The design fatigue life calculated according to EC3 is 71000 cycles (see Table 2-2), only based on fatigue shear resistance.

Table 2-2 summaries the characteristic values concerning TGAP/TNOGAP tests for fatigue phase.

<table>
<thead>
<tr>
<th>Table 2-2: Calculated summary values for TGAP/TNOGAP fatigue tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_{\text{min}} = 100$ kN , $P_{\text{max}} = 1100$ kN , $\Delta P = 1000$ kN</td>
</tr>
<tr>
<td>$H_p = 42$ mm (at mid-span)</td>
</tr>
<tr>
<td>$\Delta \delta = 4.4$ mm (with shear deformation)</td>
</tr>
<tr>
<td>$\Delta \tau_{\text{weld}} = 156$ MPa</td>
</tr>
<tr>
<td>$\Delta \sigma_{\text{weld}} = 358$ MPa at midspan</td>
</tr>
<tr>
<td>$\Delta \sigma_{\text{weld}} = 199$ MPa at transversally patch-loaded section</td>
</tr>
<tr>
<td>$N_t = 71\ 000$ cycles (shear only)</td>
</tr>
<tr>
<td>$N_{\sigma} = 85\ 000$ cycles (bending only)</td>
</tr>
</tbody>
</table>

Concerning the measurements, the same gages already stuck for transverse loading were used (except J7/J8, out of work). A displacement transducer was installed under the lower flange of the girder, at mid-span.
2.1.4.3 Stabilisation of hysteresis loops under large stresses range – TOC test

2.1.4.3.1 Aim of the tests

In order to ensure a stable structural behaviour, EC3 fatigue rules require, for homogeneous girders, nominal stresses within the elastic limits of the material and a range $\Delta \sigma$ of these stresses not exceeding $1.5 f_y$. The question arises concerning this stability for hybrid girders when the web is plastified and more specifically the limit $1.5 f_y$. This test, called TOC (for Oligo-Cyclic), aimed to answer this question by testing the stabilisation of the structural response of a girder for $\Delta \sigma > 1.5 f_{yw}$ and more especially $\Delta \sigma \geq 1.5 f_{yt}$. This test concerned only the existence of a stabilisation and did not aim at checking the fatigue resistance in these conditions. It consisted in applying an alternate loading (with a stress range $\Delta \sigma \geq 1.5 f_{yt}$) on a simply supported girder of 5 meter span and checking the stabilisation of behaviour during only few cycles.

Pre-design led to the overall dimensions and plates shown in Fig. 2-12. The section has been chosen the same as for TGAP/TNOGAP tests for fabrication facilities. Fig. 2-13 and Photo 2-5 show the realised test arrangement.

---

Fig. 2-12: Principle and dimensions for TOC Test

Fig. 2-13: Scheme of test arrangement
Six 1D-gages were stuck in the section at mid-span to measure the longitudinal strains, and five displacement transducers were used (1 at mid-span and 2 at each support) for deflections.

![Measurement locations for TOC Test](image)

**Fig. 2-14 : Measurement locations for TOC Test**

Fig. 2-15 shows the theoretical characteristic bending stress distributions for TOC Test ($f_{yt} = 492$ MPa, $f_{yw} = 295$ MPa), and the corresponding loading parameters.

![Characteristic bending stress distributions for TOC test](image)

**Fig. 2-15 : Characteristic bending stress distributions for TOC test**
2.1.5 **Plates and materials properties**

2.1.5.1 **Girder dimensions and cross-sectional properties**

Measurements of actual dimensions for all specimens confirm the indicated values in Fig. 2-3, Fig. 2-7 and Fig. 2-12. Differences between the nominal dimensions and the average measured ones were negligible. The measured imperfections of the webs were very low because the geometrical slenderness of these webs were equal to 80 and 27.

*All girders have been fabricated from three main plates only* (offered by GTS - DUNKERQUE – France), described in Table 2-3.

<table>
<thead>
<tr>
<th>Steel grade</th>
<th>Standard</th>
<th>Thickness (mm)</th>
<th>Length (mm)</th>
<th>Width (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S235 JR</td>
<td>EN 10025/93</td>
<td>8</td>
<td>14 000</td>
<td>2 300</td>
</tr>
<tr>
<td>S235 JR</td>
<td>EN 10025/93</td>
<td>12</td>
<td>6 000</td>
<td>1 500</td>
</tr>
<tr>
<td>S460 M</td>
<td>EN 10113-3/93</td>
<td>15</td>
<td>6 500</td>
<td>3 500</td>
</tr>
</tbody>
</table>

All flanges and webs were cut in the longitudinal direction of each plate.

2.1.5.2 **Steel properties of plates**

For each plate, two tensile tests have been carried out in the longitudinal and transverse directions to measure the main steel properties. The test procedure was in accordance with EN 10002-1. These properties are summarised in Table 2-4.

<table>
<thead>
<tr>
<th>Plate thickness</th>
<th>Direction</th>
<th>f_y (MPa)</th>
<th>f_u (MPa)</th>
<th>E (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8 mm</td>
<td>Longitudinal</td>
<td>310</td>
<td>399</td>
<td>207 000</td>
</tr>
<tr>
<td></td>
<td>Transverse</td>
<td>323</td>
<td>400</td>
<td>185 167</td>
</tr>
<tr>
<td>12 mm</td>
<td>Longitudinal</td>
<td>295</td>
<td>390</td>
<td>174 793</td>
</tr>
<tr>
<td></td>
<td>Transverse</td>
<td>303</td>
<td>389</td>
<td>191 901</td>
</tr>
<tr>
<td>15 mm</td>
<td>Longitudinal</td>
<td>492</td>
<td>567</td>
<td>222 990</td>
</tr>
<tr>
<td></td>
<td>Transverse</td>
<td>520</td>
<td>591</td>
<td>216 795</td>
</tr>
</tbody>
</table>

*So, the ratio f_y / f_yw for all tests was about 1.6 to 1.65*

2.1.5.3 **Welds**

The welds at weld-to-flange junctions were realised by submerged arc couple Lincoln L61-780 furnished by LINCOLN ELECTRIC. The characteristics were:

- Yield strength > 450 MPa,
- Tensile strength > 550 MPa,
- Composition: C 0.07, Mn 1.4, Si 0.6, P < 0.025 and S < 0.02,
- Impact ISO-V: +20°: 80 J, 0°: 60 J, -20°: 40 J.
The other welds were semi-automatic ones with wire under gas type OSMC 710 from LINCOLN. The characteristics were:

- Yield strength $> 480$ MPa,
- Tensile strength $> 550$ MPa,
- Composition: C 0.05, Mn 1.5, Si 0.75, P 0.015 and S 0.02,

### 2.1.6 Experimental results and analysis

#### 2.1.6.1 Fatigue resistance of transverse weld attachments – TH/M/L tests

As an example, Fig. 2-16 shows the actual loading histogram for TH2 test. At each peak S3 to S7, fatigue cycles were carried out between $P_{\text{min}}$ and $P_{\text{max}}$ without stop. Numbers of cycles are indicated at the top of the figure, near each peak. Cumulated numbers of cycles are indicated at the bottom, for each unloading. Each point corresponds to a measurement step.

![Loading Histogram](image.png)

- $P_u = 913$ kN
- $P_{\text{max}} = 880$ kN
- $P_{\text{min}} = 680$ kN

**Fig. 2-16 : Example : Loading histogram for TH2 test**

#### 2.1.6.1.1 Fatigue

During the fatigue phases, a visual inspection of all welds had been done at regular intervals (approximately each 50 000 or 100 000 cycles) using either a magnifying glass or dye penetrant tests. More frequent observations were made after 1000 000 cycles (calculated value according to EC3). For all tests, cracks initiated in the web-to-flange fillet weld, just at the toe of the vertical fillet connecting the stiffener to the web and propagated then both in the web and in the tension flange at a low speed.
After identification of a crack, the width of the crack was measured and some supplementary cycles were performed to assess the growth of the crack. In Photo 2-6 and Photo 2-7, some typical cracks are shown.
Table 2-5 gives the number of cycles corresponding to the different stages of the development of the cracks.

Table 2-5:  Results obtained for the 6 tests concerning the transverse welded attachment

<table>
<thead>
<tr>
<th>TEST</th>
<th>TH1</th>
<th>TH2</th>
<th>TH3</th>
<th>TM1</th>
<th>TM2</th>
<th>TL1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Δσ (MPa)</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>$P_{\min}$ (kN)</td>
<td>680</td>
<td>680</td>
<td>680</td>
<td>520</td>
<td>520</td>
<td>250</td>
</tr>
<tr>
<td>$P_{\max}$ (kN)</td>
<td>880</td>
<td>880</td>
<td>880</td>
<td>720</td>
<td>720</td>
<td>450</td>
</tr>
<tr>
<td>$H_p$ (theoretical) (mm)</td>
<td>105</td>
<td>105</td>
<td>105</td>
<td>60</td>
<td>60</td>
<td>0</td>
</tr>
<tr>
<td>$N$ according to EC3-2</td>
<td>$\approx 10^6$</td>
<td>$\approx 10^6$</td>
<td>$\approx 10^6$</td>
<td>$\approx 10^6$</td>
<td>$\approx 10^6$</td>
<td>$\approx 10^6$</td>
</tr>
<tr>
<td>$N_i$ number of cycles at crack initiation ($10^6$)</td>
<td>4.5</td>
<td>3.2</td>
<td>2.1</td>
<td>2.0</td>
<td>2.3</td>
<td>2.0</td>
</tr>
<tr>
<td>$N_s$ number of cycles at end of fatigue test ($10^6$)</td>
<td>5.5</td>
<td>3.7</td>
<td>3.8</td>
<td>3.8</td>
<td>4.1</td>
<td>3.8</td>
</tr>
<tr>
<td>Crossing crack in web at end of fatigue phase</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>No</td>
</tr>
</tbody>
</table>

From Table 2-5, it is clearly observed that the number of cycles at crack initiation and at the end of the fatigue phase are not affected by the height of the plastified zone in the web. This important results can be explained by the simple fact that: except for the first cycle, where the plastification appears, all the subsequent cycles are in elastic range. Due to that, the fatigue behaviour is the same as for homogenous girders.

Concerning the fatigue life, for detail category 80 and $\Delta\sigma = 100$ MPa, Chapter 9 of EC3 gives $N \approx 10^6$ cycles. Fig. 2-18 shows where tests are located versus the design fatigue curve according to EC3 (95% probability of non-failure). For each test, a segment is shown, the origin of which is the crack occurrence and the end is the end of test. Doted lines show the 50% and 5% probability of non-failure.

![Fig. 2-18: TH/M/L fatigue tests: results versus design fatigue curve according EC3](image)
2.1.6.1.2 Ultimate loads

As indicated before, at the end the fatigue phase, the load was increased incrementally from zero up to the ultimate capacity of the girders. Fig. 2-19 gives an example of the load-deflection curve obtained for TH1 test.

![Graph showing load-deflection curve for TH1 Test](image)

**Fig. 2-19 : Load-deflection curve for TH1 Test**

The load-deflection curves obtained for the 6 tests are plotted on Fig. 2-20. TH3 test was stopped at fatigue phase only.

![Graph showing load-deflection curves for TH/M/L tests](image)

**Fig. 2-20 : Load-Deflection curves for TH/M/L tests**

Table 2-6 gives the ultimate load for each test in comparison with the calculated value according to EC3.
Table 2-6: Results obtained for the 6 tests concerning the transverse welded attachment

<table>
<thead>
<tr>
<th>TEST</th>
<th>TH1</th>
<th>TH2</th>
<th>TH3</th>
<th>TM1</th>
<th>TM2</th>
<th>TL1</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_{u,\text{exp}}$ (kN)</td>
<td>920</td>
<td>914</td>
<td>—</td>
<td>930</td>
<td>913</td>
<td>920</td>
</tr>
<tr>
<td>Ultimate Load $P_u$ according to EC3 (Class 3 cross-section, measured properties and $\gamma_M = 1.0$)</td>
<td>898</td>
<td>898</td>
<td>898</td>
<td>898</td>
<td>898</td>
<td>898</td>
</tr>
</tbody>
</table>

In Fig. 2-20, the curves show a nearly linear behaviour until first plastification occurs in the web ($P_{\text{el}} \approx 617$ kN); the behaviour is hardly more non-linear at a load level corresponding to $P_{u,\text{exp}}/1.4 \approx 660$ kN ("experimental" Serviceability Limit State).

At failure, a good ductility was obtained: the deflection at the end of test was about Span/175.

An interesting conclusion here concerns the possibility of neglecting the increase of the deflection due to the non-linear behaviour at SLS.

Concerning the ultimate load, results from Table 2-6 are in very good accordance with the ultimate load $P_u$ according to EC3, which is 898 kN (calculated for Class 3 cross-section, measured properties and $\gamma_M = 1.0$). It is a very positive result because, despite the yielded zone in the web and the presence of the crack, the tests resisted the calculated values from EC3.

2.1.6.2 Longitudinal fillet weld at web-to-flange junction tests – TGAP/TNOGAP tests

2.1.6.2.1 Transverse patch loading phase

Fig. 2-21 shows load-"stress" curves obtained for the average values from rosettes in the web (see Fig. 2-10, vertical stresses in web). On this figure, "stresses" are obtained by multiplying the measured deformation by the Young modulus, without accounting for yielding (it is an "hyper-elastic stress"). True stresses are valid only up to the yield strength of the material.

![Fig. 2-21: Average vertical “stress” in the web for TGAP/TNOGAP tests under patch loading](image-url)
Fig. 2-22 shows the web shortening versus load for the two tests.

From Fig. 2-21 and Fig. 2-22, the behaviour was more non-linear for the beam with gap (TGAP) than without gap (TNOGAP), letting supposed that the weld has been plastified as expected.

In fact, this is not so obvious because, from transversal cuttings carried out along the beam (TGAP-test) at the end of the fatigue test, it has been shown that the true cross-section of the gap is nearly constant along the beam and with reduced dimensions compared to nominal values (1 mm x 7 mm instead of 2 mm x 12 mm), see Photo 2-8. Besides, the actual throats were in accordance with the nominal ones. Therefore, although accurate specifications had been given to the steel fabricator, and although the fabrication of the girder was conducted carefully, the nominal cross-section of the gap has not been obtained. So, theoretically, the plastification of welds was not so effective than expected. In fact, the welds probably remained elastic. Besides this, the patch loading has probably been sufficient to allow locally significant plastification in the web: Fig. 2-21 shows an actual non-linear behaviour, especially for the TGAP Test. In spite of that, the remaining gap can be considered as a contact imperfection interesting from fatigue point of view for these tests.
2.1.6.2.2 Fatigue loading phase

Fig. 2-23 shows the loading histogram applied for TGAP Test. The loading frequency was 1.1 Hz.

Fig. 2-23: Loading histogram for fatigue loading on Test n°7 (with gap) – Frequency 1.1 Hz

For TGAP test, the crack occurred suddenly in the web-to-flange fillet weld just at-mid-span between the inspections at 230 920 and 234 760 cycles, without being detected before, See Photo 2-9. It had crossed both the web and the flange. So, the beginning of the crack is arbitrary fixed at \( N_{TGAP.exp} = 232,000 \) cycles. No ultimate resistance test was carried out for this girder because the sudden development of the crack.

Photo 2-9: Cracks for TGAP and deformation of the upper flange
Concerning TNOGAP test at 142 900 cycles, two cracks were detected at the same time in the fillet weld, the first at 10 cm from the mid-span and the second on the opposite side of the web, right at mid-span. During cycling, the first one (the main crack) grew and crossed the web and the flange. The second one did not develop (it stopped at 8 mm long). After the fatigue phase, the load was increased up to the failure of the girder. Photo 2-10 and Photo 2-11 show these cracks.

![Crack location for TNOGAP-Test](image)

**Photo 2-10**: Crack location for TNOGAP-Test

![Main crack for TNOGAP-Test](image)

**Photo 2-11**: Main crack for TNOGAP-Test

The main steps in the development of these cracks are given below:

<table>
<thead>
<tr>
<th>Fatigue events for TNOGAP Test</th>
<th>First detection of cracks</th>
<th>Web crossing for the main crack</th>
<th>Flange crossing for the main crack</th>
<th>End of cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nb of cycles</td>
<td>142 900</td>
<td>161 000</td>
<td>164 430</td>
<td>165 220</td>
</tr>
</tbody>
</table>

The comparison between the experimental results and the calculated life from EC3 (71 000 cycles, see Table 2-2) shows a good reserve in the fatigue life despite the presence of:

- i/ a 2mm nominal gap,
- ii/ transverse local yielding in the web and
- iii/ 20 mm of yielding zone in the web due to flexural normal stresses.

The location of the cracks (at mid-span) gives a good indication on the interaction between the local vertical yielding in the web and the shear resistance of the fillet weld and the possibility of neglecting this interaction.

From these tests, a main conclusion is that the behaviour of the longitudinal fillet weld at the web-to-flange junction in high shear zone for hybrid plate girders is not affected by the plastification in the web.


2.1.6.2.3 Ultimate loads

The crack in TGAP test occurred suddenly and no ultimate resistance test was carried out for this girder. For TNOGAP test, the load was increased, after the fatigue phase, incrementally until the ultimate resistance of the girder. From this test, the ultimate load was about $P_{u,exp} = 1130$ kN.; see Fig. 2-24.

![Load-deflection curve for fatigue loading and ultimate resistance (TNOGAP-Test)](image)

Fig. 2-24: Load-deflection curve for fatigue loading and ultimate resistance (TNOGAP-Test)

The failure was obtained by a large yielding at mid-span and opening of the crack. It is very interesting that the experimental load-deflection results of TNOGAP test confirm all the behavioural results from TH, TM and TL tests.

Concerning the ultimate load of the girder (TNOGAP), the experimental value was about $P_{u,exp} = 1130$ kN. This value is to be compared with the calculated value $1225$ kN (with measured yield strength, and plastification in the web and $\gamma_M=1.0$). The experimental load was affected by the multiple cracks in the web and the tension flange, which explains the high ductility found in this test.

2.1.6.3 Hysteresis loops stabilisation under large stress range – TOC Test

The loading histogram for TOC Test was conducted in 5 phases:
- pre-cycling for initialisation: $-100kN \sim +100kN$ (2 cycles),
- phase A: $-340kN \sim +340kN$ (15 cycles),
- phase B: $-400kN \sim +400kN$ (10 cycles),
- phase C: $-440kN \sim +440kN$ (7 cycles),
- ultimate resistance: $P_{u,exp} = 485kN$.

In fact, these values of $P$ were decided on site depending on the response of the girder observed during test. Only a starting value of $340$ kN corresponding to $\Delta \sigma = 1.5 f_{yt}$ was previously calculated. Each full cycle was carried out over about 20 to 30 minutes, with 2 minute wait at $(P)$ and $(–P)$ load levels. The variation of the deflection was about $60$ mm in phase B and C.

Fig. 2-25 represents the actual loading histogram for TOC Test. NO crack occurred during test.
Fig. 2-25: Actual loading histogram for TOC test

Fig. 2-26, Fig. 2-27 and Fig. 2-28 show the load-deflection and the load-stress loops obtained for Phases A, B and C and ultimate resistance phase. For Phase A, the first cycle, which can be distinguished from the others, is to be considered as an initialisation cycle.
Fig. 2-27: Phase B loops for TOC test

Fig. 2-28: Phase C loops and ultimate resistance curve
After the phase C, the load was increased incrementally until the ultimate capacity of the girder obtained by a plastic hinge at mid-span.

Table 2-7 summarised the main results obtained for TOC Test. In this table, $\Delta \sigma$ are elastic stress ranges calculated elastically from the load $P$ concerned and are divided by the nominal values of $f_y$.

<table>
<thead>
<tr>
<th>$P$ (kN)</th>
<th>Nb cycles performed</th>
<th>$f_{yt}$ nominal (460 MPa)</th>
<th>$f_{yw}$ nominal (235 MPa)</th>
<th>Stabilisation</th>
</tr>
</thead>
<tbody>
<tr>
<td>±340</td>
<td>15</td>
<td>1.47</td>
<td>1.61</td>
<td>2.88</td>
</tr>
<tr>
<td>±400</td>
<td>10</td>
<td>1.73</td>
<td>1.89</td>
<td>3.39</td>
</tr>
<tr>
<td>±440</td>
<td>7</td>
<td>1.90</td>
<td>2.08</td>
<td>3.72</td>
</tr>
</tbody>
</table>

From this table, it is obvious that the condition $\Delta \sigma < 1.5 \ f_{yw}$ is validated. However, a simple observation on the experimental results in Table 2-7 tends to confirm the possibility of adopting the condition $\Delta \sigma < 1.5 \ f_{yt}$.

Concerning the load-deflection behaviour, the TOC test results are in good accordance with TGAP/TNOGAP tests and TH/M/L tests results. Besides that, the ultimate load for TOC test after cycling phases was $P_{u,exp} = 485$ kN in comparison with a calculated value $P_{u,cal} = 441$ kN from EC3, based on plastic hinge at mid-span ($P_{u,exp} / P_{u,cal} = 1.1$, which is usual for stocky cross-sections).
2.2 Numerical investigations

2.2.1 The Boundary –Element-Program BEASY

2.2.1.1 The Dual Boundary Element Method

The program system BEASY was developed by Computational Mechanics BEASY in Southampton, UK.

It contains calculations on corrosion, fatigue and crack propagation examinations as well as it resolves mechanical and acoustical problems. This program can handle plane and three-dimensional problems by using the linear-elastic fracture mechanics method. For the simulation of the crack development of hybrid girders only three-dimensional methods have been used.

BEASY uses dual elements for 3D crack growth analysis. This means, that every crack consists of a front- and a backside and represent a split. Both sides are networked by boundary elements. Only one crack surface has to be generated. The second crack surface, the “backside”, is automatically overlaid with elements during the analysis. These elements are signified as dual elements. The dual boundary element method incorporates two independent boundary integral equations: the displacement equation applied at the collocation point on one of the crack surfaces and the traction equation on the other surface.

2.2.1.2 The calculation of 3D stress intensity factors

To describe the crack-tip stress field BEASY uses the stress intensity factors $K_I$, $K_{II}$ and $K_{III}$ as characteristic values. They are calculated using the crack opening displacement method.

When one point formula are employed, the Mode I, II and III stress intensity factors are evaluated. As shown in Fig. 2-29 there is $u$ the displacement of a point at the crack front. The indices $b$, $n$ and $t$ signifies the displacement directions normal to the surface ($b$), tangential to the crack front ($t$) and normal to the crack front tangent ($n$). $\theta = \pi$ and $\theta = -\pi$ denote upper and lower crack surfaces respectively.

![Crack surface](image)

Fig. 2-29: Calculation of SIF using crack opening displacement

2.2.1.3 The incremental direction and size of crack extension

The crack growth direction and size are calculated by minimum strain energy density criterion.

a) The incremental direction

The strain energy density factor $S$ is considered at a spherical three-dimensional surrounding the crack tip. The direction of crack extension is identified at the point, where $S$ assumes a minimum.

The theory is based on three hypotheses:
(1) The direction of the crack growth at any point along the crack front is toward the region with 
the minimum value of strain energy density factor $S$ as compared with other regions on the 
same spherical surface surrounding the point.

(2) Crack extension occurs when the strain energy density factor in the region determined by 
hypothesis (1), $S = S_{\text{min}}$, reaches a critical value, say $S_{\text{cr}}$.

(3) The length, $r_0$, of the initial crack extension is assumed to be proportional to $S_{\text{min}}$ such that 
$S_{\text{min}}/r_0$ remains constant along the new crack front.

The explicit expression of strain energy density around the crack front can be written as:

$$
\left( \frac{dW}{dV} \right) = \frac{S(\theta)}{r \cos \varphi} + O(1) \tag{2-1}
$$

where $S(\theta)$ is given by:

$$
S(\theta) = a_{11} \cdot K_i^2 + 2a_{12} \cdot K_i \cdot K_{ij} + a_{22} \cdot K_{ij}^2 + a_{33} \cdot K_{ij}^2 \tag{2-2}
$$

b) The incremental size

The incremental size is likewise determined by the strain energy density criterion.

It is stated in the hypothesis (3) of the strain energy density criterion that the length $r_0$ of the initial 
-crack extension is assumed to be proportional to $S_{\text{min}}$. Since the strain energy factor $S_{\text{min}}$ is 
proportional to the square power of the equivalent stress intensity factor $K_{\text{eq}}$, the incremental size at 
the crack front point under consideration is given by:

$$
\Delta a = \Delta a_{\text{max}} \left( \frac{S_{\text{min}}}{\min S_{\text{min}}} \right) = \Delta a_{\text{max}} \left( \frac{K_{\text{eq}}}{\max S_{\text{eq}}} \right)^2 \tag{2-3}
$$

BEASY chooses for $\Delta a_{\text{max}}$ the maximum distance from the crack front to the opposite side of the 
element containing the crack front as shown in Fig. 2-30. The crack propagation can be controlled by demand of the extended length $\Delta a_{\text{max}}$. This can be varied by editing of a command line at the input file file.dat. The value 1.0 must be modified, if the 
extended length shall be permitted.

The new incremental crack extension distance factor can be computed as following:

$$
incremental \text{ crack extension distance factor} = \left[ \frac{? a_{\text{max,req}}}{? a_{\text{max,old}}} \right] \tag{2-4}
$$

whereas $\Delta a_{\text{max,req}}$ is the desired incremental length and $\Delta a_{\text{max,old}}$ the existing length of the maximum 
element at the old crack front.

![Fig. 2-30: Maximum incremental distance](image-url)
2.2.1.4 **Semiautomatic crack propagation calculation**

The application of the semiautomatic method is for three-dimensional crack models with three-dimensional surface cracks contrary to internal cracks the only passable way. Here a crack is generated and a crack front is defined. Starting at this crack the points of the new crack front are calculated with the minimum strain energy density criterion. Therefore the stress intensity factors are determined by the opening displacement method for the meshpoints on the old crack front.

In order of the specification of the endpoints for the new crack front it is necessary to make sure that they do not lock always with the component upper edge but lie in the material inside or outside. They must be shifted for the generation of the new crack front on the material upper edge, whereby the x- and the y-coordinates of the calculated endpoints are maintained and only the z-coordinates are adapted (see Fig. 2-31):

![Initial Crack and Predicted Crack Front](image)

![Initial Crack and New Generated Crack](image)

*Fig. 2-31: Error in the prognosis of the new crack front with 3D surface cracks*

If the new crack front has adapted, the analysis can be started for the calculation of the next crack front and the stress intensity factors.

2.2.2 **Description of the tests**

The description of the test programme has been laid down in the Technical Reports of CTICM. In this working item the tests TL 1 and TM 2 have been simulated.

For the test TL 1 the forecast accuracy of the element program BEASY can be examined because this test has been performed purely in the linear-elastic range.

TM 2 has been simulated in order to determine the deviation of the crack development which occurs between the plastified sections in the test and the elastic calculation with BEASY.

From the load histograms of the tests TM 2 and TL 1 the number of cycles between the restlines have been obtained.

For each test the optical crack occurrence, the crack crossing through the flange and the web, the end of the test as well as the ultimate load $P_u$ up to the brittle fracture have been documented. The results are presented in Table 2-8 for the regarded tests TM 2 and TL 1:
Table 2-8: Summary of the test results TM 2 and TL 1

<table>
<thead>
<tr>
<th>Test</th>
<th>$P_u$ (kN)</th>
<th>Number of cycles ($10^6$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TM 2</td>
<td>913</td>
<td>Crack occurrence: 2.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Crack crossing the flange: 4.13</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Crack crossing the web: --</td>
</tr>
<tr>
<td></td>
<td></td>
<td>End of the test: 4.13</td>
</tr>
<tr>
<td>TL 1</td>
<td>920</td>
<td>Crack occurrence: 1.96</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Crack crossing the flange: 3.83</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Crack crossing the web: 3.79</td>
</tr>
<tr>
<td></td>
<td></td>
<td>End of the test: 3.83</td>
</tr>
</tbody>
</table>

The ultimate load $P_u$ according to EC 3 for Class 3 cross sections is about 898 kN, which is a good accordance to the test results.

Furthermore the test results have been compared with regard to the fatigue life of a notch detail of category 80 after Chapter 9 of EC 3 (see Fig. 2-33).

The notch detail of category 80 signifies a standard girder with a stiffener (see point 3 in Fig. 2-33). This diagram shows for each test the origin of the crack occurrence and the end of test. The test results are presented versus the design fatigue curve according to EC 3 (5% probability).

![Mechanical fracturing model for typical notch cases](image)

From these tests result the preliminary conclusion is that for a category of a detail 80 (the transverse welded attachment) the fatigue rules of EC 3 seem to be applicable to hybrid girders.
2.2.3 Evaluations of the tests

After the tests have been performed by CTICM, the area with the fatigue crack has been cut out from the total girder. The extract of the tests TM 2 and TL 1 have been sent to the RWTH Aachen. Fig. 2-34 shows the extract from the girder of the test TM 2.

In co-operation with the Institute of Ferrous Metallurgy of the RWTH Aachen these cracks have been opened. For this purpose the extract has been roughly sawn out around the crack and put afterwards in liquid nitrogen. The extract has been cooled down in liquid nitrogen until the metal achieved the temperature of the nitrogen, i.e. -196°C. Afterwards the cooled test specimen has been smashed with a hammer.

In the remaining cross section, where the crack did not divide the material, a brittle fracture has been occured while opening the specimen. Fig. 2-35 and Fig. 2-36 show the opened crack patterns of the tests TM 2 and TL 1.

In the crack pattern of TM 2 the sheer-lips in the lower area of the web are exposed which have a typical character of a plastified web. Such sheer-lips are not to be seen in the crack pattern of TL 1 because this test has been performed in the elastic range of the material.
In the specimen TL 1 the crack initiations are in both welded joints. However, the restlines are less apparent in the left area. Therefore, an evaluation in this area is impossible. The evaluation has been made by the programs Tracer13, Corel Draw Version 9.0 and with an electron microscope. The results are documented and averaged later on. The crack initiation for both tests has been in the welded joint between flange and web and grew downwards under an angle of 45° (see Fig. 2-37).

The crack is divided in two areas: On the one hand the crack in the web resp. in the welded joint between web and flange and on the other hand the crack in the flange. These values of crack depths and widths are needed for the input of the boundary element program BEASY.

The crack areas must be regarded separately, because the flange and the web consist of different materials. This procedure is specified on the annotation of the model generation.

To put marking lines and photos (see Fig. 2-37) on the scanned crack Corel Draw has been used to ensure that the orientation is always the same by using the different methods. During the evaluation with the electron microscope the same marking lines are recorded with pencil on the test specimens. The crack photo with the marking lines has been read in Tracer 13. To indicate the distance between the individual restlines a mark points has been set. The coordinates of the mark points can be used to calculate the distances. In the first check the same distances between the restlines on the marking line are measured with the measuring function in Corel Draw.

In the second check the measurement has been performed by means of the electron microscope. Afterwards, the measured values are tabulated and averaged. For the test TL 1 Table 2-9 and Table 2-10 show the determined values. Table 2-11 and Table 2-12 show the determined values for TM 2.
### Table 2-9: Evaluation of the restlines for the web for TL 1

<table>
<thead>
<tr>
<th>Crack depth $a_i$ [mm]</th>
<th>Tracer13</th>
<th>Corel Draw 9.0</th>
<th>Electron microscope</th>
<th>averaging</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a_0$</td>
<td>0.82</td>
<td>0.8</td>
<td>0.77</td>
<td>0.79</td>
</tr>
<tr>
<td>$a_1$</td>
<td>0.89</td>
<td>0.9</td>
<td>0.87</td>
<td>0.89</td>
</tr>
<tr>
<td>$a_2$</td>
<td>0.88</td>
<td>0.87</td>
<td>0.85</td>
<td>0.87</td>
</tr>
<tr>
<td>$a_3$</td>
<td>1.63</td>
<td>1.65</td>
<td>1.64</td>
<td>1.64</td>
</tr>
<tr>
<td>$a_4$</td>
<td>0.98</td>
<td>1.01</td>
<td>1.06</td>
<td>1.02</td>
</tr>
<tr>
<td>$a_5$</td>
<td>1.45</td>
<td>1.46</td>
<td>1.52</td>
<td>1.48</td>
</tr>
<tr>
<td>$S$</td>
<td>6.65</td>
<td>6.69</td>
<td>6.71</td>
<td>6.69</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Crack width $c_i$ [mm]</th>
<th>Tracer13</th>
<th>Corel Draw 9.0</th>
<th>Electron microscope</th>
<th>averaging</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c_0$</td>
<td>2.39</td>
<td>2.39</td>
<td>2.32</td>
<td>2.37</td>
</tr>
<tr>
<td>$c_1$</td>
<td>0.45</td>
<td>0.46</td>
<td>0.5</td>
<td>0.47</td>
</tr>
<tr>
<td>$c_2$</td>
<td>0.58</td>
<td>0.6</td>
<td>0.64</td>
<td>0.61</td>
</tr>
<tr>
<td>$c_3$</td>
<td>1.98</td>
<td>1.95</td>
<td>2.06</td>
<td>2</td>
</tr>
<tr>
<td>$c_4$</td>
<td>1.75</td>
<td>1.77</td>
<td>1.83</td>
<td>1.78</td>
</tr>
<tr>
<td>$c_5$</td>
<td>1.9</td>
<td>1.92</td>
<td>1.82</td>
<td>1.88</td>
</tr>
<tr>
<td>$S$</td>
<td>9.05</td>
<td>9.09</td>
<td>9.17</td>
<td>9.11</td>
</tr>
</tbody>
</table>

### Table 2-10: Evaluation of the restlines for the flange for TL 1

<table>
<thead>
<tr>
<th>Crack depth $a_i$ [mm]</th>
<th>Tracer13</th>
<th>Corel Draw 9.0</th>
<th>Electron microscope</th>
<th>averaging</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a_3$</td>
<td>1.13</td>
<td>1.15</td>
<td>1.25</td>
<td>1.18</td>
</tr>
<tr>
<td>$a_4$</td>
<td>0.99</td>
<td>0.95</td>
<td>0.96</td>
<td>0.97</td>
</tr>
<tr>
<td>$a_5$</td>
<td>1.69</td>
<td>1.71</td>
<td>1.75</td>
<td>1.72</td>
</tr>
<tr>
<td>$a_6$</td>
<td>1.71</td>
<td>1.74</td>
<td>1.79</td>
<td>1.75</td>
</tr>
<tr>
<td>$a_7$</td>
<td>3.15</td>
<td>3.09</td>
<td>3.11</td>
<td>3.12</td>
</tr>
<tr>
<td>$a_8$</td>
<td>2.27</td>
<td>2.22</td>
<td>2.23</td>
<td>2.24</td>
</tr>
<tr>
<td>$a_9$</td>
<td>4.02</td>
<td>4.16</td>
<td>4.25</td>
<td>4.14</td>
</tr>
<tr>
<td>$S$</td>
<td>14.96</td>
<td>15.02</td>
<td>15.34</td>
<td>15.12</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Crack width $c_i$ [mm]</th>
<th>Tracer13</th>
<th>Corel Draw 9.0</th>
<th>Electron microscope</th>
<th>averaging</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c_3$</td>
<td>2.72</td>
<td>2.77</td>
<td>2.98</td>
<td>2.82</td>
</tr>
<tr>
<td>$c_4$</td>
<td>2.34</td>
<td>2.31</td>
<td>2.4</td>
<td>2.35</td>
</tr>
<tr>
<td>$c_5$</td>
<td>1.24</td>
<td>1.09</td>
<td>0.98</td>
<td>1.11</td>
</tr>
<tr>
<td>$c_6$</td>
<td>1.96</td>
<td>1.92</td>
<td>1.74</td>
<td>1.87</td>
</tr>
<tr>
<td>$c_7$</td>
<td>3.54</td>
<td>3.76</td>
<td>3.8</td>
<td>3.7</td>
</tr>
<tr>
<td>$c_8$</td>
<td>2.74</td>
<td>2.69</td>
<td>2.95</td>
<td>2.8</td>
</tr>
<tr>
<td>$c_9$</td>
<td>3.46</td>
<td>3.65</td>
<td>3.45</td>
<td>3.52</td>
</tr>
<tr>
<td>$S$</td>
<td>18</td>
<td>18.19</td>
<td>18.3</td>
<td>18.16</td>
</tr>
</tbody>
</table>

### Table 2-11: Evaluation of the restlines for the web for TM 2

<table>
<thead>
<tr>
<th>Crack depth $a_i$ [mm]</th>
<th>Tracer 13</th>
<th>Corel Draw 9.0</th>
<th>Electron microscope</th>
<th>averaging</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a_0$</td>
<td>1.09</td>
<td>1.09</td>
<td>0.97</td>
<td>1.05</td>
</tr>
<tr>
<td>$a_1$</td>
<td>0.99</td>
<td>1.02</td>
<td>1.04</td>
<td>1.02</td>
</tr>
<tr>
<td>$a_2$</td>
<td>1.59</td>
<td>1.61</td>
<td>1.65</td>
<td>1.62</td>
</tr>
<tr>
<td>$a_3$</td>
<td>1.34</td>
<td>1.35</td>
<td>1.31</td>
<td>1.33</td>
</tr>
<tr>
<td>$S$</td>
<td>5.01</td>
<td>5.07</td>
<td>4.97</td>
<td>5.02</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Crack width $c_i$ [mm]</th>
<th>Tracer 13</th>
<th>Corel Draw 9.0</th>
<th>Electron microscope</th>
<th>averaging</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c_0$</td>
<td>1.99</td>
<td>2</td>
<td>1.92</td>
<td>1.97</td>
</tr>
<tr>
<td>$c_1$</td>
<td>1.07</td>
<td>1.03</td>
<td>1.09</td>
<td>1.06</td>
</tr>
<tr>
<td>$c_2$</td>
<td>1.42</td>
<td>1.4</td>
<td>1.24</td>
<td>1.35</td>
</tr>
<tr>
<td>$c_3$</td>
<td>2.42</td>
<td>2.39</td>
<td>2.47</td>
<td>2.43</td>
</tr>
<tr>
<td>$S$</td>
<td>6.9</td>
<td>6.82</td>
<td>6.72</td>
<td>6.84</td>
</tr>
</tbody>
</table>
The photogrammetric procedures are more precise. The results of the evaluations with Corel Draw 9.0 and Tracer 13 show a good accordance to the photogrammetric procedure. The evaluation by means of the electron microscope has a high optical quality which has the effect of wide restlines. Because the specimen cannot be clamped into a device, small movements of the specimen lead to big deviations. Therefore, a number of measurements have been performed to obtain a statistical average.

The number of cycles belonging to the crack propagations have been determined by the transmitted data from the CTICM. For the test TL 1 the numbers of cycles are shown in Table 2-13. The numbers of cycles for test TM 2 are shown in Table 2-14.

<table>
<thead>
<tr>
<th>Crack depth ai [mm]</th>
<th>Tracer 13</th>
<th>Corel Draw 9.0</th>
<th>Electron microscope</th>
<th>averaging</th>
</tr>
</thead>
<tbody>
<tr>
<td>a2</td>
<td>1.55</td>
<td>1.56</td>
<td>1.57</td>
<td>1.56</td>
</tr>
<tr>
<td>a3</td>
<td>1.01</td>
<td>1.02</td>
<td>0.95</td>
<td>0.99</td>
</tr>
<tr>
<td>a4</td>
<td>0.9</td>
<td>0.89</td>
<td>1.09</td>
<td>0.96</td>
</tr>
<tr>
<td>a5</td>
<td>0.92</td>
<td>0.93</td>
<td>1.32</td>
<td>1.06</td>
</tr>
<tr>
<td>a6</td>
<td>1.43</td>
<td>1.45</td>
<td>1.06</td>
<td>1.31</td>
</tr>
<tr>
<td>a7</td>
<td>1.39</td>
<td>1.39</td>
<td>1.39</td>
<td>1.39</td>
</tr>
<tr>
<td>a8</td>
<td>1.97</td>
<td>1.95</td>
<td>2.02</td>
<td>1.98</td>
</tr>
<tr>
<td>a9</td>
<td>4.86</td>
<td>4.89</td>
<td>5.17</td>
<td>4.97</td>
</tr>
<tr>
<td>S</td>
<td>14.03</td>
<td>14.08</td>
<td>14.57</td>
<td>14.23</td>
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</table>

<table>
<thead>
<tr>
<th>Crack width ci [mm]</th>
<th>Tracer 13</th>
<th>Corel Draw 9.0</th>
<th>Electron microscope</th>
<th>averaging</th>
</tr>
</thead>
<tbody>
<tr>
<td>c2</td>
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<td>2.81</td>
<td>3.15</td>
<td>2.91</td>
</tr>
<tr>
<td>c3</td>
<td>1.1</td>
<td>1.13</td>
<td>1.15</td>
<td>1.13</td>
</tr>
<tr>
<td>c4</td>
<td>1.3</td>
<td>1.32</td>
<td>1.32</td>
<td>1.31</td>
</tr>
<tr>
<td>c5</td>
<td>1.05</td>
<td>1.07</td>
<td>1.41</td>
<td>1.18</td>
</tr>
<tr>
<td>c6</td>
<td>2.77</td>
<td>2.77</td>
<td>2.86</td>
<td>2.8</td>
</tr>
<tr>
<td>c7</td>
<td>1.56</td>
<td>1.56</td>
<td>1.68</td>
<td>1.6</td>
</tr>
<tr>
<td>c8</td>
<td>3.59</td>
<td>3.59</td>
<td>3.52</td>
<td>3.57</td>
</tr>
<tr>
<td>c9</td>
<td>8.32</td>
<td>8.33</td>
<td>8.05</td>
<td>8.23</td>
</tr>
<tr>
<td>S</td>
<td>22.47</td>
<td>22.58</td>
<td>23.14</td>
<td>22.73</td>
</tr>
</tbody>
</table>

Table 2-12: Evaluation of the restlines for the flange for TM 2
### Table 2-13: Crack propagation according to the load histogram for TL 1

<table>
<thead>
<tr>
<th>Points (see Load-Histogram)</th>
<th>Load (kN)</th>
<th>Number of cycles N (10^6)</th>
<th>dN_i (10^6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S4 = Fatigue cycling start</td>
<td>450/250</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>S5 = a_0</td>
<td>449/250</td>
<td>735</td>
<td>0.735</td>
</tr>
<tr>
<td>S6 = a_1</td>
<td>450/250</td>
<td>1.21</td>
<td>0.475</td>
</tr>
<tr>
<td>S7 = a_2</td>
<td>450/0</td>
<td>1.522</td>
<td>0.312</td>
</tr>
<tr>
<td>S8 = a_3</td>
<td>450/250</td>
<td>1.895</td>
<td>0.373</td>
</tr>
<tr>
<td>S9 = a_4</td>
<td>450/250</td>
<td>2.247</td>
<td>0.352</td>
</tr>
<tr>
<td>S10 = a_5</td>
<td>450/250</td>
<td>2.614</td>
<td>0.367</td>
</tr>
<tr>
<td>S11 = a_6</td>
<td>450/0</td>
<td>2.982</td>
<td>0.368</td>
</tr>
<tr>
<td>S12 = a_7</td>
<td>450/250</td>
<td>3.326</td>
<td>0.344</td>
</tr>
<tr>
<td>S13 = a_8</td>
<td>450/250</td>
<td>3.686</td>
<td>0.36</td>
</tr>
<tr>
<td>S14 = a_9</td>
<td>450/0</td>
<td>3.828</td>
<td>0.142</td>
</tr>
</tbody>
</table>

### Table 2-14: Crack propagation according to the load histogram for TM 2

<table>
<thead>
<tr>
<th>Points (see Load-Histogram)</th>
<th>Load (kN)</th>
<th>Number of cycles N (10^6)</th>
<th>dN_i (10^6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S3 = Fatigue cycling start</td>
<td>720/520</td>
<td>4</td>
<td>0</td>
</tr>
<tr>
<td>S4 = a_0</td>
<td>720/520</td>
<td>706</td>
<td>666</td>
</tr>
<tr>
<td>S5 = a_1</td>
<td>719/519</td>
<td>1171</td>
<td>465</td>
</tr>
<tr>
<td>S6 = a_2</td>
<td>720/519</td>
<td>1536</td>
<td>365</td>
</tr>
<tr>
<td>S7 = a_3</td>
<td>720/519</td>
<td>1896</td>
<td>360</td>
</tr>
<tr>
<td>S8 = a_4</td>
<td>719/519</td>
<td>2257</td>
<td>361</td>
</tr>
<tr>
<td>S9 = a_5</td>
<td>720/0</td>
<td>2701</td>
<td>444</td>
</tr>
<tr>
<td>S10 = a_6</td>
<td>720/520</td>
<td>3080</td>
<td>379</td>
</tr>
<tr>
<td>S11 = a_7</td>
<td>720/520</td>
<td>3426</td>
<td>346</td>
</tr>
<tr>
<td>S12 = a_8</td>
<td>720/520</td>
<td>3762</td>
<td>336</td>
</tr>
<tr>
<td>S13 = a_9</td>
<td>719/0</td>
<td>4134</td>
<td>372</td>
</tr>
</tbody>
</table>
2.2.4 Generation of BE-models of the total girder and of selected extracts

2.2.4.1 Generation of the half total girder

The dimensions and the steel strength of the cross sections as well as the arrangement of the stiffeners are described in chapter 2 of this Report. The single girders have been designed according to Fig. 2-38.

Fig. 2-38: Static systems of the complete and half hybrid girder

By using the symmetry of the system only the half girder has been considered (Fig. 2-38).

The model has been divided into two zones in order to be able to consider the different material properties:

- Zone 1: flanges, which consist of a high-strength steel (S 460 M)
  
  Young’s modulus \( E = 222.990 \, \text{N/mm}^2 \), Poisson’s ratio \( \mu = 0.3 \)

- Zone 2: web, the stiffeners and the welded joints (S 235 JR)

  Young’s modulus \( E = 207.000 \, \text{N/mm}^2 \), Poisson’s ratio \( \mu = 0.3 \)

The sections where two zones are linked to each other, i.e. between web and flanges as well as between stiffeners and flanges, boundary surfaces have been created which are only once networked. These surfaces are called INTERFACE PATCHES and belong to one and to the other zone. For the one zone the normal vectors of these surfaces are outward arranged, for the other one inward. Fig. 2-39 shows the BE-model with its boundary conditions of supports and loads. On the left hand side the half girder is supported in z-direction, on the right hand side the moment is fixed. Additionally, the upper flange is held in the quarterly points in y-direction (lateral support). On the right hand side the half girder is loaded with a cyclic load of \( P/2 = 100 \, \text{kN} \). It has been distributed as a load per unit area on a small section. In BEASY no individual loads on nodes can be applied whereas displacements on nodes can be considered.

Fig. 2-39: Support and load at BE-model

Fig. 2-40. Initial crack at the BE-model
The crack has been generated at the overlap between the three welding joints web, flange and stiffener in the center of the total girder as half-elliptical surface crack (Fig. 2-39). The crack occurred in all tests in the same place. The initial crack has been set in the same dimensions as the first restline (Fig. 2-40). The BE-model of the half hybrid girder requires a very large storage capacity. A data check must be carried out before the actual analysis in order to check whether the used plate has enough storage space. These BE-models were calculated for the tests TL 1 and TM 2 with and without modification of the crack extension distance factor for the definition of the crack propagation. The simulations with demand of the crack propagation has been created in order to be able to illustrate exactly the crack propagation measured in the test. The simulations without demand of the crack propagation are to show whether realistic results can be obtained if no test results are present, from which the actual crack propagation can be determined.

### 2.2.4.2 Generation of the extracts from the total girder

With regard to less storage capacities for the simulations an extract has been selected around the significant section. In addition the following extract has been selected (Fig. 2-41).

![Fig. 2-41: Extract from the total girder](image)

With the finite element program MARC MENTAT a stress calculation of the total girder has been performed. The obtained deformations have been compared with the test data and showed a good accordance. The deformations of the edges from the extract have been taken over from the finite element model to BEASY. For the input of the displacements in BEASY the following items have to be considered:

1. the displacement should be input on patches- not on elements- in order to keep the quantity of data small
2. if the displacements of such surfaces are different and their generation direction is not well-known, it is possible to consider this in the INTERFACE menu LOAD+BCS \(^*\) PATCH_STRESS_B \(^*\) DISPLACEMENT.

Because of the model of the extract is smaller than the half total girder, the element net can generally be selected substantially more finely and so a higher accuracy will be obtained. Also the BE-models for the tests TL 1 and TM 2 with and without modification of the crack extension distance factor were calculated for the definition of the crack propagation.

![Fig. 2-42: Direction of rotation for input of displacements](image)
2.2.4.3 Evaluation of the results

The stress intensity factors are determined by the simulations. They are set into the Paris equation for the determination of the crack propagation.

\[
\frac{da}{dN} = C \Delta K^m
\]  

(2–5)

2.2.4.3.1 Range of validity

First it has to be checked whether the application of the Paris equation is allowed. The lower limit is defined over the threshold value \( K_{th} \). In the literature some equations are present for the calculation of \( K_{th} \), by which two are called in the following:

1. according to Schwalbe:
   \[
   \Delta K_{th} (R) = E (2,75 \pm 0,75) \cdot 10^{-5} (1 - R)^{0.31} \sqrt{1000} \quad \left[ N / mm^{3/2} \right]
   \]  

(2–6)

2. according to Barsom & Rolfe:
   \[
   \Delta K_{th} = 7 (1 - 0.85 R) \cdot \sqrt{1000} \quad \left[ N / mm^{3/2} \right]
   \]  

(2–7)

It is not well-known, for which materials these formulas have been developed, however this estimation shows the influence of the stress ratio on \( \Delta K_{th} \).

For the tests TL 1 and TM 2 the stress ratios are:

TL 1: \( R = 250 / 450 = 0,6 \) 
TM 2: \( R = 520 / 720 = 0,7 \)

According to the literature the threshold value \( \Delta K_{th} \) for such a stress ratio is about 65-70 N/mm3/2. The values from the simulations, calculated by BEASY, have been about \( \Delta K = 140 \) N/mm3/2 (and higher values).

The upper limit forms the fracture toughness \( K_{IC} \). If this value is achieved, crack failure occurs.

In the literature values for structural steel of 1000 - 4000 N/mm3/2 are mentioned but have never occurred in the calculations. Therefore, the range of validity is in the stable crack growth section; the Paris equation is applicable.

2.2.4.3.2 Selection of the constants \( C \) and \( m \)

Problems during the evaluation occurred mainly due to the fact that no measured values for the material constants of \( C \) and \( m \) from the Paris equation for steel S235JR were present. For steel S460M values from the IEHK Aachen were present from another research project. Usually these material constants are determined at CT-Samples (Compact Tension). These samples are manufactured according to ASTM E 647-regulation. The ASTM represents a standard test method for crack propagation measurements with constant load amplitude. The crack length can be measured optically or with the direct current potential method.

Fig. 2-43 shows a CT-sample and the principle of the direct current potential method.

In the tests of CTICM no values for the constants of \( m \) and \( C \) have been determined. Therefore, different values for \( m \) have been taken from the literature and \( C \) has been determined over the Gurney relationship. The values in Table 2-15 and Table 2-16 were consulted for the individual evaluations.
**Fig. 2-43: CT-sample and principle of the current potential method**

**Table 2-15: Selection of the constants m and C for crack in the web**

<table>
<thead>
<tr>
<th>Crack in the welded joint/web</th>
<th>Literature</th>
</tr>
</thead>
<tbody>
<tr>
<td>m</td>
<td>C</td>
</tr>
<tr>
<td>2.25</td>
<td>3.00*10^{-11}</td>
</tr>
<tr>
<td>2</td>
<td>1.64*10^{-10}</td>
</tr>
<tr>
<td>3</td>
<td>1.83*10^{-13}</td>
</tr>
<tr>
<td>2.13</td>
<td>6.78*10^{-11}</td>
</tr>
</tbody>
</table>

**Table 2-16: Selection of the constants m and C for crack in the flange**

<table>
<thead>
<tr>
<th>Crack in the flange</th>
<th>Literature</th>
</tr>
</thead>
<tbody>
<tr>
<td>m</td>
<td>C</td>
</tr>
<tr>
<td>2.5</td>
<td>5.48*10^{-12}</td>
</tr>
<tr>
<td>1.92</td>
<td>* 1.29*10^{-10}</td>
</tr>
<tr>
<td>2.36</td>
<td>* 1.17*10^{-11}</td>
</tr>
<tr>
<td>2.43</td>
<td>* 6.36*10^{-12}</td>
</tr>
<tr>
<td>2.46</td>
<td>* 4.51*10^{-12}</td>
</tr>
<tr>
<td>2.54</td>
<td>* 2.80*10^{-12}</td>
</tr>
<tr>
<td>2.66</td>
<td>* 2.22*10^{-12}</td>
</tr>
<tr>
<td>2.73</td>
<td>* 1.05*10^{-12}</td>
</tr>
<tr>
<td>2.75</td>
<td>* 1.05*10^{-12}</td>
</tr>
<tr>
<td>2.83</td>
<td>* 7.02*10^{-13}</td>
</tr>
<tr>
<td>2.85</td>
<td>* 5.15*10^{-13}</td>
</tr>
<tr>
<td>3.01</td>
<td>* 1.82*10^{-13}</td>
</tr>
<tr>
<td>3.09</td>
<td>* 9.23*10^{-14}</td>
</tr>
<tr>
<td>2.65</td>
<td>1.98*10^{-12}</td>
</tr>
</tbody>
</table>

* Experimentally determined values of the IEHK Aachen
2.2.4.3.3 Comparison of the results

- Test TL 1 (given crack growth)

Crack in the web:

The simulations show a good accordance with the test results by using the constants $m = 2.25$ and $C = 3.00 \times 10^{-11}$. The deviation after the second restline can be explained by the fact that with the crack propagation the element net grows, whereby the necessary storage capacity increases. For the simulated half girder this effect of increasing storage capacity is more significant than for the simulation of the extract. If the net achieves the limits of the available capacity, the element net must be arranged subsequently rougher in order to save storage capacity. Therefore, the extract of the girder converges more and more over the number of cycles with the test results. Investigations showed that the quality of the results depends strongly on the refinement of the element net. Furthermore the calculation has been performed with $m = 2$, the lower limit indicated in the literature and with $m = 3$, the recommendation of the EC 3, appendix C.

![Fig. 2-44: Evaluation for the test TL1, crack in the web](image)

The results for $m = 2$ shows deviations from approx. 20-30% on the safe side for the extract and the half girder. The calculation with the value $m = 3$ points out the large influence of the exponent. In comparison to the test, the simulation showed three times more numbers of cycles for the same crack development.

Crack in the flange:

The calculation has been performed with the values between $m = 1.92$ up to $m = 3.09$ taken from former investigations with the steel S460M. The experimentally determined values for $C$ and $m$ show that these constants include a large dispersion. The steel with the indication M behaves substantially more toughly and, therefore, better concerning the crack propagation than a normal steel with the indication JR. That leads to the acceptance of a larger $m$-value for the material of the flange (S460M) and, therefore, a higher number of cycles (longer fatigue life). By using a range of constants the figure 4.9 shows that some have a good accordance to the test while a small deviation of the constants has an enormous influence on the calculation.
Additionally, the experimental determined values were analysed statistically. The standard deviation has been determined on the assumption of an unknown expectancy value \( \mu \) to \( s = 0.3214 \). Because the standard deviation is very small, it can be assumed that the arithmetical mean is \( x = 2.65 \). If the arithmetical mean for \( m \) is taken to the experimental determined value \( m = 2.65 \), \( C \) can be calculated with the Gurney relationship \( C = \frac{1.315 \times 10^{-4}}{895.4 m} \). With the Paris equation, a very good accordance for the extract has been reached. The deviation is only at approx. 3%.

For the half girders the results deviates with the increasing of the crack propagation.

- **Test TL 1** (without given crack growth):
  
The same simulations have been performed without information about the crack propagation (number and distances of restlines). By using the values \( m = 2.25 \) for the web and the average value \( m = 2.65 \) for the flange the extract shows better results than the half girder.

- **Test TM 2** (given crack growth):

  **Crack in the web:**

  Due to the fact that BEASY is not able to calculate plastic problems the constants have, the plastified web has to be suited to the brittle material behaviour. With \( m = 2.13 \) a good accordance of the extract to the test results have been achieved, the deviation was lower than 1%.

---

**Fig. 2-45: Evaluation for the test TL1, crack in the flange**

**Fig. 2-46: Evaluation for the test TM2, crack in the web**
Therefore, by using adapted material constants the crack development of a small plastified area can be simulated with BEASY, although the numerical analyses is performed by using the linear-elastic fracture mechanics method. It could be useful to simulate further tests with small plastified areas to check, whether a general accepted factor can be introduced to transfer the constants for elastic to plastified calculations.

Crack in the flange:

By using the averaged value \( m = 2.65 \) a very good accordance for the extract with the tests has been reached. The calculation for the half girder continues with increasing of the crack, which is once again the problem of storage capacity.

- **Test TM 2** (without given crack growth):

  The same simulations have been again performed without information about the crack propagation (number and distances of restlines). For the web and the flange the results for the extract show a good accordance to the test results.

\[ \begin{array}{cccccccc}
\text{BEASY extract} & \text{Test} \\
\text{m} = 1.92 & \text{m} = 2.36 & \text{m} = 2.43 & \text{m} = 2.46 & \text{m} = 2.54 & \text{m} = 2.66 & \text{m} = 2.73 & \text{m} = 2.75 & \text{m} = 2.83 & \text{m} = 2.85 & \text{m} = 3.01 & \text{m} = 3.09
\end{array} \]

Fig. 2-47: Evaluation for the test TM2, crack in the flange

### 2.2.5 Analysis of sensibility

The structural component on which fatigue loads are applied passes through two phases up to the fracture:

1. the phase of the crack initiation from the crack free initial state
2. the phase of stable crack growth up to the fracture

The life span of a component which is under swinging load consists therefore of the initial crack phase and the stable crack-growing phase up to the fracture.

The life span up to the initial technical crack is covered by the S/N-curve (Wöhler- concept). After this initial technical crack has occurred the fracture mechanic methods is applied.
Mr. Jae-Byung Jo investigated in the combination and transformation of the notch-root concept and the fracture mechanics and obtained in general good results. Due to the tests executed by the CTICM have been performed without an initial swung crack but driven as a simple S/N- test without well-known starting damage, it is not assignable, at which crack depth a fracture mechanics method can be applied.

The first initial crack in the simulation was situated in the area of the well-known first restline, obtained from the opening of the test. This first restline occurred for the test TL 1 after 735,000 numbers of cycles and for the test TM 2 after 666,000 numbers of cycles. Therefore, these technical initial cracks have occurred already before these number of cycles. The target of this sensibility study is to check the influence of different initial crack lengths which have been taken from the literature.

2.2.5.1 Selection of initial crack lengths

The appendix C of the EC 3 is using an half elliptical surface crack as the initial crack. The crack depth is calculated according to the formula $a_0 = 0.5 \ln \left( \frac{t}{t_0} \right)$, the crack width according to the formula $2c_0 = 5a_0$. The variable $t$ is for the regarded plate thickness and the reference plate thickness $t_0$ amounts to 1 mm. The EC3 assumes that such a crack can be detected in the thorough quality-assurance nondestructive test measures for fatigue-stressed structures. However these calculated crack dimensions for the tests TL 1 and TM 2 are more largely than the dimensions of the first restlines and therefore unsuitable for the analysis of sensibility.

According to J. Bild an initial crack depth is indicated for $a_0 = 0.1$ mm and an initial crack width for $c_0 = 0.13$ mm.

S. Sähn, H. Göldner and J.-B. Jo set $a_0 = 0.25$ mm as initial crack depth for structural steel. The initial crack width $c_0$ is calculated over in the appendix C of the EC 3 specified relation $a_0/c_0 = 0.4$.

D. Radaj specifies the initial crack depth to $a_0 = 0.5$ mm and the initial crack width $c_0 = 1$ mm, with an half elliptical surface crack.
Table 2-17: Selection of the to scrutinizing initial crack depths

<table>
<thead>
<tr>
<th>Literature</th>
<th>(a_0) [mm]</th>
<th>(c_0) [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>J. Bild</td>
<td>0.1</td>
<td>0.13</td>
</tr>
<tr>
<td>Sähn, Göldner, Jo</td>
<td>0.25</td>
<td>0.63</td>
</tr>
<tr>
<td>Radaj</td>
<td>0.5</td>
<td>1</td>
</tr>
</tbody>
</table>

The sensibility study has been executed with the BE-models for the extracts because of their results showed the best accordance with the tests. Additionally, the computing duration and the necessary storage capacity have to be limited, which is ensured by these models.

2.2.5.2 Interpretation of the results

2.2.5.2.1 Test TL1

The Fig. 2-49 shows the crack depth and the number of cycles \(N\), which were calculated with the initial crack depth as \(a_0 = 0.1\) mm up to the first restline. Approx. 1 million numbers of cycles have been achieved. This exceeds the number of cycles (735,000) obtained in the test. For the individual phases the crack growth stress intensity factors has been calculated, which were situated in the area of the threshold value \(K_{th}\) (below \(K_{th}\) no more crack propagation take place). Therefore, it cannot be presumed that this simulation takes place in the phase of a stable crack growth. The using of these dimensions as the initial crack is unsuitable.

![Fig. 2-49: Calculated number of cycles for the initial crack depth \(a_0 = 0.1\) mm](image)

Fig. 2-50 shows the calculated number of cycles \(N\) for an initial crack depth of \(a_0 = 0.25\) mm. Approx. 600,000 numbers of cycles have been calculated.

By taking the assumption that a component is almost intact at the beginning of test (and therefore a number of cycles must to be driven until the initial crack occurs) this result seems to be realistic.

Fig. 2-51 shows the calculated number of cycles \(N\) with an assumed initial crack with the depth of \(a_0 = 0.5\) mm. To achieve the first restline only approx. 220,000 numbers of cycles have been neccessary. This crack depth seems to be too inaccurate, due to the fact that no specification concerning crack history can be given, which had occurred before this initial crack size.
2.2.5.2.2 Test TM2

For the test TM 2 a higher number of cycles of approx. 2.4 millions for the initial crack depth of \( a_0 = 0.1 \) mm has been calculated (see Fig. 2-52). The determined numbers of cycles result from the calculated stress intensity factors, which has been determined too far below of the threshold value \( K_{th} \). Therefore, the initial crack depth according to J. Bild is also unsuitable for this test.

An initial crack with the depth of \( a_0 = 0.25 \) mm supplies the best results (see Fig. 2-53). Approx. 620,000 numbers of cycles have been calculated up to achieve the first restline. This number is nearly the value of 666,000 numbers of cycles from the test. Due to the web plastified in this test in contrast to test TL 1, it can be assumed that a lower number of cycles up to achieving the first restline must be driven, because of the material behaves by modification of the lattice structure during the plastification substantially more inflexible than the purely flexible material of the test TL 1.

The assumption of \( a_0 = 0.5 \) mm presents similar values for the number of cycles \( N \) as the test TL 1. Fig. 2-54 clarifies that this assumption does not permit an exact predicate.
Fig. 2-52: Calculated number of cycles for the initial crack depth $a_0 = 0.1 \text{ mm}$

Fig. 2-53: Calculated number of cycles for the initial crack depth $a_0 = 0.25 \text{ mm}$

Fig. 2-54: Calculated number of cycles for the initial crack depth $a_0 = 0.5 \text{ mm}$
2.2.6 Calculation of the crack propagation with well-known manual formulas at simplified back-up models

In the context of this work it has been examined whether the specified manual formulas for the determination of the stress intensity factor (SIF) of the appendix C of the Eurocode 3 are applicable for these kind of test specimens. In the appendix C of the EC 3 the manual formulas for the correcting functions Y and Mk from Raju-Newman, Murakami, Hobbacher, Fischer and Zettelmoyer were developed for simple geometries.

2.2.6.1 Selection of the manual formulas

The model of a double-T-girder with welded stiffeners is not specified in the appendix C of the EC 3 neither in the Stress Intensity Factor Handbook, volume I+II. Therefore this model has to be highly simplified, in order to be able to apply the available formulas. The codes assume that for welded components the weld itself tears at last and only the transitions between welded joint and base material, which are called heat influence zones, represent endangered areas concerning the crack propagation.

Therefore in the literature only formulas for cracks before welded joints are specified.

2.2.6.1.1 Manual formula for the crack at the welded joint

However the tests performed in this research project shows that the initial cracks occure always at the transition of two welded joints, as shown in fig. 3.4. Due to the fact that for this position of the initial cracks no manual formulas are given, an imaginary plate has been put into the welded joint to assume the material of the welded joint as the base material (see Fig. 2-55). For this modified model manual formulas for the simple plate with half elliptical surface crack, developed by Raju Newman, have been used. The dimensions of the imaginary plate have been selected in such a way that the range of validity for the formulas are kept. The formulas for the calculation of the correction function Y for the simple plate with a surface crack are to be found in the appendix C of the EC 3, part 2.

![Fig. 2-55: Imaginary plate for the crack at the welded joint](image_url)
2.2.6.1.2 Manual formula for the crack at the flange

For the case of a crack in the flange the model of the plate with welded stiffener by Hobbacher has been selected, due to this model is the best approximation to the problem.

2.2.6.2 Placing of stresses

The manual formulas specified in the appendix C of the EC 3 are based on the placing of flexible rated stresses. For plastic areas or notch root stresses these formulas are not applicable. Therefore the flexible stresses from the finite element calculation with MARC MENTAT have been used.

The stresses for the individual crack depths were determined by interpolation from the linear stress gradient for the individual girders.

![Linear stress gradient for the test TL 1](image)
2.2.6.3 Calculation and evaluation

2.2.6.3.1 Imaginary plate for the crack at the welded joint

The manual formulas have been used to calculate the correcting function \( Y \), which determines the stress intensity factor \( \Delta K_I \). The SIF has been inserted into the Paris equation. By using equation (2–8) the number of cycles \( dN \) according to the individual crack propagation or crack depths can be determined.

\[
d a = dN \cdot C \cdot \Delta K_I^{m}
\]  

(2–9)

The calculations have been performed with the following different numbers of cycles, in order to be able to proof the sensibility with regard to different load steps:

**Table 2-18: Different numbers of cycles for the calculation of the crack depths**

<table>
<thead>
<tr>
<th>Notation</th>
<th>Number of cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Appendix C, EC 3</td>
<td>100000</td>
</tr>
<tr>
<td>Appendix C, EC 3, Test 1</td>
<td>50000</td>
</tr>
<tr>
<td>Appendix C, EC 3, Test 2</td>
<td>10000</td>
</tr>
</tbody>
</table>

The results of the calculations are summarised in a special appendix (not included in this report). These results show that the manual formulas are too safe sided in contrast to the BE-simulation and the tests, which leads to uneconomic solutions (see Fig. 2-59).

It has to be mentioned that a selection of smaller number of cycles leads to more exact results than a larger number of cycles as used in this calculation.

Additionally the geometry data of the imaginary plate has been modified. Different plate thickness and -widths were selected. However these modifications had no influence on the results.

Due to the fact that the results of the manual formulas do not converge with the BE-simulation and the tests, it can be concluded that the assumption of the imaginary plate for the crack in the welded joint represents a too rough simplification. Therefore new manual formulas for cracks in welded joints would have to be developed.
2.2.6.3.2 Plate with welded stiffener for the crack at the flange

Additionally the correcting function $M_k$ has been used to consider the influences of welded details. With these correcting functions the stress intensity factor $\Delta K_I$ for the model of the plate with stiffener has been calculated. The evaluation of the results has been performed with different numbers of cycles (see Table 2-18). The crack depths or widths have been calculated according to formula (2–10).

The results of this calculation are also safe sided. However these results show a better accordance with the test results and the BE-simulations than those of the manual formulas by using the imaginary plate.

![Graph showing comparison of crack depth vs. number of cycles for different conditions.](image)

Fig. 2-60: Comparison with the manual formulas for test TL 1, crack at the flange

2.2.7 Summary

The crack propagation of the hybrid girders in the working item 1.1 has been simulated with the BE-Program BEASY. The numbers of cycles were assigned to the individual restlines and therefore to the individual crack growth steps from which the crack propagation $da/dN$ has been determined.

The evaluation of the crack patterns have been performed by photogrammetric procedures (Corel Draw 9.0 and Tracer) and by a light electron microscope. The photogrammetric procedures have been considered as more precisely. Additionally these procedures can be applicable more easily.

The simulations of the crack propagation have been performed by using the symmetry of a half girder and an extract due to saving of storage capacity. The results show that the crack propagation curves of the extracts with the Paris constants of $m = 2.25$ for the web and $m = 2.65$ for the flange have a very good accordance to the test results. The results of the simulated half girders indicate a larger deviation in all cases with increasing crack propagation.

The reason for this deviation is the modification of the element net while generating the new crack zone. The element net must be arranged rougher with each extension step in order to reduce the increasing necessary storage requirements.

The selection of the initial crack size has been performed in the frame of the sensibility study. The consideration of the initial crack depth of $a_0 = 0.25$ mm showed the best accordance in all tests.

The calculations of the crack propagation with well-known manual formulas from the appendix C of the EC 3, part 2 at simplified models show results which are safe sided. However, the manual formulas deviate from the tests and simulation results.
2.1 Welding Procedures of the test specimens

2.1.1 Welding Procedure of test specimens 1 to 9

For web-to-flange joints, the welds were realised by submerged arc couple Lincoln L61-780 furnished by LINCOLN ELECTRIC. The characteristics of the weld metal were:

Yield strength > 450 MPa,
Tensile strength > 550 MPa,
Composition: C 0.07, Mn 1.4, Si 0.6, P < 0.025 and S < 0.02,
Impact ISO-V: +20°: 80 J, 0°: 60 J, -20°: 40 J.

The other welds were semi automatic ones with wire under gas type OSMC 710 from LINCOLN. In this case the characteristics were:

Yield strength > 480 MPa,
Tensile strength > 550 MPa,
Composition: C 0.05, Mn 1.5, Si 0.75, P 0.015 and S 0.02,

The welding procedure was defined in relation with the weld throat dimensions. In this project, the main value of the weld throat dimension being 4 to 5 mm, one welding pass was sufficient for the execution of such welds. For weld throat dimensions greater than 5 mm, two welding passes were used for weld execution.

2.1.2 Welding details of test specimen 1 to 6

The main detail concerned the web-to-flange junctions. One welding pass on each side was used for the top and bottom web-to-flange junctions.
2.1.3 Welding details of test specimen 7

For this specimen, 3 welding passes on each side were used for the weld execution.

![Weld For TEST N° 7 diagram]

2.1.4 Welding details of test specimen 8

The specific aspect in this test was the need of 2 mm web-to-flange gap. In order to obtain this value the welding procedure explained in the figure below was adopted.

![Weld For TEST 8 diagram]

Welding Procedure
For the bottom flange in tension side

- Adjustement of positioning plates 2 mm thick each 300 mm
- Spot positioning welds
- Remove positioning plates
- Fillet weld with \( a = 4 \) mm (automatic or manual)

Web
2 mm
Flange

Spot positioning welds @ 300 mm

weld preparation for the bottom flange longitudinal view

2mm gap
2.1.5 Welding details of test specimen 9

For this specimen, weld throat dimensions were 6 mm on each side of the two web-to-flange junctions. According to 2.1.1, two welding passes were used on each side (in total 8 passes for the beam welding).
2.3 Conclusions

2.3.1 General conclusions from experimental investigations

This large experimental program on hybrid plate girders allows to answer, within the tested domain, questions raised at the beginning of the ECSC project. This domain concerns especially a nominal ratio \( f_{yf}/f_{yw} < 2 \) (actual ratio for tests \( \approx 1.65 \)) and rather small web slenderness (84 and 27).

The main conclusions are:

I/ For fatigue design of hybrid plate girders, it is possible to apply the fatigue rules developed in EC3 for homogeneous girders. The effect of the yielded zone in the web on the fatigue resistance may be disregarded;

II/ The fatigue condition for longitudinal normal stress range \( \Delta \sigma < 1.5 f_{yf} \) should apply for hybrid plate girders;

III/ Beyond the first yielding in the web, the behaviour of hybrid plate girders becomes nonlinear due to the progressive extent of the plastified zone in the web. But the increase in the deflection at the serviceability limit state, compared to an elastic calculation, may be neglected;

IV/ No additional or particular rules need be developed for hybrid plate girders concerning the interaction between longitudinal shear and a transverse patch loading producing a yielding in the web or in the fillet weld. The previous rules for homogeneous girders may be applied;

V/ No additional rules need be developed for the execution of the fillet weld at the web-to-flange junction in hybrid plates girders. Tolerances defined for homogeneous girders may be applied. The welding material may match the web material.

2.3.2 General conclusions from numerical investigations

Considering all performed Boundary-Element-Simulations the following points can be summarised:

- The results of the simulations by using extracts have a very good conformity to the test results. Although the expenditure of computation appears larger, because of using a finite element program for the stresses and displacements, this model offers significant advantages. The generation is more clearly and the required storage capacity of computation is substantially smaller than using a half girder and supplies by the possibility of the finer selection of the element net significant more exact results.

- Since no experimental determined values for the constants \( C \) and \( m \) of the Paris equation are available, it is difficult to determine the crack propagation \( da/dN \) exactly. It is necessary to execute CT-tests in order to determine at least the constant \( m \) for all structural steels. These experimental determined values should be analysed statistically and summarised in a standard, since they represent important instruments for the determination of the crack propagation. The constant \( C \) can be determined e.g. over the Gurney relationship as a function of \( m \). This function has been confirmed in a number of former investigations.

- For the evaluation of the crack propagation in small plastified zones further tests have to be simulated in order to be able to determine a factor for the material constants by using a linear elastic fracture mechanics methods.

- By using BE-programs for calculating the crack propagation it is possible to develop new manual formulas for complex spatial design details and therefore to extend fatigue strength catalogues already existing.
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3 Push Out Tests with Groups of Studs

3.1 Experimental investigations

3.1.1 General

The use of prefabricated concrete elements can improve durability and economy for composite bridges with small and medium spans. The main objective of the following research is the improvement of shear connection with regard to

- the use of partially prefabricated concrete decks or precast elements in combination with headed studs
- the static and fatigue strength of studs with a shank diameter of 25mm
- the use of bolted shear connection for composite bridges with regard to an exchange of the concrete deck during design life or for temporary bridges.

This research includes new static and fatigue push-out tests. Additionally test data in the literature will be analysed in order to control the design rules for the static and fatigue strength of headed studs in Eurocode 4 Part 2 [1]. Table 3-1 shows the program for the new tests described in detail in 3.1.3.

Table 3-1: Test program

<table>
<thead>
<tr>
<th>Test</th>
<th>Type of test specimen</th>
<th>Total number of tests</th>
<th>Static tests</th>
<th>Fatigue tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ia</td>
<td>Headed studs ∅ 25mm and bolts M20</td>
<td>3</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Ia</td>
<td>High strength bolts M20</td>
<td>3</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Ii</td>
<td>Partially prefabricated slabs and studs ∅ 25mm</td>
<td>6</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Iii</td>
<td>Prefabricated slabs and studs ∅ 25mm</td>
<td>6</td>
<td>3</td>
<td>3</td>
</tr>
</tbody>
</table>

3.1.2 Detailing of shear connection in case of prefabricated slabs

The objective of type (Ia) shear connectors according to Table 3-1 and Fig. 3-1 is that the concrete slab can be easily separated from the steel girder. The shear connectors with a shank diameter of 25mm are welded on a steel plate, which is connected to the top flange of the steel girder by high strength bolts (Fig. 3-1 a). In this case, the heads of the bolts and the studs connectors act together and produce a combined longitudinal shear resistance. For type (Ib) shear connection in Fig. 3-1 high strength bolts are used. The bolt and the nuts on the top side act like studs in concrete and transmit the shear force through the thread into the steel profile. The corresponding tests series (Ia) and (Ib) are only basic tests to study the principle behaviour of this new type of shear connection. The number of tests performed is not sufficient to develop detailed design rules.
Type II shear connection in Fig. 3-1 represents an often built solution with partially prefabricated slabs which haven’t been tested before. Several bridges have been built this way without knowing the fatigue behaviour of the studs. The fatigue strength of headed studs is significantly influenced by the quality of the concrete in the region of the weld collar.

In case of partially prefabricated slabs the distance between the stud and the prefabricated concrete element can be very small and an insufficient compaction of the concrete can influence the static and fatigue strength. Furthermore, the bottom layer of the transverse reinforcement is arranged above the partially prefabricated element and is not so effective in the regions of the weld collars of the studs as in solid concrete slabs. In order to get more information additional type-II tests are planned.

The type III shear connectors cover the use of precast slabs. This construction type is preferred in the Scandinavian countries because the solid slab can be casted in a precasting plant and the shear connection will be made in situ and, if necessary, under heating tents. The corresponding tests were provide more information about the static and fatigue behaviour of this type of shear connection with regard to the use of high strength mortar in combination with the precast elements. For this type of shear connection normally studs arranged in groups are used. The use of headed studs with shank diameters of 25mm leads to several advantages.

![Fig. 3-1: Different possibilities of detailing of shear connection](image)

### 3.1.3 Test program and test results

#### 3.1.3.1 Test specimens and preparation of tests

Detailing of the three types of test specimens are shown in Fig. 3-2, Fig. 3-3, Fig. 3-4 and Fig. 3-5. Each concrete slab was cast in horizontal position as it is done for slabs of composite bridges. Therefore steel profiles were cut and welded together after hardening of concrete (Fig. 3-4). Reinforcement was designed in accordance with Eurocode 4.
Fig. 3-2: Test specimen type Ia

Fig. 3-3: Test specimen type Ib
Fig. 3-4: Test specimen type II

Fig. 3-5: Test specimen type III
3.1.3.2 Material Properties

3.1.3.2.1 Concrete

During the tests the compressive strength and the modulus of elasticity of the concrete had been determined by testing of cylinders Ø 150mm x 300 mm. Cylinders were stored 28 days in a wet environment according EN 206 and, additionally cylinders were stored adjacent to the test specimen. Table 3-2 gives the measured values of the compressive strength and the modulus of elasticity of the concrete cylinders for the different storage of specimens.

<table>
<thead>
<tr>
<th>Type of test specimen</th>
<th>Type of loading</th>
<th>No.</th>
<th>Cylinder strength $f_c$ (28 days) [N/mm²]</th>
<th>Modulus of elasticity $E_c$ (28 days) [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>wet environment</td>
<td>adjacent to the test specimens</td>
</tr>
<tr>
<td>Ia</td>
<td>static</td>
<td>Ia/1</td>
<td>49,1</td>
<td>38,5</td>
</tr>
<tr>
<td></td>
<td>static</td>
<td>Ia/2</td>
<td>38,5</td>
<td>34520</td>
</tr>
<tr>
<td></td>
<td>fatigue</td>
<td>Ia/3</td>
<td>49,1</td>
<td>38,5</td>
</tr>
<tr>
<td>Ib</td>
<td>static</td>
<td>Ib/1</td>
<td>46,9</td>
<td>32,9</td>
</tr>
<tr>
<td></td>
<td>static</td>
<td>Ib/2</td>
<td>32,9</td>
<td>34260</td>
</tr>
<tr>
<td></td>
<td>fatigue</td>
<td>Ib/3</td>
<td>46,9</td>
<td>32,9</td>
</tr>
<tr>
<td>II</td>
<td>static</td>
<td>II/1</td>
<td>33,0</td>
<td>30,0</td>
</tr>
<tr>
<td></td>
<td>static</td>
<td>II/2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>static</td>
<td>II/3</td>
<td>33,0</td>
<td>30,0</td>
</tr>
<tr>
<td></td>
<td>fatigue</td>
<td>II/4</td>
<td>42,9</td>
<td>33,7</td>
</tr>
<tr>
<td></td>
<td>fatigue</td>
<td>II/5</td>
<td>33,7</td>
<td>33250</td>
</tr>
<tr>
<td></td>
<td>fatigue</td>
<td>II/6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>III</td>
<td>static</td>
<td>III/1</td>
<td>38,1</td>
<td>34,0</td>
</tr>
<tr>
<td></td>
<td>static</td>
<td>III/2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>static</td>
<td>III/3</td>
<td>38,1</td>
<td>34,0</td>
</tr>
<tr>
<td></td>
<td>fatigue</td>
<td>III/4</td>
<td>44,5</td>
<td>38,3</td>
</tr>
<tr>
<td></td>
<td>fatigue</td>
<td>III/5</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>fatigue</td>
<td>III/6</td>
<td>44,5</td>
<td>38,3</td>
</tr>
</tbody>
</table>

3.1.3.2.2 Mortar for tests of type III

The compressive strength of the mortar of the test specimens of type III was determined with cubes 100 mm. The mean value of the compressive strength was 102,5 N/mm². The corresponding compressive cylinder strength $f_c$ results to 86,1 N/mm².
3.1.3.2.3 Headed studs

The studs were fabricated and welded by the company Köster, Ennepetal, Germany. From three tests the yield strength $f_y$ of the headed studs was measured to 420 N/mm² and the tensile strength $f_u$ to 465 N/mm². The geometrical values of the studs and the weld collars were measured.

- Mean shank diameter $d_1$: 25 mm
- Mean diameter of the weld collars: 32.2 mm, $> 1.25 \cdot d_1 = 31.25$ mm
- Mean height of the weld collars: 7.1 mm, $> 0.2 \cdot d_1 = 5$ mm
- Maximum height of the weld collars: 10.9 mm
- Minimum height of the weld collars: 4.4 mm, $> 0.15 \cdot d_1 = 3.75$ mm
- Mean height of the welded studs: 125.1 mm

3.1.3.3 Measurements

In tests of type Ib, II and III the slip between concrete and steel section was measured by four displacement transducers shown in Fig. 3-2, Fig. 3-3, Fig. 3-4 and Fig. 3-5. For the test specimen of type Ia four additional displacement transducers (Fig. 3-2) had been provided in order to measure the slip between steel plate and the concrete and additionally the slip between the steel section and the concrete. Fig. 3-6 shows the typical arrangement of the displacement transducers.

![Fig. 3-6: Displacement transducers for the determination of the slip between steel and concrete](image-url)
3.1.3.4 Testing procedure

3.1.3.4.1 General

The testing procedure of the static push out tests follows the recommendations of section 10 of Eurocode 4 [1]. Furthermore these static tests are the basis for the determination of the upper load level and the shear range for the fatigue tests.

3.1.3.4.2 Static tests

For the static tests the load was first applied in increments up to 40 % of the expected failure load. The expected failure load of type II and type III test specimen was determined in accordance with Eurocode 4 and the statistical analysis in the background document on Eurocode 4 [3]. Furthermore the test results according to [5] were taken into account. After reaching this load level the test specimen was cycled 100 times between 5% and 40% of the expected ultimate load (Fig. 3-7).

![Testing procedure of static push out tests](image)

For type II and type III test specimen the peak load of initial cyclic loading were chosen to $F_{\text{max},1} = 600 \text{ kN}$. The subsequent load increments were imposed that failure occurred between 15 and 30 minutes. All static tests were carried out displacement controlled.

3.1.3.4.3 Fatigue tests

For the fatigue tests the upper load level was approximately 60% of the ultimate static load determined from the static push out tests. The peak loads and force ranges as well as the peak loads and shear ranges per stud and the shear stress amplitudes are given in Table 3-3. The fatigue loading was performed by a frequency of approximately 2 Hz during the whole test without any break.
### Table 3-3: Loading of fatigue tests

<table>
<thead>
<tr>
<th>Test</th>
<th>$F_{\text{max}}$ [kN]</th>
<th>$P_{\text{max}}$ [kN]</th>
<th>$\Delta F$ [kN]</th>
<th>$\Delta P$ [kN]</th>
<th>$\Delta \tau$ [N/mm$^2$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ia/3</td>
<td>1050</td>
<td>-</td>
<td>510</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Ib/3</td>
<td>900</td>
<td>-</td>
<td>450</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>II/4</td>
<td>880</td>
<td>110</td>
<td>590</td>
<td>73,75</td>
<td>150</td>
</tr>
<tr>
<td>II/5</td>
<td>880</td>
<td>110</td>
<td>510</td>
<td>63,75</td>
<td>130</td>
</tr>
<tr>
<td>II/6</td>
<td>880</td>
<td>110</td>
<td>430</td>
<td>53,75</td>
<td>110</td>
</tr>
<tr>
<td>III/4</td>
<td>690</td>
<td>86,25</td>
<td>590</td>
<td>73,75</td>
<td>150</td>
</tr>
<tr>
<td>III/5</td>
<td>690</td>
<td>86,25</td>
<td>510</td>
<td>63,75</td>
<td>130</td>
</tr>
<tr>
<td>III/6</td>
<td>690</td>
<td>86,25</td>
<td>430</td>
<td>53,75</td>
<td>110</td>
</tr>
</tbody>
</table>

### 3.2 Evaluation of test results and conclusions

#### 3.2.1 Test results

##### 3.2.1.1 Tests of type Ia

The test results of the static push out tests are given in Table 3-4 and the load slip-curves are shown in Fig. 3-8, Fig. 3-9 and Fig. 3-11. The test Ia/3 was planned as a fatigue test with a peak load $P_{\text{max}} = 1050$ kN and a shear range of $\Delta P = 510$ kN. After 4,000,000 cycles no fatigue failure occurred. The test specimen was then loaded statically until failure. The load slip curve under fatigue loading $P_{\text{max}}$ is shown in Fig. 3-10.

### Table 3-4: Results of tests of type Ia

<table>
<thead>
<tr>
<th>Test specimen</th>
<th>$f_c$ [N/mm$^2$]</th>
<th>$E_c$ [N/mm$^2$]</th>
<th>$F_u$ [kN]</th>
<th>$P_u$ [kN]</th>
<th>$\delta_u$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ia/1</td>
<td>49,1</td>
<td>34520</td>
<td>1740</td>
<td>217,5</td>
<td>1,81</td>
</tr>
<tr>
<td>Ia/2</td>
<td></td>
<td></td>
<td>1810</td>
<td>226,3</td>
<td>2,7</td>
</tr>
<tr>
<td>Ia/3*</td>
<td></td>
<td></td>
<td>1260</td>
<td>157,5</td>
<td>0,45</td>
</tr>
</tbody>
</table>

* residual strength after fatigue loading with 4,000,000 cycles

In the static tests the specimens failed due to shear of the bolts in the interface between the steel section and the additional steel plate. The ultimate strength of test Ia/3 shows, that the ultimate static strength decreases significantly due to the fatigue preloading.

Fig. 3-9 and Fig. 3-11 (concrete in front of the screw heads) show clearly that there is a combined shear resistance of the headed studs and the screw head.
Fig. 3-8: Load-slip curve of test la/1

Test la/1
\[ F_u = 1740 \text{ kN} \]
\[ \delta_u = 1.81 \text{ mm} \]

Fig. 3-9: Load-slip curve of test la/2

Test la/2
\[ F_u = 1810 \text{ kN} \]
\[ \delta_u = 2.7 \text{ mm} \]
Fig. 3-10: Load-slip curve of test la/3 after fatigue Loading

$F_{\text{max}} = 1050 \text{kN}$

$\Delta F = 510 \text{kN}$

$\delta_{u} = 0,45 \text{mm}$

Fig. 3-11: Load-slip curve of test la/3 during fatigue loading

$F_{\text{max}} = 1050 \text{kN}$

$\Delta F = 510 \text{kN}$

$\delta_{u} = 0,45 \text{mm}$
### 3.2.1.2 Tests of type Ib

The test results of the static push out tests are given in Table 3-5 and the load slip-curves in Fig. 3-12, Fig. 3-13, Fig. 3-14 and Fig. 3-15. The test Ia/3 was planned as a fatigue test with a peak load of $P_{\text{max}} = 1050$ kN and a shear range of $\Delta P = 510$ kN. After 3,000,000 cycles no fatigue failure occurred. The test specimen was then loaded statically until failure. The slip-curve under fatigue loading $P_{\text{max}}$ is shown in Fig. 3-14. Shear failure of the bolts occurred in every test in the interface between the concrete member and the steel plate. Typical failure modes are shown in the following figures.

#### Table 3-5: Results of tests of type Ib

<table>
<thead>
<tr>
<th>Test specimen</th>
<th>$f_c$ [N/mm$^2$]</th>
<th>$E_c$ [N/mm$^2$]</th>
<th>$F_u$ [kN]</th>
<th>$P_u$ [kN]</th>
<th>$\delta_u$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ib/1</td>
<td>46,9</td>
<td>34260</td>
<td>1540</td>
<td>192,5</td>
<td>9,8</td>
</tr>
<tr>
<td>Ib/2</td>
<td>1500</td>
<td>187,5</td>
<td>1490</td>
<td>186,25</td>
<td>8,6</td>
</tr>
<tr>
<td>Ib/3*</td>
<td>46,9</td>
<td>34260</td>
<td>1540</td>
<td>192,5</td>
<td>9,8</td>
</tr>
</tbody>
</table>

*) residual strength after fatigue loading with 3,000,000 cycles

#### Fig. 3-12: Load-slip curve of test Ib/1

Test Ib/1

$F_u = 1540$ kN

$\delta_u = 9,8$ mm
Fig. 3-13: Load-slip curve of test Ib/2

Fig. 3-14: Load-slip curve of test Ib/3 after fatigue Loading
Fig. 3-15: Load-slip curve of test lb/3 during fatigue loading

### 3.2.1.3 Tests of type II

The test results of the static push-out tests are given in Table 3-6 and Fig. 3-16, Fig. 3-17 and Fig. 3-18 show the load slip-curves of these tests. The results of the fatigue tests are shown in Table 3-7 and the corresponding slip curves under the peak load are shown in Fig. 3-19, Fig. 3-20 and Fig. 3-21. In all static tests the failure mode was crushing of concrete. In the fatigue tests fatigue cracks through the weld collar were observed.

**Table 3-6: Test results of static push out tests of type II**

<table>
<thead>
<tr>
<th>Test specimen</th>
<th>$f_c$ [N/mm$^2$]</th>
<th>$E_c$ [N/mm$^2$]</th>
<th>$F_u$ [kN]</th>
<th>$P_u$ [kN]</th>
<th>$\delta_u$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>II/1</td>
<td>33</td>
<td>30480</td>
<td>1510</td>
<td>188.75</td>
<td>4.9</td>
</tr>
<tr>
<td>II/2</td>
<td></td>
<td></td>
<td>1403</td>
<td>175.37</td>
<td>3.8</td>
</tr>
<tr>
<td>II/3*</td>
<td></td>
<td></td>
<td>1485</td>
<td>185.62</td>
<td>4.2</td>
</tr>
</tbody>
</table>

**Table 3-7: Test results of fatigue tests**

<table>
<thead>
<tr>
<th>Test specimen</th>
<th>$f_c$ [N/mm$^2$]</th>
<th>$E_c$ [N/mm$^2$]</th>
<th>$P_{max}$ [kN]</th>
<th>$\Delta P$ [kN]</th>
<th>$\Delta \tau$ [N/mm$^2$]</th>
<th>Number of cycles N</th>
</tr>
</thead>
<tbody>
<tr>
<td>II/4</td>
<td>42.9</td>
<td>33250</td>
<td>110</td>
<td>73.75</td>
<td>150</td>
<td>136.190</td>
</tr>
<tr>
<td>II/5</td>
<td></td>
<td></td>
<td>110</td>
<td>63.75</td>
<td>130</td>
<td>388.900</td>
</tr>
<tr>
<td>II/6*</td>
<td></td>
<td></td>
<td>110</td>
<td>53.75</td>
<td>110</td>
<td>654.200</td>
</tr>
</tbody>
</table>
Fig. 3-16: Load-slip curve of test II/1

Fig. 3-17: Load-slip curve of test II/2
Fig. 3-18: Load-slip curve of test II/3

Fig. 3-19: Load-slip curve of fatigue test II/4
Fig. 3-20: Load-slip curve of fatigue test II/5

Fig. 3-21: Load-slip curve of fatigue test II/6
3.2.1.4 Tests of type III

The test results of the static push out tests are given in Table 3-8 and the Fig. 3-22, Fig. 3-23 and Fig. 3-24 show the load slip-curves. The results of the fatigue tests are shown in Table 3-9 and the corresponding slip curves under the peak load are given in Fig. 3-25, Fig. 3-26 and Fig. 3-27. In all static tests the failure mode was crushing of mortar. At first the mortar cracked horizontally in the area of the upper studs. In the second stage failure of the whole block of mortar occurred. The studs were not deformed. The whole block of mortar failed like a big block connector. In all fatigue tests fatigue cracks occurred in the weld collars of the studs.

Table 3-8: Test results of static push out tests of type II

<table>
<thead>
<tr>
<th>Test specimen</th>
<th>f_c [N/mm²]</th>
<th>E_c [N/mm²]</th>
<th>F_u [kN]</th>
<th>P_u [kN]</th>
<th>δ_u [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>III/1</td>
<td>38,1</td>
<td>31400</td>
<td>1395</td>
<td>174,4</td>
<td>2,1</td>
</tr>
<tr>
<td>III/2</td>
<td>1254</td>
<td>156,8</td>
<td>1254</td>
<td>156,8</td>
<td>1,8</td>
</tr>
<tr>
<td>III/3*</td>
<td>38,1</td>
<td>31400</td>
<td>1355</td>
<td>169,4</td>
<td>2,2</td>
</tr>
</tbody>
</table>

Table 3-9: Test results of fatigue tests

<table>
<thead>
<tr>
<th>Test specimen</th>
<th>f_c [N/mm²]</th>
<th>E_c [N/mm²]</th>
<th>P_max [kN]</th>
<th>ΔP [kN]</th>
<th>Δτ [N/mm²]</th>
<th>Number of cycles N</th>
</tr>
</thead>
<tbody>
<tr>
<td>III/4</td>
<td>86,25</td>
<td>73,75</td>
<td>150</td>
<td>108.120</td>
<td></td>
<td></td>
</tr>
<tr>
<td>III/5</td>
<td>86,25</td>
<td>63,75</td>
<td>130</td>
<td>193.280</td>
<td></td>
<td></td>
</tr>
<tr>
<td>III/6*</td>
<td>86,25</td>
<td>53,75</td>
<td>110</td>
<td>3.893.300</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. 3-22: Load-slip curve of test III/1

Test III/1
F_u = 1395 kN
δ_u = 2,8 mm
Fig. 3-23: Load-slip curve of test III/2

Fig. 3-24: Load-slip curve of test II/3
Fig. 3-25: Load-slip curve of fatigue test III/4

Fig. 3-26: Load-slip curve of fatigue test III/5
3.2.2 Statistical evaluation of existing static and fatigue push out tests

3.2.2.1 General

The design rules in Eurocode 4 are based on the background document [3] for the static strength of headed studs and on paper [4] regarding the fatigue strength of headed studs. The studies were published in 1988 and 1990 and took not into account studs with diameters of 25mm. In the meantime new static and fatigue test were carried out [5, 6 and 13], where stud diameters of 22 and 25 mm were used. In order to compare the test results in 3.2.2 with the existing tests for headed studs in solid slabs, the following gives a new statistical analysis for static and fatigue strength of headed studs.

3.2.2.2 Evaluation of static push-out tests with headed studs

3.2.2.2.1 General

The design rules in Eurocode 4 are based on the semi-empirical model according to [7]. The following procedure describes the determination of the characteristic strength and the partial safety factors in accordance with Annex D of EN 1990 [17] for the model according to [7]. The available test results are divided in two sub-sets according the different failure modes observed in the tests. Equation 3-1 gives the theoretical values due to crushing of concrete (subset 1) and from equation 3-2 the theoretical values for shear failure of the shank can be obtained (subset 2).

\[
\text{sub-set No.1:}
\]
\[ P_{t,c} = g_{rt}(X_1, X_2, X_3) = k_{c,m} \frac{d^2}{\sqrt{E_{cm} f_c}} \text{ with } k_{cm} = 0.35 \quad (3-1) \]

sub-set No. 2:

\[ P_{t,s} = g_{rt}(X_1, X_4) = k_{s,m} \frac{\pi d^2}{4} f_u \text{ with } k_{sm} = 1.0 \quad (3-2) \]

In the equations 3-1 and 3-2 \( X_1 = d \) is the diameter of the shank of the stud, \( X_2 = f_c \) is the cylinder strength of concrete in accordance with ENV 206, \( X_3 = E_c \) is the secant modulus of elasticity of concrete and \( X_4 = f_u \) is the tensile strength of stud material. The probabilistic model of the resistance is given in the format

\[ P_{t,c} = b_c \delta k_{cm} d^2 \sqrt{E_{cm} f_c} \quad (3-3) \]

\[ P_{t,s} = b_s \delta k_{s,m} \frac{\pi d^2}{4} f_u \quad (3-4) \]

where \( b_c \) and \( b_s \) are the least squares best fit to the slopes for the subsets 1 and 2, given by

\[ b = \frac{\sum P_{ei} P_{ti}}{\sum P_{ti}^2} \quad (3-5) \]

and the error term \( \delta_i \) for each experimental value \( P_{ei} \) is given by

\[ \delta_i = \frac{P_{ei}}{b P_{ti}} \quad (3-6) \]

For the errors terms \( \delta_i \) a log-normal distribution is assumed. The variance is given by equation 7 and the mean value is equal 1,0:

\[ s_\Delta^2 = \frac{1}{n-1} \sum (\Delta_i - \bar{\Delta})^2 \text{ with } \Delta_i = \ln \delta_i \text{ and } \bar{\Delta} = \frac{1}{n} \sum \Delta_i \quad (3-7) \]

For the coefficient of variation of the error terms results to:

\[ V_\delta = \sqrt{\exp(s_\Delta^2)} - 1 \quad (3-8) \]

The mean value \( E(r) \) of the resistance function may be obtained from equation 3-9 and the coefficient of variation \( V_r \) results from equation 3-10

\[ E(r) = b g_{r}(X_{1,m}) \quad (3-9) \]

\[ V_r^2 = V_{rt}^2 + V_\delta^2 \quad (3-10) \]

where the coefficient \( V_{rt} \) can be obtained with

\[ V_{rt}^2 = \left( \sum \frac{\partial g_{rt}}{\partial X_i} \sigma_i \right)^2 \frac{g_{rt}^2(X_m)}{g_{rt}^2(X_m)} \quad (3-11) \]
From the equations 3-1 and 3-2 results for the coefficient of variation \( V_{rt,1} \) for sub-set 1 and \( V_{rt,2} \) for subset 2

\[
V_{rt,1} = \frac{1}{2} \sqrt{16V_d^2 + V_{Ec}^2 + V_{Efd,rt}^2} \approx 0.12
\]

\[
V_{rt,2} = \sqrt{4V_d^2 + V_{fu}^2} \approx 0.08
\]

where for the basic variables the coefficients of variation \( V_d=0.03, V_{Ec}=0.15, V_{Efd,rt}=0.15 \) and \( V_{fu}=0.05 \) are assumed. The characteristic resistance \( P_k \) and the design resistance value \( P_d \) are given by

\[
P_k = b \cdot g_{rt}(X_m) \cdot \exp \left( -1.645 \alpha_{rt} Q_{rt} - k_n \alpha_{\delta} Q_\delta - 0.5 Q^2 \right)
\]

\[
P_d = b \cdot g_{rt}(X_m) \cdot \exp \left( -3.04 \alpha_{rt} Q_{rt} - k_{d,n} \alpha_{\delta} Q_\delta - 0.5 Q^2 \right)
\]

where

\[
Q = \sqrt{\ln (V_{rt}^2 + 1)} \quad Q_{rt} = \sqrt{\ln (V_{rt}^2 + 1)} \quad Q_\delta = \sqrt{\ln (V_{\delta}^2 + 1)}
\]

and the weighting factors \( \alpha_{rt} \) and \( \alpha_{\delta} \) are given by

\[
\alpha_{rt} = Q_{rt} / Q \quad \alpha_{\delta} = Q_\delta / Q
\]

The characteristic fractile factor \( k_n \) for the 5% fractile can be taken from table D1 and the design fractile factor \( k_{n,d} \) from table D2 of EN 1990. Both factors must be determined for \( V_x \) unknown. The partial safety factor results from the equations 3-14 and 3-15 by

\[
\gamma_R = \frac{P_k}{P_d}
\]

In the equations 3-14 and 3-15 the mean values of the material properties are used. Normally in the resistance function the characteristic or nominal material properties are taken into account. For the relation ship between the characteristic strength function and the nominal strength function follows:

\[
\Delta k = \frac{g_{rt}(X_n)}{g_{rt}(X_m)}
\]

For the design resistance related to the nominal values of material properties the partial safety factor is then given by

\[
\gamma^*_R = \Delta k \cdot \gamma_R
\]

For the cylinder strength of concrete and the tensile strength of the stud material a log normal distribution can be assumed. The correction factor \( \Delta k_c \) for equation 3-20 can be obtained from the following equation with the coefficient \( V_{fc}=0.15 \).

\[
\Delta k_c = \sqrt{\exp \left( -1.645 V_{fc}^2 - 0.5 V_{fc}^2 \right) } = 0.88
\]

The coefficient of variation of the tensile strength of the stud material is in the range of 7%. Assuming a log-normal distribution, the correction factor can be obtained from equation 3-22.

\[
\Delta k_s = \exp \left(-1.645 \cdot V_{fu}^2 - 0.5 V_{fu}^2 \right) = 0.92
\]
3.2.2.2 Existing test data and statistical analysis according to EN 1990

The static strength of headed studs is influenced by the geometry and the transverse reinforcement of the test specimen. For the following statistical analysis tests results are used, where the test specimen were in accordance with Eurocode 4 or were the test specimen were comparable with the standard push-out specimen given in Eurocode 4. The test results for subset 1 according to equation 3-1 are given in Table 3-10 and the test results for subset 2 according to equation 3-2 are given in Table 3-11 where n is the number of studs. For the design of composite structures in accordance with Eurocode 4 concrete strength classes lower than C20/25 and higher than C60/75 are not covered. Therefore only tests are used where the cylinder concrete strength was within this range. The results of the statistical analysis are given in the Table 3-12 and the Fig. 3-28 and Fig. 3-29 show the test results and the results of the statistical analysis according to EN 1990.

Fig. 3-28: Results of the statistical analysis for sub set 1 (failure mode crushing of concrete)

Fig. 3-29: Results of the statistical analysis for sub set 2 (failure mode shear failure of the stud)
Table 3-10: Test results for sub-set 1 (failure mode crushing of concrete)

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<td>232,8</td>
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Table 3-12: Results of the statistical analysis of sub-set 1

<table>
<thead>
<tr>
<th>Resistance function</th>
<th>sub-set 1</th>
<th>sub-set 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_{t,c} = 0.35 \sigma^2 \sqrt{E_{cm}} t_c$</td>
<td>$P_{t,s} = \frac{\pi \sigma^2}{4} t_u$</td>
<td></td>
</tr>
<tr>
<td>$n$ - number of tests</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>$b = \frac{\sum P_{p,t} P_{t,i}}{\sum P_{t,i}^2}$</td>
<td>$b = 1.005$</td>
<td>$b = 1.0007$</td>
</tr>
<tr>
<td>$\bar{\Delta} = \frac{1}{n} \sum \Delta_i$ with $\Delta_i = \ln \delta_i = \ln \left( \frac{P_{p,i}}{b P_{t,i}} \right)$</td>
<td>0.0239</td>
<td>-0.0021</td>
</tr>
<tr>
<td>$s^2_\Delta = \frac{1}{n-1} \sum (\Delta_i - \bar{\Delta})^2$</td>
<td>0.017</td>
<td>0.0089</td>
</tr>
<tr>
<td>$V_{\delta} = \sqrt{\exp(s^2_\Delta) - 1}$</td>
<td>0.131</td>
<td>0.0945</td>
</tr>
<tr>
<td>$V_{r}^2 = \sqrt{V_{r,t}^2 + V_{\delta}^2}$</td>
<td>0.192</td>
<td>0.123</td>
</tr>
<tr>
<td>$Q_{\delta} = \sqrt{\ln(V_{\delta}^2 + 1)}$</td>
<td>0.1302</td>
<td>0.0943</td>
</tr>
<tr>
<td>$Q_{r,t} = \sqrt{\ln(V_{r,t}^2 + 1)}$</td>
<td>0.1393</td>
<td>0.078</td>
</tr>
<tr>
<td>$Q = \sqrt{\ln(V_r^2 + 1)}$</td>
<td>0.1899</td>
<td>0.1221</td>
</tr>
<tr>
<td>$\alpha_{r,t} = \frac{Q_{r,t}}{Q}$</td>
<td>0.733</td>
<td>0.638</td>
</tr>
<tr>
<td>$\alpha_{\delta} = \frac{Q_{\delta}}{Q}$</td>
<td>0.685</td>
<td>0.772</td>
</tr>
<tr>
<td>Fractile factors $k_n$ and $k_{d,n}$</td>
<td>1.71/3.40</td>
<td>1.71/3.40</td>
</tr>
<tr>
<td>$\frac{P_k}{b g_{rt}(X_m)} = \exp \left( -1.645 \alpha_{r,t} Q_{r,t} - k_n \alpha_{\delta} Q_{\delta} - 0.5 Q^2 \right)$</td>
<td>0.734</td>
<td>0.808</td>
</tr>
<tr>
<td>$\frac{P_d}{b g_{rt}(X_m)} = \exp \left( -3.04 \alpha_{r,t} Q_{r,t} - k_{d,n} \alpha_{\delta} Q_{\delta} - 0.5 Q^2 \right)$</td>
<td>0.557</td>
<td>0.667</td>
</tr>
<tr>
<td>$\gamma_R = \frac{P_k}{P_d}$</td>
<td>1.318</td>
<td>1.212</td>
</tr>
<tr>
<td>$\Delta k = \frac{g_{rt}(X_n)}{g_{rt}(X_m)}$</td>
<td>0.88</td>
<td>0.92</td>
</tr>
</tbody>
</table>
Composite Bridge Design for Small and Medium Spans

Page 3-27

3.2.2.2.3 Comparison with Eurocode 4 and conclusions

In Eurocode 4 the design resistance of headed studs, automatically welded in accordance with EN ISO 14555 [18] and EN ISO 13918 [19] is given by equations 3-23 and 3-24 whichever is smaller

\[ P_{Rd} = \left( k_s f_u \frac{\pi d^2}{4} \right) \frac{1}{\gamma_v} \quad \text{with} \quad k_s = 0.8 \quad (3-23) \]

\[ P_{Rd} = \left( k_c \alpha \frac{d^2}{\sqrt{f_{ck} E_{cm}}} \right) \frac{1}{\gamma_v} \quad \text{with} \quad k_c = 0.29 \quad (3-24) \]

where:

- \( \gamma_v = 1.25 \) is the nominal partial safety factor for the ultimate limit state;
- \( d \) is the diameter of the shank of the stud;
- \( f_u \) is the specified ultimate tensile strength of the material of the stud but not greater than 500 N/mm\(^2\);
- \( f_{ck} \) is the characteristic cylinder strength of the concrete at the age considered;
- \( E_{cm} \) is the mean value of the secant modulus of the concrete in accordance with EN 1992-1 for normal and for lightweight concrete;
- \( \alpha = 0.2 \left[ (h_{sc} / d) + 1 \right] \) for \( 3 \leq h_{sc} / d \leq 4 \)
- \( \alpha = 1 \) for \( h_{sc} / d > 4 \)
- \( h_{sc} \) is the overall nominal height of the stud.

The tests according to Table 3-10 and Table 3-11 were carried out displacement controlled and the ultimate loads of the test results take not into account the effects of relaxation. It can be seen from the load slip curves given in 3.1.3 and from the tests described in [5] that this effect leads to a reduction of ultimate strength and can be taken into account by a further reduction of approximately 10 %.

From the statistical analysis the design values of the factors \( k_s \) and \( k_c \) of equations 3-23 and 3-34 result to:

\[ k_s = 0.9 \times k_{s,m} \frac{p_k}{b_s g_{rt}(X_m)} \frac{\gamma_v}{\Delta k \gamma_{R,s}} = 0.9 \times 1.0 \times 0.808 \frac{1.25}{1.12} = 0.81 \quad (3-25) \]

\[ k_c = 0.9 \times k_{cm} \frac{p_k}{b_s g_{rt}(X_m)} \frac{\gamma_v}{\Delta k \gamma_{R,c}} = 0.9 \times 0.35 \times 0.734 \frac{1.25}{1.16} = 0.25 \quad (3-26) \]

Comparison of the calculated values with the values in Eurocode 4 shows, that there is a good agreement regarding equation 3-23. The coefficient \( k_c = 0.29 \) in equation 3-24 leads to an overestimation of the design strength. It is recommended to reduce this value to 0.25.
3.2.2.3 Evaluation of fatigue push-out tests

3.2.2.3.1 General

In the following the test results according to [4] are analysed again but taking into account new test data from [6] and [13]. The statistical evaluation proceeds in two steps by a linear regression analysis and the evaluation of the characteristic fatigue strength at 2 million cycles.

As described in [4], for the analysis only tests are used, where the test specimens were in accordance with Eurocode 4 or where the geometry and transverse reinforcement of the specimens is comparable with the standard test specimen in Eurocode 4. Furthermore only tests are taken into account where all basic parameters are known and where studs with diameters of 19, 22 and 25 mm were used.

The fatigue strength of headed studs is significantly influenced by the peak load and the concrete strength of the test specimen. Therefore, in the analysis only tests are used where loading was unidirectional in compression, where peak loads were less than 60% of the static strength, where concrete cylinder strength was in the range of 20 to 40 N/mm² and where the concrete members of the test specimens were casted in horizontal position. These test data are summarised in Table 3-13.

In order to compare the test results, the concrete strength of the test specimens given in the different publications is converted into the cylinder strength according to ENV 206 taking into account the different storage and the different shape of specimen (cylinder or different types of cubes).

The fatigue strength curves in the Eurocodes are given in a double–logarithmic scale. In order to eliminate the influence of the stud diameter the fatigue strength is related to the nominal value of shear stress in the shank of the stud. The fatigue strength curve is then given by:

\[
\log N_i = \log a + m \cdot \log \Delta \tau_{R,i} \quad \text{with} \quad \Delta \tau_{R,i} = \frac{\Delta P}{\pi \cdot d^2 / 4} \quad (3–27)
\]

The linear regression analysis implies an existence of a linear relation between \(\log N_i\) and \(\log \Delta \tau_{R,i}\). The coefficients \(\log a\) and \(m\) are the estimates to be determined that the sum of the squares of the residuals is a minimum. This condition leads to the estimates according to equations 3.2.2.3.

\[
m = \frac{S_{xy}}{S_x} \quad \text{and} \quad \log \tilde{a} = \bar{y} - \bar{x} \quad (3–28)
\]

where for simplicity the following notations are used

\[
x_i = \log \Delta \tau_{R,i} \quad y_i = \log N_i \quad (3–29)
\]

where

- \(\bar{x}\) and \(\bar{y}\) are the mean values of \(x_i\) and \(y_i\) with

\[
\bar{x} = \frac{1}{n} \sum x_i \quad \text{and} \quad \bar{y} = \frac{1}{n} \sum y_i \quad (3–30)
\]

- \(S_x^2\) and \(S_y^2\) are the variations of the random variables \(x_i\) and \(y_i\)

\[
S_x^2 = \frac{1}{n-1} \left[ \sum x_i^2 - \frac{1}{n} \sum x_i^2 \right] \quad S_y^2 = \frac{1}{n-1} \left[ \sum y_i^2 - \frac{1}{n} \sum y_i^2 \right] \quad (3–31)
\]
- $S_{xy}$ is the covariance

$$S_{xy} = \frac{1}{n-1} \left[ \sum_{i=1}^{n} x_i y_i - n \overline{x} \overline{y} \right]$$  \hfill (3–32)

The random test correlation coefficient $R_{xy}$ results to

$$R_{xy} = \frac{S_{xy}}{\sqrt{S_x^2 \cdot S_y^2}}$$  \hfill (3–33)

and the standard deviation can be obtained from

$$s_N = \sqrt{\frac{n}{n-2} \left( 1 - R_{xy} \right) S_y^2}$$  \hfill (3–34)

For the determination of the characteristic strength at $N_c = 2 \times 10^6$ cycles it has to be considered that there is a difference between the estimation given by equation 3–30 and the real values. According to [14] the characteristic value $y_{c,k} = \log N_{c,k}$ is given by:

$$\log N_{c,k} = \log N_c - t(\alpha) \cdot s_N \sqrt{f}$$  \hfill (3–35)

where $s_N$ is given by equation 3-34 and:

$$f = 1 + \frac{1}{n} + \frac{(x_c - \overline{x})^2}{S_x^2}$$  \hfill (3–36)

$$x_c = \log \Delta \tau_{c,m} = \frac{\log N_c - \log a}{m}$$  \hfill (3–37)

$t(\alpha)$ results from the Students t-distribution with a degree of freedom on $(n-2)$ for the 5% confidence interval

The characteristic value of fatigue strength at 2 million cycles results then to:

$$\log \Delta \tau_{c,k} = \log \Delta \tau_{cm} - t(\alpha) \cdot \frac{s_y \sqrt{f}}{m}$$  \hfill (3–38)

The characteristic fatigue strength curve is then given by:

$$\log N_R = \log a_k + m \cdot \log \Delta \tau_R \quad \text{with} \quad \Delta \tau_{R,i} = \frac{\Delta P}{\pi d^2 / 4}$$  \hfill (3–39)

with

$$\log a_k = \log a - \frac{t(\alpha) \cdot s_N \sqrt{f}}{m}$$  \hfill (3–40)
3.2.2.3.2 Existing test data and statistical analysis

As explained before the tests summarised in [4] and new test results from [6] and [13] are used for the statistical analysis. The tests results are shown in Table 3-13 where the mean value of the static strength $P_{t,c,m}$ and $P_{t,s,m}$ is determined in accordance with Fig. 3-28 and Fig. 3-29 and $R$ is the stress ratio $P_{\text{min}}/P_{\text{max}}$. The results of the statistical analysis are given in Table 3-14 and Fig. 3-30. The random test correlation coefficient $R_{xy}=0,94$ shows the good agreement between the test results and the mean regression line.

Table 3-13: Fatigue test data

| No. | Test | $R_{ef}$ | $f_c$ N/mm² | $f_u$ N/mm² | d mm | $P_{\text{max}}$ kN | $P_{\text{min}}$ kN | R | $P_{\text{max}}/P_{t,c,m}$ | $P_{\text{max}}/P_{t,s,m}$ | $\tau_{\text{max}}$ N/mm² | $\Delta \tau$ N/mm² | $N \times 10^3$ |
|-----|------|--------|------------|------------|------|----------------|----------------|---|----------------|----------------|----------------|----------------|----------------|------------|
| 1   | S1   | 9      | 27,4       | 600        | 19   | 50,0           | 5,0            | 0,10 | 0,45           | 0,29           | 176             | 159             | 76,0           |
| 2   | S2   | 9      | 28,3       | 600        | 19   | 40,0           | 4,0            | 0,10 | 0,35           | 0,24           | 141             | 127             | 439,0          |
| 3   | S7   | 9      | 29,5       | 600        | 19   | 40,0           | 4,0            | 0,10 | 0,34           | 0,24           | 141             | 127             | 1940,0         |
| 4   | S9   | 9      | 31,2       | 600        | 19   | 55,0           | 5,5            | 0,10 | 0,45           | 0,32           | 194             | 175             | 42,0           |
| 5   | S10  | 9      | 27,6       | 600        | 19   | 67,0           | 33,5           | 0,50 | 0,60           | 0,39           | 236             | 118             | 1700,0         |
| 6   | S12  | 9      | 27,7       | 600        | 19   | 75,0           | 37,5           | 0,50 | 0,67           | 0,44           | 265             | 132             | 679,0          |
| 7   | PS4.2| 14     | 28,1       | 421        | 19   | 49,8           | 2,3            | 0,05 | 0,44           | 0,42           | 176             | 168             | 52,8           |
| 8   | PS5.1| 14     | 28,1       | 421        | 19   | 49,8           | 2,3            | 0,05 | 0,44           | 0,42           | 176             | 168             | 58,6           |
| 9   | PS5.2| 14     | 28,1       | 421        | 19   | 49,8           | 2,3            | 0,05 | 0,44           | 0,42           | 176             | 168             | 67,9           |
| 10  | PS10.1| 14    | 24,2       | 421        | 19   | 44,3           | 2,3            | 0,05 | 0,43           | 0,37           | 156             | 148             | 61,7           |
| 11  | PS10.2| 14    | 24,2       | 421        | 19   | 44,3           | 2,3            | 0,05 | 0,43           | 0,37           | 156             | 148             | 75,5           |
| 12  | PS11.1| 14    | 24,2       | 421        | 19   | 44,3           | 2,3            | 0,05 | 0,43           | 0,37           | 156             | 148             | 110,0          |
| 13  | PS11.2| 14    | 24,2       | 421        | 19   | 44,3           | 2,3            | 0,05 | 0,43           | 0,37           | 156             | 148             | 110,0          |
| 14  | I/1  | 13     | 31,0       | 468        | 25   | 94,3           | 40,3           | 0,43 | 0,45           | 0,41           | 192,1            | 110             | 1500,0         |
| 15  | I/2  | 13     | 31,0       | 468        | 25   | 94,3           | 30,5           | 0,32 | 0,45           | 0,41           | 192,2            | 130             | 485,0          |
| 16  | I/3  | 13     | 31,0       | 468        | 25   | 94,3           | 20,5           | 0,22 | 0,45           | 0,41           | 192,2            | 150             | 98,0           |
| 17  | I/1  | 13     | 40,0       | 468        | 25   | 106,3          | 52,3           | 0,49 | 0,43           | 0,46           | 216,5            | 110             | 1500,0         |
| 18  | I/2  | 13     | 40,0       | 468        | 25   | 106,3          | 42,5           | 0,40 | 0,43           | 0,46           | 216,5            | 130             | 406,0          |
| 19  | I/3  | 13     | 40,0       | 468        | 25   | 106,3          | 32,6           | 0,31 | 0,43           | 0,46           | 216,5            | 150             | 133,0          |
| 20  | S7/1 | 6      | 39,1       | 505        | 22   | 70,0           | 0,0            | 0,37 | 0,36           | 0,36           | 184              | 184             | 12,3           |
| 21  | S7/2 | 6      | 39,1       | 505        | 22   | 70,0           | 20,0           | 0,29 | 0,37           | 0,36           | 184              | 184             | 181,0          |
| 22  | S7/3 | 6      | 41,6       | 505        | 22   | 70,0           | 20,0           | 0,29 | 0,35           | 0,36           | 184              | 184             | 350,0          |
| 23  | S7/4 | 6      | 43,4       | 505        | 22   | 70,0           | 20,0           | 0,29 | 0,34           | 0,36           | 184              | 184             | 492,0          |
| 24  | S7/5 | 6      | 43,4       | 505        | 22   | 70,0           | 0,0            | 0,34 | 0,36           | 0,36           | 184              | 184             | 26,9           |
| 25  | S7/6 | 6      | 43,4       | 505        | 22   | 70,0           | 0,0            | 0,34 | 0,36           | 0,36           | 184              | 184             | 38,0           |
| 26  | S7/9 | 6      | 41,6       | 505        | 22   | 70,0           | 20,0           | 0,29 | 0,35           | 0,36           | 184              | 184             | 779,5          |
| 27  | S7/10| 6      | 40,8       | 505        | 22   | 70,0           | 10,0           | 0,14 | 0,36           | 0,36           | 184              | 158             | 110,3          |
Table 3-14: Results of the statistical analysis

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of tests n</td>
<td>27</td>
</tr>
<tr>
<td>Mean value of $x_i = \log \Delta \tau_i$</td>
<td>$\bar{x} = \frac{1}{n} \sum x_i$</td>
</tr>
<tr>
<td>Mean value of $y_i = \log N_i$</td>
<td>$\bar{y} = \frac{1}{n} \sum y_i$</td>
</tr>
<tr>
<td>Variance of $x_i$</td>
<td>$S_x^2 = \frac{1}{n-1} \left[ \sum x_i^2 - \frac{1}{n} \sum x_i \bar{x} \right]$</td>
</tr>
<tr>
<td>Variance of $y_i$</td>
<td>$S_y^2 = \frac{1}{n-1} \left[ \sum y_i^2 - \frac{1}{n} \sum y_i \bar{y} \right]$</td>
</tr>
<tr>
<td>Covariance of $x_i$ and $y_i$</td>
<td>$S_{xy} = \frac{1}{n-1} \left[ \sum x_i y_i - n \bar{x} \bar{y} \right]$</td>
</tr>
<tr>
<td>Random test correlation coefficient $R_{xy}$</td>
<td>$R_{xy} = \frac{S_{xy}}{\sqrt{S_x^2 \cdot S_y^2}}$</td>
</tr>
<tr>
<td>Standard deviation</td>
<td>$s_N = \sqrt{\frac{n}{n-2} \left( 1 - R_{xy}^2 \right) S_y^2}$</td>
</tr>
<tr>
<td>Estimate of $m$</td>
<td>$m = \frac{S_{xy}}{S_x}$</td>
</tr>
<tr>
<td>Estimate of $\log a$</td>
<td>$\log a = \bar{y} - m \bar{x}$</td>
</tr>
<tr>
<td>Mean value $\Delta \tau_{cm}$ at $N_c = 2 \times 10^6$</td>
<td>109,5</td>
</tr>
<tr>
<td>Correction factor $f$ for the standard deviation at 2 million cycles</td>
<td>$f = 1 + \frac{1}{n} \left( \frac{x_c - \bar{x}}{S_x^2} \right)^2$</td>
</tr>
<tr>
<td>Student’s t-distribution $t(\alpha)$ for (n-2) and $\alpha = 0,05$</td>
<td>1,71</td>
</tr>
<tr>
<td>Characteristic value $N_{c,k}$</td>
<td>$\log N_{c,k} = \log N_c - t(\alpha) s_N \sqrt{f}$</td>
</tr>
<tr>
<td>Characteristic value $\log \Delta \tau_{ck}$</td>
<td>$\log \Delta \tau_{c,k} = \log \Delta \tau_{cm} - \frac{t(\alpha) s_N \sqrt{f}}{m}$</td>
</tr>
<tr>
<td>Characteristic value $\Delta \tau_{ck}$</td>
<td>89,22</td>
</tr>
<tr>
<td>Characteristic value $\log a_k$</td>
<td>$\log a_k = \log a - \frac{t(\alpha) s_N \sqrt{f}}{m}$</td>
</tr>
</tbody>
</table>
3.2.2.3.3 Comparison with Eurocode 4 and conclusions

In ENV 1994-2 the characteristic fatigue strength of headed studs at two million cycles is given as $\Delta \tau_{ck} = 95 \text{ N/mm}^2$ and the slope of the fatigue strength curve is given by $m=8$. Fig. 3-31 shows a comparison of the fatigue strength curves resulting from the statistical analysis and the curve given in ENV 1994-2. The factor $m$ is strongly influenced by the number of tests but the statistical analysis demonstrates that the value of $m=8$ is in the correct order of magnitude. Furthermore the new statistical analysis with test results mainly based on test specimens according to Eurocode 4 shows, that the characteristic fatigue strength at two million cycles according to ENV 1994-2 overestimates the real fatigue strength. Therefore for design it is recommended to reduce the characteristic fatigue strength given in ENV 1994-2 to $\Delta \tau_{ck}=90 \text{ N/mm}^2$ and to maintain the slope of the fatigue strength curve $m=8$. Table 3-15 shows a comparison of the new proposal with the values of the statistical analysis and the values according to ENV 1994-2.

Table 3-15: Comparison of the new proposal with ENV 1994-2 and the results of the statistical analysis

<table>
<thead>
<tr>
<th></th>
<th>Fatigue strength curve</th>
<th>$\Delta \tau_{fl} \text{ [N/mm}^2\text{]}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$N=10^5$</td>
</tr>
<tr>
<td>ENV 1994-2</td>
<td>Log $N = 22,123 - 8,0 \log \Delta \tau$</td>
<td>138,1</td>
</tr>
<tr>
<td>Statistical analysis</td>
<td>Log $N = 23,187 - 8,658 \log \Delta \tau$</td>
<td>125,8</td>
</tr>
<tr>
<td>New proposal</td>
<td>Log $N = 21,935 - 8,0 \log \Delta \tau$</td>
<td>130,6</td>
</tr>
</tbody>
</table>
3.2.3 Comparison of the new tests with existing tests and the design rules in Eurocode 4

3.2.3.1 Static tests

3.2.3.1.1 General

In the following the new test results given in 3.2.1 are compared with the results of the statistical analysis according to 3.2.2.2. The tests of type II and III can be compared directly. For the tests of type Ia and Ib modified theoretical model must be developed.

3.2.3.1.2 Tests of type Ia

For type Ia shear connection according to Fig. 3-1 the longitudinal shear resistance must be determined for two critical sections (Fig. 3-32). Critical section I-I is the interface between the cover plate and the concrete. In this section combined resistance of the headed studs and the heads of the bolts can be assumed. The resistance of the heads of the bolts is governed by the concrete strength and the dimensions of the heads of the bolts. The total shear resistance $F_u$ of the test specimen is given by

$$ F_u = n_s F_{stud} + n_b F_{c,bolt} $$

(3–41)

where $F_{stud}$ is the shear resistance of the studs, $F_{c,bolt}$ is the resistance of the heads of the bolts and $n_s$ and $n_b$ are the number of studs and bolts. The mean value of shear resistance of the stud $F_{stud}$ results from equations 3-42 whichever is smaller.

$$ P_{stud,1} = 0.35 \, d^2 \, \sqrt{E_{cm} \, f_c} $$

$$ P_{stud,2} = \pi \, d^2 \, f_u / 4 $$

(3–42)
The resistance of the heads of the bolts is given by

\[ F_{c,bolt} = \sigma_{c,R} \, s_b \, h_{hb} \quad \text{with} \quad \sigma_{c,R} = \beta \, f_c \] (3–43)

where \( s_b \) and \( h_{hb} \) are the width and height of the head of the bolt and \( \beta \) is a factor taking into account the effect of local stress concentration in front of the head of the bolt. The factor \( \beta \) can be taken as twice the cylinder strength of concrete.

![Diagram](https://via.placeholder.com/150)

**Fig. 3-32: Model for the determination of longitudinal shear resistance of Type Ia**

**Table 3-16: Comparison of the theoretical model \( F_{u,t} \) with test results \( F_{u,test} \)**

<table>
<thead>
<tr>
<th></th>
<th>Test (Ia/1)</th>
<th>Test (Ia/2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( F_{u,test} ) [kN]</td>
<td>1740</td>
<td>1810</td>
</tr>
</tbody>
</table>

**Section I-I: Failure in the interface between the cover plate and concrete**

- \( P_{stud} = \pi \sigma_d^2 f_u/4 \)
- \( P_{bolt} = \beta \, s_b \, h_{hb} \, f_c \)
- \( s_b = 32 \text{ mm} \) and \( h_{hb} = 13 \text{ mm} \)
- \( F_{u,t} = 8 \, P_{stud} + 8 \, F_{bolt} \)
- \( F_{u,t} = 8 \cdot 228,1 + 8 \cdot 40,85 = 2151,6 \text{ [kN]} \)
- \( F_{u,t}/F_{u,test} = 0,81 \)

**Section II-II: Failure in the interface between the steel plate and the steel section**

- \( F_{vs,m} = 0,6 \, f_{ub} \, A \) \( (a=3,14 \text{ cm}^2) \)
- \( F_{u,t} = n_b \, F_{vs,m} \)
- \( F_{u,t} = 8 \cdot 218,5 = 1748 \text{ kN} \)
- \( F_{u,t}/F_{u,test} = 0,99 \)

The second critical section (section II-II in Fig. 3-32) is the interface between the cover plate and the steel profile. The resistance results from the shear resistance of the bolts.
According to [20] the mean values of resistance for grade 10.9 and shear failure in the shank is given by

\[ F_{vs,m} = 0.6 \, f_{ub} \, A \quad (3-44) \]

where \( f_{ub} \) is the tensile strength and \( A \) the area of the shank of the bolt. The total longitudinal shear resistance is given by

\[ F_u = n_b \, F_{vs} \quad (3-45) \]

Table 3-16 shows the comparison between the test results and the theoretical model. For the comparison with the theoretical model only the test Ia/1 and Ia/2 can be used. The test Ia/3 with four million cycles of fatigue preloading shows that the static strength was reduced caused by initial cracks in thread of the bolt. This can also be seen from Fig. 3-11, where an increase of slip can be seen after \( 3.5 \times 10^6 \) cycles. The material properties of the bolts were not measured. For the determination of the mean value of the tensile strength a coefficient of variation of \( V_{fu} = 9 \% \) and a lognormal distribution can be assumed. The mean value results then to

\[ \frac{f_{ub,k}}{f_{ub,m}} = \exp \left( -1.645 \, V_{fu} - 0.5 \, V_{fu}^2 \right) = 0.86 \quad (3-46) \]

It can be seen that for the test specimen of type Ia the shear failure of the bolts gives the minimum value of resistance. The tests of type Ia were only basic tests to study the principle behaviour of this type of combined shear connection. Due to the small number of tests no design model based on a statistical analysis can be developed and further tests are necessary. Regarding the fatigue behaviour it must be pointed out, that the fatigue strength of this type of shear connection seems to be better than the fatigue strength of headed studs directly welded to the steel section due to combined action of studs and the heads of the bolts.

3.2.3.1.3 Tests of type Ib

Two critical failure modes can occur for this type of shear connection (Fig. 3-33). The first failure mode is crushing of concrete in front of the nuts and the bolt. The resistance can be determined by the resistance of the shank of the bolt \( F_{c,s} \) and the resistance of the nuts \( F_{c,nut} \) where the resistance \( F_{c,s} \) can be determined from equation 3-42 using the diameter \( d_s \) of the shank of the bolt and \( F_{c,nut} \) can be determined from equation 3-43 but using twice the height of the nut. For the shear failure in the thread of the bolt the more critical section II-II must be considered. The mean value of resistance results to:

\[ F_u = n_b \left( F_{c,s} + F_{c,nut} \right) \quad (3-47) \]

\[ F_{c,nut} = \sigma_{c,R} \, s_n \, 2 \, h_n \quad \text{with} \quad \sigma_{c,R} = \beta \, f_c \quad (3-48) \]

\[ F_{c,s} = 0.35 \, d_s^2 \, \sqrt{E_{cm}} \, f_c \quad (3-49) \]

For the critical section II-II between the steel plate and the steel section the ultimate load is given by

\[ F_u = n_b \, F_{vs} \quad (3-50) \]
where $F_{v,s}$ must be determined for shear in the thread of the bolt. According to [20] the mean value of shear resistance can be obtained from

$$F_{v,s,m} \approx 0.56 \ f_{ub} \ A_s$$  \hspace{1cm} \text{(3–51)}$$

where $A_s$ is the effective area in the thread of the bolt. Table 3-17 shows a comparison of the theoretical resistance $F_{u,t}$ with the test results $F_{u,test}$. For the comparison with the test results all tests of this series can be used, because due to fatigue preloading no fatigue failure was observed (constant slip in the fatigue test, Fig. 3-14).

![Fig. 3-33: Model for the determination of shear connection of type Ib](image)

**Table 3-17: Comparison of the theoretical model ($F_{u,t}$) with test results ($F_{u,test}$)**

<table>
<thead>
<tr>
<th></th>
<th>Test Ib/1</th>
<th>Test Ib/2</th>
<th>Test Ib/3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_{u,test}$</td>
<td>1540</td>
<td>1500</td>
<td>1490</td>
</tr>
</tbody>
</table>

**Failure due to crushing of concrete**

$$F_{c,s} = 0.35 \ \sigma^2_s \ \sqrt{E_{cm}} \ f_c$$

$$F_{c,nut} = \sigma_{c,R} \ s_n \ h_n \ \text{with} \ \sigma_{c,R} = \beta \ f_c$$

with $h_n=16$ mm and $s_n=32$ mm

$$F_{u,t} = n_s (F_{c,s} + F_{c,nut})$$

$$F_{u,test}/F_{u,t} = 0.70 \quad 0.68 \quad 0.68$$

**Shear failure in the thread of the bolt**

$$F_{v,s,m} = 0.56 \ f_{ub} \ A_s \ \text{with} \ A_s = 2.45 \ \text{cm}^2$$

$$F_{u,t} = n_b \ F_{v,s,m}$$

$$F_{u,test}/F_{u,t} = 1.21 \quad 1.17 \quad 1.17$$
The statistical analysis for shear failure of bolts in the thread gives a coefficient of variation \( V_\delta = 0.127 \). The 5% and 95% fractile values is given then by

\[
\frac{F_{u,k}}{F_{u,m}} = \exp \left( \pm 1.645 \ V_{f_u} - 0.5 \ V_{f_u}^2 \right) = \begin{cases} 1.222 \\ 0.804 \end{cases}
\quad (3–52)
\]

The comparison with the results of Table 3-17 shows, that the difference between the theoretical model and the test results is within normal range of scattering of test results of shear failure of bolts.

The load slip curves of the test series Ib show, that the deformation behaviour is characterised by a high stiffness at the beginning of loading and significant increase of slip for loads higher than 500 kN. With regard to design rules this effect must be taken into account in order to avoid significant slip in serviceability limit states. Therefore for the development of design rules further tests are necessary. It should be pointed out, that this type of shear connection seems to have a high fatigue resistance. In test Ib/3 after three million cycles no fatigue failure and no significant increase of slip was observed.

### 3.2.3.1.4 Tests of type II

The test results of static tests II/1, II,2 and II,3 can be directly compared with test evaluation according to Fig. 3-28 and Fig. 3-29. Table 3-18 gives the comparison between the theoretical and experimental values.

**Table 3-18: Comparison of test results of type II with theoretical values**

<table>
<thead>
<tr>
<th></th>
<th>Test II/1</th>
<th>Test II/2</th>
<th>Test II/3</th>
</tr>
</thead>
<tbody>
<tr>
<td>( P_{u,test} ) [kN]</td>
<td>188,8</td>
<td>175,4</td>
<td>185,6</td>
</tr>
<tr>
<td><strong>Failure due to crushing of concrete</strong> ( (E_{cm} = 30480 \ N/mm^2 \ f_c = 33 \ N/mm^2) )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>mean value: ( P_{t,m} = 0.35 \ d^2 \sqrt{E_{cm} f_c} )</td>
<td>( P_{t,m} = 0.35 \cdot 2.5^2 \sqrt{3.3 \cdot 3048} = 219,4 ) kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>characteristic value: ( P_k = 0.257 \ d^2 \sqrt{E_{cm} f_c} )</td>
<td>( P_k = 0.257 \cdot 2.5^2 \sqrt{3.3 \cdot 3048} = 161,1 ) kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( P_{u,test}/P_{t,m} )</td>
<td>0.86</td>
<td>0.79</td>
<td>0.85</td>
</tr>
<tr>
<td>( P_{u,test}/P_k )</td>
<td>1.17</td>
<td>1.09</td>
<td>1.15</td>
</tr>
<tr>
<td><strong>Shear failure in the shank</strong> ( (f_u = 465 \ N/mm^2) )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>mean value: ( P_{t,m} = \pi d^2 f_u / 4 )</td>
<td>( P_{t,m} = 3.14 \cdot 2.5^2 \cdot 46.5 / 4 = 228,14 ) kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>characteristic value: ( P_k = 0.808 \pi d^2 f_u / 4 )</td>
<td>( P_k = 0.808 \cdot 3.14 \cdot 2.5^2 \cdot 46.5 / 4 = 184,3 ) kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( P_{u,test}/P_{t,m} )</td>
<td>0.83</td>
<td>0.77</td>
<td>0.81</td>
</tr>
<tr>
<td>( P_{u,test}/P_k )</td>
<td>1.02</td>
<td>0.95</td>
<td>1.007</td>
</tr>
</tbody>
</table>
The comparison of the test results with the theoretical values derived from the statistical analysis for headed studs in solid slabs shows that the results of test series II follow not the mean values of the statistical analysis. The main reason for the reduced resistance is the lateral distance between the weld collar and the inclined side-faces of the partially prefabricated slabs and the reduced lateral shear transfer in the concrete.
The transverse reinforcement is located above the prefabricated slabs and can not prevent splitting of concrete in the same way as in solid slabs with transverse reinforcement in the region of the weld collar of the stud. A similar effect was observed in tests with profiled sheeting not continuous over the beam [21].

The tests show that for the design of shear connection with headed studs in combination with partially prefabricated slabs a reduction of the design resistance is necessary. The reduction depends mainly on the lateral distance between the stud and the inclined side face of the partially prefabricated element. Where the transverse reinforcement $A_{sf}$ above the partially prefabricated element is anchored in with the design lap length $l_0$ according to Eurocode 2, the design resistance of headed studs, automatically welded in accordance with EN ISO 14555 [18] and EN ISO 13918 [19] is given by equations 52 and 53 whichever is smaller

\[
P_{Rd} = \kappa_p \left( k_s f_u \frac{\pi d^2}{4} \right) \frac{1}{\gamma_v} \quad \text{with} \quad k_s = 0.8 \tag{3–53}
\]

\[
P_{Rd} = \kappa_p \left( k_c \alpha d^2 \sqrt{\frac{f_{ck}}{E_{cm}}} \right) \frac{1}{\gamma_v} \quad \text{with} \quad k_c = 0.25 \tag{3–54}
\]

where:

- $\gamma_v = 1.25$ is the nominal partial safety factor for the ultimate limit state;
- $d$ is the diameter of the shank of the stud;
- $f_u$ is the specified ultimate tensile strength of the material of the stud but not greater than 500 N/mm$^2$;
- $f_{ck}$ is the characteristic cylinder strength of the concrete at the age considered;
- $E_{cm}$ is the mean value of the secant modulus of the concrete in accordance with EN 1992-1 for normal and for lightweight concrete;
- $\alpha = 0.2 \left( \frac{h_{sc}}{d} + 1 \right)$ for $3 \leq \frac{h_{sc}}{d} \leq 4$
- $\alpha = 1$ for $\frac{h_{sc}}{d} > 4$
- $h_{sc}$ is the overall nominal height of the stud;
- $\kappa_p$ is a reduction factor according to Fig. 3-36, taking into account the distance $e_s$ between the stud and the side faces of the partially prefabricated slab.

**Fig. 3-36: Reduction factor for the design resistance of headed studs in case of partially prefabricated slabs**
3.2.3.1.5 Tests of type III

Table 3-19, Fig. 3-37 and Fig. 3-38 show the comparison between the test results and the statistical analysis for headed studs in solid slabs. The resistance of all tests is significant lower than the mean values of the statistical analysis. It can be seen from the tests that the whole block of mortar failed in tests instead of local failure of the studs. The main reason for this failure mode is the inclination of the side faces of the holes in the precast elements and the small distance between the edge of the weld collar and the side face of the precast element. A further reason for the reduced strength is the missing transverse reinforcement in the holes of the precasted element and the reduced shear transfer over the concrete to the side faces of the holes for the studs.

Table 3-19: Comparison of test results of type III with theoretical values

<table>
<thead>
<tr>
<th></th>
<th>Test III/1</th>
<th>Test III/2</th>
<th>Test III/3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_{u,test}$ [kN]</td>
<td>174,4</td>
<td>156,8</td>
<td>169,4</td>
</tr>
<tr>
<td><strong>Failure due to crushing of concrete ($E_{cm} = 31400$ N/mm$^2$ $f_c = 38$ N/mm$^2$)</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>mean value: $P_{t,m} = 0.35 d^2 \sqrt{E_{cm} f_c}$</td>
<td>$P_{t,m} = 0.35 \cdot 2.5^2 \sqrt{3.8 \cdot 3140} = 238,4$ kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>characteristic value: $P_k = 0.257 d^2 \sqrt{E_{cm} f_c}$</td>
<td>$P_k = 0.257 \cdot 2.5^2 \sqrt{3.8 \cdot 3140} = 175,4$ kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$P_{u,test}/P_{t,m}$</td>
<td>0,73</td>
<td>0,66</td>
<td>0,71</td>
</tr>
<tr>
<td>$P_{u,test}/P_k$</td>
<td>0,99</td>
<td>0,89</td>
<td>0,97</td>
</tr>
<tr>
<td><strong>Shear failure in the shank ($f_u = 465$ N/mm$^2$)</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>mean value: $P_{t,m} = \pi d^2 f_u / 4$</td>
<td>$P_{t,m}= 3.14 \cdot 2.5^2 \cdot 46,5/4= 228,14$ kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>characteristic value: $P_k = 0.808 \pi d^2 f_u / 4$</td>
<td>$P_k = 0.808 \cdot 3.14 \cdot 2.5^2 \cdot 46,5/4= 184,3$ kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$P_{u,test}/P_{t,m}$</td>
<td>0,76</td>
<td>0,69</td>
<td>0,74</td>
</tr>
<tr>
<td>$P_{u,test}/P_k$</td>
<td>0,95</td>
<td>0,85</td>
<td>0,92</td>
</tr>
</tbody>
</table>

For the design of shear connector in groups and located in holes of precast elements the inclination of the side faces of the holes in the concrete elements should be avoided. The influence of the distance between the weld collar of the studs and the side faces of the holes can be taken into account by the reduction factor $\kappa_p$ according to the equations 3-52 and 3-53.
Fig. 3-37: Comparison of test results of type III with theoretical values (failure mode crushing of concrete)

\[ P_{t,m} = 0.35 \pi d^2 \sqrt{E_{cm} f_c} \]

\[ P_k = 0.734 P_{t,m} \]

\[ P_d = 0.557 P_{t,m} \]

Tests III/1, III/2 and III/3

Fig. 3-38: Comparison of test results of type III with theoretical values (shear failure in the shank)

\[ P_{t,m} = \frac{\pi d^2}{4} f_u \]

\[ P_k = 0.808 P_{t,m} \]

\[ P_d = 0.667 P_{t,m} \]
3.2.3.2 Fatigue tests

Fatigue tests of series II and III are compared with the results of the statistical analysis according to 3.2.2.3.2. Fig. 3-39 gives the results of the fatigue tests and the comparison with the statistical analysis.

![Fatigue Tests Diagram](image)

Fig. 3-39: Comparison of test results of series II and III with the results of the statistical analysis

It can be seen that there is no significant difference between the fatigue strength of studs in combination with prefabricated elements and studs in solid concrete slabs. Therefore for design the fatigue strength curves according 3.2.2.3.3 can be used with the reduction of the characteristic value to $\Delta \tau_{ck} = 90 \text{ N/mm}^2$ at two million cycles. With regard to the fatigue strength of headed studs in regions of hogging bending the interaction between the tensile stresses in the top flange of the girder and the shear stresses in the shank of the stud can be taken into account according ENV 1994-2 (Fig. 3-40).

![Design of Headed Studs Diagram](image)

Fig. 3-40: Design of headed studs for fatigue loading
3.3 Summary

The main topic of this working item is the development of design rules for the static and fatigue resistance of headed studs in combination with partially prefabricated concrete slabs and precast slabs in combination with studs arranged in groups. Based on new static and fatigue tests and new statistical analysis of test results for headed studs in solid in situ concrete slabs design rules were developed. Furthermore some basic tests were carried out where bolts and bolts in combination with headed studs were used as shear connectors. The tests show that bolts of grade 10.9 can be an alternative possibility for shear connection in composite structures. Due to the small number of tests, for the development of design rules further tests are necessary.

3.4 References


[13] ECSC Research Project: Use of High Strength Steel S460


[17] prEN 1990: Eurocode: Basis of structural design


4 Behaviour of Joints of Slabs

4.1 Joints in partially prefabricated slabs

4.1.1 Scope and objectives

One demonstration was performed to simulate the behaviour of a joint in the concrete slab of a composite bridge in hogging. The test specimen represents a composite girder at an intermediate support of a two span or multi span bridge (Fig. 4-1). The section is composed of two HEA 900 in S460 and a partially prefabricated concrete slab at a total thickness of 20 cm.

A vertical primary load $P$ was applied at the end of the specimen representing the unfrequent load in a reference bridge to produce constant tension and cracks in the concrete slab. Then a working load was applied on each side of the joint at mid-span of the slab to simulate a passing wheel according to the relevant load model given in ENV 1991-3.

Using prefabricated elements may be supposed to be a very economic way of constructing as only little formwork and quick erection is provided. However for transportation and mounting reasons the elements are limited in span and width which causes joints in the slab. Therefore, the transverse reinforcement of the slab which is located in the in-situ concrete layer should be designed very carefully as the continuity of the composite beam section should be provided as well. The slab undergoes a combined action due to global bending of the bridge and the local bending and shear effects resulting from running wheel loads. The aim of this research is to verify whether this detail is acceptable for composite girders in hogging.

4.1.2 Specimen

4.1.2.1 Reference project and design of the specimen

All tests that were performed at the University of Liège were aligned to the design of a reference bridge to reflect realistic loading and dimensions. The design of this reference project is summarised in 5.1.4.

The specimen had a length of 10 m and a width of 3.30 m. The steel profiles were two rolled HEA 900 grade S460. The concrete was C35/45 for the prefabricated element as well as for the in-situ concrete layer. The shear connectors were headed studs with a diameter of 22 mm and a length of 125 mm arranged in a spacing of 150 mm in one line on each steel beam.
The reinforcement in the slab corresponds to the reinforcement needed in the reference bridge disregarding the longitudinal reinforcement of the composite beams. The section of the main rebars of the slab (transverse to the girders) result from LM1 acc. to ENV 1991-3, 4.3.2 whereas the section of the transverse slab rebars (parallel to the girders) result from LM2 (4.3.3).

The main reinforcement (transverse to the girders) which were located in the prefabricated slab element were determined to have 14mm diameter spaced by 100mm.

Apart from the calculation of the reference bridge it was decided by the team of this research project to reduce the section of transverse reinforcement (parallel to the girders) from 16 mm diameter to 10 mm spaced by 100mm. The reason for this modification was the available reference to actual bridges. The bars were placed directly on the prefabricated element. The same section was placed in the upper part on the bridge directly onto the shear trusses of the prefabricated slab elements. Hereby the ratio of reinforcement to concrete section was 0.785% which is more than the minimum required in Eurocode ENV 1994-1-1, 5.3.1. In case of non-satisfactory results from this test the section of the lower longitudinal reinforcement (transverse slab reinforcement) was 12 mm diameter at a spacing of 100 mm to have the option for a second test at the joint located 1.20m from the support. The centre of overlapping re-bars was 600 mm beside the joint directly at the intermediate support such that the influence for the first test is supposed to be neglectable. Fig. 4-2 shows the arrangement of the reinforcement in the slab.

![Fig. 4-2: arrangement of the reinforcement in the slab](image)

Between the main beams a cross beam was forseen to prevent lateral torsional buckling in the test. It was designed to resist at 1/10 of the maximum normal compression of the longitudinal main beam. The buckling resistance was checked and all connections were made with pretensioned bolts M24 grade 10.9.

### 4.1.2.2 Material properties

The mechanical properties of the rebars were determined in coupon tests according to EN10002 and ENV 10080. The resistance of the concrete for both the prefabricated slab element and the in-situ concrete were derived from tests on standardised cubes. Tables 4-1, 4-2 and 4-3 summarise the tests results.
Table 4-1  mechanical properties of re-bars according to EN 10002 and ENV 10080

<table>
<thead>
<tr>
<th>nominal diameter [mm]</th>
<th>cross section [mm²]</th>
<th>yield stress Re [N/mm²]</th>
<th>ultimate tensile strength Rm [N/mm²]</th>
<th>Rm/Re [%]</th>
<th>A10 [%]</th>
<th>A5 [%]</th>
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<tr>
<td>14</td>
<td>153.68</td>
<td>559.9</td>
<td>655.8</td>
<td>1.171</td>
<td>15.14</td>
<td>21.4</td>
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<td>12</td>
<td>109.21</td>
<td>535.5</td>
<td>606.1</td>
<td>1.132</td>
<td>17.6</td>
<td>24.0</td>
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<tr>
<td>10</td>
<td>81.67</td>
<td>568.9</td>
<td>622.7</td>
<td>1.094</td>
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<td>22.4</td>
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<tr>
<td>8</td>
<td>49.80</td>
<td>568.5</td>
<td>638.6</td>
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<td>14.3</td>
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<td>7</td>
<td>37.72</td>
<td>554.4</td>
<td>599.4</td>
<td>1.081</td>
<td>10.6</td>
<td>18.0</td>
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<tr>
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<td>19.41</td>
<td>596.1</td>
<td>621.3</td>
<td>1.042</td>
<td>10.2</td>
<td>16.8</td>
</tr>
</tbody>
</table>

Table 4-2  mechanical properties of prefabricated concrete element (concrete cubes acc. to NBN B15-220)

<table>
<thead>
<tr>
<th>age [days]</th>
<th>weight [kg]</th>
<th>Dimensions S [mm x mm]</th>
<th>h [mm]</th>
<th>ultimate load [kN]</th>
<th>resistance [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
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<td>158 x 158</td>
<td>158</td>
<td>1130</td>
<td>45.3</td>
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<td>7</td>
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<td>158 x 158</td>
<td>158</td>
<td>975</td>
<td>39.1</td>
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<tr>
<td>7</td>
<td>9.25</td>
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<td>158</td>
<td>1045</td>
<td>41.9</td>
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<tr>
<td>53</td>
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<td>1440</td>
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<tr>
<td>53</td>
<td>9.15</td>
<td>158 x 158</td>
<td>158</td>
<td>1355</td>
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<td>9.00</td>
<td>158 x 158</td>
<td>158</td>
<td>1310</td>
<td>52.5</td>
</tr>
</tbody>
</table>

Table 4-3  mechanical properties of concrete (concrete cubes acc. to NBN B15-220)

<table>
<thead>
<tr>
<th>age [days]</th>
<th>weight [kg]</th>
<th>Dimensions S [mm x mm]</th>
<th>h [mm]</th>
<th>ultimate load [kN]</th>
<th>resistance [N/mm²]</th>
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<td>158</td>
<td>820</td>
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<td>7</td>
<td>9.20</td>
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<td>158</td>
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<td>9.25</td>
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<td>158</td>
<td>820</td>
<td>32.8</td>
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<td>1240</td>
<td>49.7</td>
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<td>1205</td>
<td>48.3</td>
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<td>35</td>
<td>9.30</td>
<td>158 x 158</td>
<td>158</td>
<td>1150</td>
<td>46.1</td>
</tr>
</tbody>
</table>

### 4.1.3 Testing procedure

In a first stage a static load $P$ was progressively applied at the end of the test specimen to produce the same stresses in the longitudinal rebars at the intermediate support as determined for the
reference bridge under the unfrequent moment (see 5.1.4). This implies the application of a minimum load of 710kN. As no cracks appeared in the concrete slab the load was enhanced to 1000 kN because the fatigue test should have started on a cracked concrete slab. The cracks appearing in the concrete slab were recorded by marking the position and the corresponding load level.

The global bending test was repeated in order to obtain the stiffness of the cracked specimen but eliminating the effects of the settlement of the test set-up and the degradation due to the cracking concrete from the measurement before.

Then a load $P_1$ was progressively applied up to 150 kN while $P$ remained constant. Then the actuator was deloaded and the same was realized for load $P_2$. Some loadings on a lower level were performed to achieve stabilisation.

During the first part of the fatigue test $\Delta P_1$ varied between 20 and 100 kN while $\Delta P_2$ varied in the same load range but opponent to $\Delta P_1$. After 2 900 000 cycles at a frequency of 2.7 Hz the test was interrupted and the static tests were repeated to check the stiffness of the girder and the slab.

The static test was repeated again but with a maximum load of 320 kN for $P_1$ and $P_2$. The load range was increased as well. For the second part of the fatigue test $\Delta P_1$ and $\Delta P_2$ varied between 20 and 300 kN. The test was stopped after 860 000 cycles. Table 4-4 gives a listing of the aforementioned stages of the test.

<table>
<thead>
<tr>
<th>Table 4-4 testing procedure</th>
</tr>
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<tbody>
<tr>
<td>stage</td>
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<td>0b</td>
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<td>0c</td>
</tr>
<tr>
<td>1a</td>
</tr>
<tr>
<td>1b</td>
</tr>
<tr>
<td>1c</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3a</td>
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<tr>
<td>3b</td>
</tr>
<tr>
<td>3c</td>
</tr>
<tr>
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<td>6a</td>
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<tr>
<td>6b</td>
</tr>
<tr>
<td>6c</td>
</tr>
</tbody>
</table>

Figure 4-3 shows a schematic presentation of the fatigue load history.
4.1.4 Instrumentation and measurements

The next figures show the arrangement of strain gauges and displacement transducers.

Fig. 4-3: schematic presentation of the fatigue load history

Fig. 4-4: arrangement of strain gauges on the lower longitudinal reinforcement
Fig. 4-5: arrangement of vertical displacement transducers

Fig. 4-6: arrangement of the strain gauges on the beam flanges (longitudinal direction)

Fig. 4-7: arrangement of horizontal displacement transducers
Furthermore the opening of the cracks in the vicinity of the working load was recorded before and after the static tests.

It should be noticed that in the beginning it was intended to repeat the test for the joints situated 1.20 m to both sides from the intermediate support. Therefore measuring devices were also foreseen at these locations. However after the test on the first joint the constitution of the slab and reinforcement did not allow for any further tests.

### 4.1.5 Numerical analysis of the slab

To obtain the nominal stress range in the slab under the pair of alternating loads $\Delta P_1$ and $\Delta P_2$ a numerical model of the slab was generated by using 8-node thick shell elements. The slab was simply supported at the ends of the shorter span with a spacing of 3.00 m which corresponds to the distance between the centrelines of the profiles. For the first calculation isotropic material properties were chosen. A second analysis was performed with orthotropic material properties to show in how far the stiffness according to different degrees of reinforcement and cracked concrete affect the distribution of internal forces and moments. The stiffnesses for the two principal directions $x$ and $y$ were determined in alignment with EC4, 7.6.2.2. assuming the mean value of the cracked and uncracked section. For the finite element analysis equivalent moduli of elasticity were implemented for the directions $x$ and $y$ to achieve same stiffnesses at a constant shell thickness. It was decided to neglect the permanent load $P$ as the influence is supposed to be marginal for the moment range.

Figure 4-8 shows the positions for the readout of stresses for determining the relevant bending moment range. Supposing the compression strut in the concrete acting under an angle of 45° in the cracked x-direction the relevant position for $\Delta m_x$ is situated 10cm beside the joint according to the distance between the upper and lower reinforcement which is 10 cm as well.

![Fig. 4-8: positions of the relevant bending moment ranges](image)

Table 4-5 gives the relevant bending moment ranges corresponding to the appropriate load ranges for isotropic and orthotropic material properties.

<table>
<thead>
<tr>
<th>type of calculation</th>
<th>load range</th>
<th>$\Delta m_x$ [Nmm/mm]</th>
<th>$\Delta m_y$ [Nmm/mm]</th>
</tr>
</thead>
<tbody>
<tr>
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<td>$\Delta P_1 = 20kN , ? , 160kN$</td>
<td>10267</td>
<td>27008</td>
</tr>
<tr>
<td></td>
<td>$\Delta P_2 = 160kN , ? , 20kN$</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\Delta P_1 = 20kN , ? , 320kN$</td>
<td>22000</td>
<td>57875</td>
</tr>
<tr>
<td></td>
<td>$\Delta P_2 = 320kN , ? , 20kN$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>orthotropic</td>
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<td>8517</td>
<td>27008</td>
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<td></td>
<td>$\Delta P_2 = 160kN , ? , 20kN$</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>$\Delta P_1 = 20kN , ? , 320kN$</td>
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<td>57875</td>
</tr>
<tr>
<td></td>
<td>$\Delta P_2 = 320kN , ? , 20kN$</td>
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</tbody>
</table>
4.1.6 Fatigue resistance according to the codes

According to prEN1992-1, 6.8.2 the stress calculation shall be based on the assumption of cracked cross sections neglecting the tensile strength of the concrete but satisfying compatibility of strains. Beside the uncracked section these sections were already used to determine the stiffness of the orthotropic material. For the longitudinal direction (x-direction) the relevant section is assumed to be composed of the reinforcement alone.

Table 4-6 gives the limit stress range spectra supposing a primary load range between 20 kN and 160 kN for $\Delta P_1$ and $\Delta P_2$ at $2.9*10^6$ load cycles and the damage equivalent stress range indicated by a dot on the relevant S-N-curve.

Table 4-6 presentation fatigue resistances acc. to prEN 1992-1, 6.8.

<table>
<thead>
<tr>
<th>isotropic material properties</th>
<th>orthotropic material properties</th>
</tr>
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<tbody>
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<td>Longitudinal reinforcement</td>
<td>Longitudinal reinforcement</td>
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<td>(isotropic calculation)</td>
<td>(orthotropic calculation)</td>
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<tr>
<td>Transverse reinforcement</td>
<td>Transverse reinforcement</td>
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<td>(isotropic calculation)</td>
<td>(orthotropic calculation)</td>
</tr>
</tbody>
</table>

failure of longitudinal reinforcement  
failure of transverse reinforcement
4.2 Joints in fully prefabricated slabs

4.2.1 Scope

4.2.1.1 Description of the test procedure

The aim of the test was to simulate the behaviour of a composite girder with open joints in the concrete element deck when hogging bending moment is applied. The steel beams was HEA 900, S460 and the concrete was C35/45. Shear connectors was 22 x 100 mm placed in a channel in the concrete slab. The scale of the test specimen was approximately 1:2. Two actuators at the cantilevering end of the composite girder applied the primary load. The joint opening, the displacement between steel and concrete was measured in a number of points, as well as the deflection of the girder.

Several tests were performed on the specimen. The first part was a test of the resistance to fatigue for the overlapping concrete tongues. A working load, with a frequency of 2 Hz and 1 million load cycles was applied at the joint over the mid-span.

In the second part of the test working load was applied at the cantilevering end of the specimen in order to check the resistance to fatigue for the headed shear studs. The load was applied with a frequency of 1,5 Hz and 1 million load cycles. A final test to investigate if the overlapping concrete tongues had suffered from fatigue was also performed by a static load test. The joint at the mid-span was tested and compared with one of the other joints that have not been subjected to fatigue.

4.2.1.2 Aim of the test

The aim of this study is:
- to study how the joints will behave under loading by measuring the joint openings.
- to check if the shear connectors will suffer from fatigue.
- to study the transfer of vertical shear between elements

4.2.2 Calculations

4.2.2.1 Calculation of the frequent load applied over the joint

The frequent wheel load can be taken as 90 kN (EC 1-3, Table 4.6), but it should be increased by partial factor. For concrete there is a material factor γε=1,5. It includes a factor accounting for the difference between test specimen and in situ strength, which is assumed to be 1,2. A further increase of the load is needed because only one test is made. This was chosen to be 1,2. The wheel load should then be:

\[ 90 \times (1,5/1,2) \times 1,2 = 135 \text{ kN} \]

The fatigue load should vary between 5 – 140 kN respectively 140 – 5 kN.

1 000 000 cycles with a frequency of 1,5 Hz.
The area at which the load should be applied was smaller than in EC 1-3. The area used was 200 x 200 mm in order to concentrate the load to the concrete tongues.

### 4.2.2.2 Frequent load for the fatigue test of the shear connectors

The applied load of 245 kN (5-250 kN) at the cantilevering end of the test specimen, gave a shear force on the shear connectors of 116 MPa. The normal stress in the steel beam at the midsection would be about 100 MPa.

The calculations according to the codes, of the predicted fatigue strength are presented in Table 4-7. It shows that the shear connectors are most likely to suffer from fatigue.

<table>
<thead>
<tr>
<th>Parts</th>
<th>Code</th>
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<td>71</td>
<td>3</td>
</tr>
<tr>
<td>Flange near shear connectors</td>
<td>EC 3-2</td>
<td>80</td>
<td>3</td>
</tr>
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<td>Shear connectors</td>
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<td>102 300</td>
</tr>
<tr>
<td></td>
<td>EC 4-2 pr. EN</td>
<td>400 000</td>
</tr>
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</table>

### 4.2.3 Details of the test specimen

#### 4.2.3.1 Description of the test specimen

The test specimen had a length of 8 meter and a width of 3.6 meter. The steel beams were two rolled HEA 900 beams and the steel grade was S460. The concrete slabs were prefabricated with geometry of 200x2000x3600 mm. The shear connectors were headed studs with a diameter of 22 mm and a length of 125 mm.

Between the two main girders cross bracings were applied to prevent lateral torsional buckling. The cross bracings were connected to the stiffeners at the midsupport and at each end of the specimen. The test specimen was designed in such a way that it imitates the part of a real bridge at an intermediate support.

First the steel girders and the secondary beams were assembled. The secondary beams were connected to the stiffeners by bolted joints. The bolts were tightened with at least 450 Nm.
The concrete slabs were assembled on top of the steel girders and pushed together with an axially force, of 400 kN, in order to achieve a tight joint. With this method the average initial gap was 0,15 mm for the joint at the mid-support. The trough in the slabs was filled with in-situ concrete to achieve composite action. A prefabricated concrete slab can be seen in Fig. 4-9: Prefabricated concrete element. In Fig. 4-10: The test specimen the test set-up is shown.

Fig. 4-9: Prefabricated concrete element

Fig. 4-10: The test specimen
4.2.4 Applied loads

![Diagram of applied loads](image)

Fig. 4-11: Applied loads for fatigue test of the overlapping concrete tongues.

In the part of the test of the resistance to fatigue of the overlapping concrete tongues two hydraulic jacks, each with a capacity of 270 kN, were used. They were located at each side of the joint over the mid-support. The jacks applied the load with a phase shift of 180° in order to simulate a passing wheel. In order to simulate the worst case, a static load of 250 kN was applied at the cantilevering end of the test specimen. This applied global moment represents the effect of traffic load on the bridge.

The wheel loads were transferred into the specimen by a steel plate of 200x200x50 mm. The size of the plate is less than recommended according to EC 1-3 (400x400). The reason to use a smaller plate was to concentrate the load to the overlapping concrete tongues. The scale of the test specimen was approximately 1:2 compared to a real bridge. The steel plate was placed in a bedding of high strength gypsum mortar.

![Diagram of hydraulic jack and steel plate](image)

Fig. 4-12: Hydraulic jack over the mid-support
For the test set-up for the fatigue test of the shear connectors the jacks were moved from the midsupport to the cantilevering end of the test specimen. The jacks were now synchronised and applied a load of 245 kN. The applied load resulted, according to a FE-analysis, in a shear force of 44 kN on the shear connectors in the region closest to the mid-support.

![Diagram of composite bridge design](image)

**Fig. 4-13: Applied loads for fatigue test of headed shear studs.**

### 4.2.5 Measurements

#### 4.2.5.1 Determination of material properties

**Table 4-9: Mixing proportions of the concrete used in the tests**

<table>
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<td>1000</td>
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<tr>
<td>Sand 4-16 mm [kg/m$^3$]</td>
<td>619</td>
</tr>
<tr>
<td>Sand 16-24 mm [kg/m$^3$]</td>
<td>180</td>
</tr>
<tr>
<td>Cement Degerhamn [kg/m$^3$]</td>
<td>402</td>
</tr>
<tr>
<td>Cement, std. P [kg/m$^3$]</td>
<td>47</td>
</tr>
<tr>
<td>SIKA FF86 [kg/m$^3$]</td>
<td>4.5</td>
</tr>
<tr>
<td>Micro-air [kg/m$^3$]</td>
<td>3.5</td>
</tr>
<tr>
<td>Cold water [kg/m$^3$]</td>
<td>80</td>
</tr>
<tr>
<td>Hot water [kg/m$^3$]</td>
<td>28</td>
</tr>
</tbody>
</table>

Content of air was 5% at a temperature of 23°C and the density was 2.364 ton/m$^3$.

The compressive strength as well as the tensile strength of the concrete was tested and documented. In total fourteen cubes were tested. The results are presented in Table 4-10.

**Table 4-10: Mean values from cubetests**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>fcc (MPa)</th>
<th>fct (MPa)</th>
<th>Ec,calc (GPa)</th>
<th>$\rho$ (kg/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Element</td>
<td>74.1</td>
<td>4.18</td>
<td>45.8</td>
<td>2290</td>
</tr>
<tr>
<td>Channel 1</td>
<td>50.3</td>
<td>2.81</td>
<td>42.3</td>
<td>2118</td>
</tr>
<tr>
<td>Channel 2</td>
<td>59.7</td>
<td>3.67</td>
<td>43.9</td>
<td>2169</td>
</tr>
</tbody>
</table>
The geometry of the tensile coupons and the results from the tensile tests are presented in this chapter. The strain rate was 0.0025 1/sec up to maximum load and after that point and to failure the strain rate was 0.008 1/sec. The tensile coupons was cut out from the cross-section as shown in Fig. 4-14.

Fig. 4-14: Notations for the tensile coupons and cut-out locations.

Table 4-11: Geometry and results from the tensile tests, nm= not measured

<table>
<thead>
<tr>
<th>Prov</th>
<th>t (mm)</th>
<th>b (mm)</th>
<th>S0 (mm2)</th>
<th>L0 (mm)</th>
<th>LC (mm)</th>
<th>fy (MPa)</th>
<th>fu (MPa)</th>
<th>E (MPa)</th>
<th>A120 (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FL 1:1</td>
<td>28,12</td>
<td>20,12</td>
<td>567,2</td>
<td>134,6</td>
<td>170,3</td>
<td>531,6</td>
<td>655,3</td>
<td>2.02⋅10^5</td>
<td>nm</td>
</tr>
<tr>
<td></td>
<td>28,10</td>
<td>20,10</td>
<td>563,5</td>
<td>134,1</td>
<td>169,7</td>
<td>555,3</td>
<td>660,5</td>
<td>2.05⋅10^5</td>
<td>26,06</td>
</tr>
<tr>
<td></td>
<td>28,15</td>
<td>20,08</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FL 1:2</td>
<td>28,10</td>
<td>20,03</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>28,08</td>
<td>20,03</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>28,21</td>
<td>20,03</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FL 2:1</td>
<td>28,12</td>
<td>20,03</td>
<td>563,3</td>
<td>134,1</td>
<td>169,7</td>
<td>546,3</td>
<td>657,9</td>
<td>1.99⋅10^5</td>
<td>25,64</td>
</tr>
<tr>
<td></td>
<td>28,14</td>
<td>20,03</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td></td>
<td>28,10</td>
<td>20,03</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FL 2:2</td>
<td>28,11</td>
<td>35,03</td>
<td>986,6</td>
<td>177,5</td>
<td>224,6</td>
<td>nm</td>
<td>nm</td>
<td>nm</td>
<td>nm</td>
</tr>
<tr>
<td></td>
<td>28,10</td>
<td>35,10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>28,16</td>
<td>35,11</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LIV 1</td>
<td>16,20</td>
<td>29,99</td>
<td>486,4</td>
<td>124,6</td>
<td>157,7</td>
<td>484,2</td>
<td>621,1</td>
<td>1.96⋅10^5</td>
<td>22,98</td>
</tr>
<tr>
<td></td>
<td>16,23</td>
<td>30,03</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>16,20</td>
<td>30,00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LIV 2</td>
<td>16,16</td>
<td>30,04</td>
<td>485,4</td>
<td>124,5</td>
<td>157,5</td>
<td>465,8</td>
<td>620</td>
<td>1.99⋅10^5</td>
<td>22,78</td>
</tr>
<tr>
<td></td>
<td>16,15</td>
<td>30,03</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>16,16</td>
<td>30,05</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LIV 3</td>
<td>16,12</td>
<td>30,03</td>
<td>485,4</td>
<td>124,5</td>
<td>157,5</td>
<td>479</td>
<td>621,1</td>
<td>1.96⋅10^5</td>
<td>22,88</td>
</tr>
<tr>
<td></td>
<td>16,15</td>
<td>30,07</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>16,16</td>
<td>30,10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

4.2.5.2 Determination of geometrical properties of the sections

Before the test the geometrical properties were measured. The measured geometry of the specimen are presented in Table 4-12.

Fig. 4-15: Determination of geometrical properties of the sections
Centre girder 1 – Centre girder 2  
\[
\begin{align*}
C_1 &= 3006 \\
C_2 &= 3006 \\
C_3 &= 3006 \\
\end{align*}
\]

Diagonal  
\[
\begin{align*}
D_1 &= 8453 \\
D_2 &= 8449 \\
\end{align*}
\]

\[
\begin{align*}
L_{\text{Tot, Girder 1}} &= 7990 \\
L_{\text{Tot, Girder 2}} &= 7990 \\
\end{align*}
\]

Table 4-12: Geometry of the girders

<table>
<thead>
<tr>
<th>Girder</th>
<th>bfl,u</th>
<th>bfl,l</th>
<th>tfi,u</th>
<th>tfi,l</th>
<th>hw</th>
<th>tw</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>304,7</td>
<td>305,2</td>
<td>30,2</td>
<td>29,7</td>
<td>893</td>
<td>16,1</td>
</tr>
<tr>
<td>B</td>
<td>304,1</td>
<td>303,9</td>
<td>29,6</td>
<td>30,1</td>
<td>893</td>
<td>16,1</td>
</tr>
</tbody>
</table>
4.2.5.3 **Positions of measurements**

All deflections were measured with LVDT-gauges. The three joints were measured in the horizontal direction using LVDT 5, with a measuring range of 5 mm. The joint at the middle section was measured in five different positions, over each girder web, in the middle between the girders and at the middle point between the one in the middle and the one over the girder webs. The other two joints were measured in three different positions, over each girder web and in the middle between the girders.

Additionally the vertical relative deflection between the two elements at the middle section was measured. Also the vertical deflection at the same position was measured relative to the lower steel flanges in order to see how large the deflection of the elements was.

Strain gauges were placed at the steel section at middle support, or actually 100 mm from the middle. It was not possible to apply the gauges closer to the middle due to the stiffener. The gauges were placed 100 mm from the edge of the flange, one at the upper flange and one at the lower flange. Strain gauges were also placed in the middle of one of the element closest to the middle of the specimen, 1000 mm from the middle support, at the upper and the lower flanges.

On the concrete there was applied five strain gauges in the middle of one of the element closest to the middle of the specimen, at the same location as the above mention steel gauges. The first and the fifth was placed 100 mm from the steel flange, one in the middle between the steel girders and one in the middle between the ones 100 mm from the steel flange and the one in the middle between the steel girders. See Fig. 4-17.

In order to try to find a way of measuring the behaviour of the studs strain gauges were applied on the underside of the upper flange and displaced 15 mm in the longitudinal direction from the centre of the shear stud. The bending of the studs would also apply a bending of the flange, and it should be possible to capture that bending by measuring the change of strain in the flange. If the measured strains changes during the test it should indicate that the stud has suffer from fatigue.

The figure below shows the position of the measuring devices on the specimen.
Fig. 4-16: Position of the strain gauges. The gauges are placed on the flanges.

Fig. 4-17: Position of the LVDT gauges. → Represents the LVDT gauges.

Fig. 4-18: Position of the strain gauges for the shear connectors
Fig. 4-19: Strain gauges on the concrete slab

Due to the fact that the shear connectors were not visual inspectable acoustic emission were used to detect crack propagation. Measuring devices were applied with a displacement of 500 mm from the edge of the girder, on one of the two girders composite flange. Two devices were used on each end of the girder, they were applied in the transversal direction in the middle between the edge of the flange and the end of the radius. In Fig. 4-20 one of the devices is shown.

Fig. 4-20: Devices for measuring acoustic emission.
4.3 Evaluation of test results

4.3.1 Results from test on joints in partially prefabricated slabs

4.3.1.1 Observations and failure mode

To determine the occurrence of failure as accurately as possible the deformation under the load $\Delta P_1$ was recorded in stage 5 of the test procedure. Regarding the difference between maximum and minimum deformation (Fig. 4-21) failure can be assessed at 666700 cycles at a load range between 20 kN and 300 kN as the stiffness decreased rapidly.

![Graph showing evolution of displacement under load $P_1$](image)

Fig. 4-21: evolution of the displacement under the load $\Delta P_1$

Then the slab was opened in order to determine the reason for the degradation of the stiffness. Whilst only one re-bar of the upper longitudinal reinforcement was cracked in the vicinity of the joint, this applied to the majority of lower transverse reinforcement (located in the prefabricated elements).

![Cracked re-bar of the upper longitudinal reinforcement](image)

Fig. 4-22: cracked re-bar of the upper longitudinal reinforcement

Figures 4-22 and 4-23 show the cracked re-bars.
Figure 4-24 shows the load vs deflection curves of the static tests for the vertical permanent load P. The limits of the 'theoretical range' represent the stiffness for the uncracked composite section and the section with the concrete slab entirely cracked. The tests give a good indication that the composite section was cracked when the fatigue test was started.

Furthermore the curves show that no significant degradation of the stiffness had occured. Especially test 1a before starting the fatigue test and 5a at the end of the test almost show identical behaviour.

Figure 4-25 shows the relative deflections of the static tests for the wheel loads P₁ and P₂. The lines represent similar stiffness after 2 900 000 load cycles at a range of 20 – 160 kN.
Fig. 4-25: deflections of the static tests at the stages 1b, 1c; 3b, 3c; 4b, 4c; 5b, 5c

4.3.1.2 Comparisons with theoretical results

Figures 4-26 and 4-27 show a comparison of the distribution of theoretical stresses and the stresses recalculated from the measured strains. The compliance is better when tension stiffening is taken into account.

Fig. 4-26: comparison of stresses in the profile before the fatigue test

Fig. 4-27: comparison of stresses in the profile after 2 900 000 cycles
Figure 4-28 shows the S-N curve for straight bars and the damage equivalent stress range according to the stress spectrum measured in the test.

![Graph showing stress range spectrum, S-N-curve and damage equivalent stress range.](image)

**Fig. 4-28: stress range spectrum, S-N-curve and damage equivalent stress range**

From the numerical analysis (4.1.5) it can be stated that the material model (isotropic, orthotropic) influences the design of the present slab. Regarding the damage after opening the concrete it can be concluded that the analysis using an orthotropic model allows for a more accurate prediction of the test. With an orthotropic model it is the transverse reinforcement that cracks first whereas with an isotropic model it is the longitudinal reinforcement that governs the fatigue design.
4.3.2 Results from test of joints in full prefabricated slabs

4.3.2.1 Fatigue test of concrete tongues

The test of the overlapping concrete tongues indicated good results when subjected to fatigue load. The horizontal opening of the joint at the midspan remains almost constant through out the test of 1 million load cycles.

The joint over the midsupport had an opening of 1,06 mm due to the fact that the specimen was subjected to a global negative moment. The increase of the horizontal opening of the joint at the midspan due to the fatigue load was 0,073 mm. The average initial opening of the joint at the midspan was 0,15 mm. The total opening in the horizontal direction caused by the loads was 1,13 mm. For the other two joints the horizontal deflection was constant through out the test.

The relative vertical deflection between the two concrete slabs on each side of the joint at the midspan was at the beginning 1,24 mm and at the end of the test, after 1 million load cycles, 1,42 mm. The vertical movement increased with 0,18 mm during the test. I Fig. 4-29 the vertical deflection at the midspan is shown.

![Vertical deflection of concrete tongue at midspan](image)

There were no indications that the concrete tongues had suffered from fatigue during the test. No visible cracks of the concrete tongues or dramatic change of behaviour was found through out the test. After the fatigue test a test was performed to check the resistance to static load of the concrete tongues. The joint at the midspan, that have been subjected to fatigue load, was tested as well as one of the other joints that had not been subjected to fatigue load as a reference. The results confirm that the joint at the midspan had not suffered from fatigue. The maximum static load was about the same for the two tested joints. For the joint at the midspan the maximum load was 1046 kN and for the other joint 1001 kN.
Fatigue test of shear connectors

For this part of the test the horizontal opening of the joints as well as the vertical deflection of the cantilevering end of the specimen was constant during the test. The vertical movement of the concrete slabs was measured at the midspan from 100 000 to 1 million load cycles. Due to technical limitations it was not possible to apply the LVDT-gauge until after 100 000 load cycles. The vertical movement increases with 0,05 mm.

The slip between the steel girders and one of the concrete slab was measured in four different positions, see Fig. 4-30.

![Gauges for measuring of the slip between the steel girders and the concrete slabs.](image)

**Fig. 4-30:** Gauges for measuring of the slip between the steel girders and the concrete slabs.

Due to technical limitations it was only possible to measure the slip between one of the concrete slab and the steel girders. Unfortunately all shear connectors that failed were located in the other slab. Therefore it was not possible to detect any increase in the slip between the slab and the steel girders. The measured slip remained constant throughout the test on a level of 0,13 mm on one side of the specimen and about 0,17 mm on the other side.

![Slip between steel and concrete](image)

**Fig. 4-31:** Slip between the steel girders and the concrete slabs.
The results from the measuring of acoustic emission, AE, and the measuring of the strain caused by the bending of the headed studs indicated that at least three, but probably four, headed studs had failed due to fatigue. Failure for three headed studs was confirmed visually after the end of the test. In Fig. 4-32 to Fig. 4-33 the result from the AE measuring after 400 000 and 1 million load cycles are shown. On the x-axis the number of cycles are shown and the y-axis shows the signal. The signal has no unit, but can be seen as a measure of the emitted energy from a crack. If the value for the emitted energy increases then is the activity of the crack also higher.

**Fig. 4-32: 400 000 load cycles, channel 2.**

The figure shows the emitted energy from a crack. The signal after 85 000 collected values shows a propagating crack. The energy is about 400. In Fig. 4-33 the energy for the same crack after 1 million cycles is shown. It can be clearly seen that the emitted energy is much higher, about 11 000. This indicates that the crack is propagating over the load cycles. The position of the propagating crack is close to two of the headed studs at the midspan.

**Fig. 4-33: 1 015 000 load cycles, channel 2.**
The measured strains caused by the bending of the headed studs shows the same tendency as the AE-measuring, that at least three headed studs had failed due to fatigue. The measured strains also indicate that probably one more studs had failed. As can be seen from Fig. 4-34 headed studs number 1 to 3 has a constant strain through out the test, but stud number 4 shows a clearly increase of the strain. The probable explanation of the increasing strain is that the concrete surrounding the headed studs has been crushed, and due to that the horizontal force climbs upwards on the shaft of the headed stud. This will cause an increase of the bending of the studs and will cause failure from fatigue at some time. Fig. 4-35 shows the headed studs on each side of the midspan, on all those studs the strain in the flange were measured. Headed studs 4, 8 and 16 had a visually confirmed failure to fatigue and probably also stud 13 which had a similar behaviour concerning the measured strain as studs 4, 8 and 16. Fig. 4-36 shows photos of the cracked surface for studs 4 and 8.

![Measuring strains in the steel flange](image)

**Fig. 4-34: Measured strains in the steel flange**
The measured strains in the steel beams, at a distance of 1000 mm from the joint at the midspan, were 281 respectively 274 microstrain for the composite flange for girder B respectively girder A. For the noncomposite flange the strains were –316 and –315 microstrain respectively. The differences of the measured strains for the composite and the noncomposite flange correspond to a stress of 10,5 MPa for girder A and 7,6 MPa for girder B. The measured strains in the concrete slab correspond to stresses 0,43 MPa, 0,38 MPa, 0,47 MPa, 0,54 MPa and 0,49 MPa for strain gauges 1 to 5. Gauges 1 were located near girder A and gauges 5 near to girder B. Gauges 2-4 were located between gauges 1 and 5.
4.4 Conclusions

4.4.1 Partially prefabricated concrete slabs

The behaviour of a multi-span composite bridge with a partially prefabricated slab was investigated in laboratory. The test simulated the region at an intermediate support in permanent hogging. A pair of alternating loads representing a passing wheel was applied at the joint between the prefabricated elements at mid-span of the slab. The test itself, the accompanying analytical works and the fabrication of the specimen allow for the conclusions following:

- As the slab is stressed into the longitudinal direction the cracking of the concrete can be supposed to reduce the corresponding stiffness when vertical loads are applied. For a more accurate prediction of the relevant stress range this aspect was taken into account by reducing the stiffness for the determination of the internal forces and moments. Orthotropic shell elements were used in the numerical model implementing the recommendation for composite slabs in EC4, 7.6.2.2.

- The result of the fatigue test is in good compliance with the provisions given in EC2, 6.8.

- The joint between the slab elements usually provides a gap during concreting. This gap acts like a drain for the liquid concrete such that the concrete aggregates remain in the region of the joint and cement flows off partially. This aspect affects the properties of the slab locally as the material can be supposed to be more brittle which is likely to favour fatigue failure. The gap should therefore be reduced to a minimum to avoid significant discontinuities in the concrete quality.

- In the test the joint between the elements tend to open. As the lower longitudinal reinforcement is located directly on the slab the opening reduces the corrosion resistance. Regarding the aforementioned prompt it is therefore recommended to close properly the gap between the elements before concreting the slab.

4.4.2 Fully prefabricated concrete slabs

In this part of the project the behaviour of multiple span bridges with fully prefabricated concrete slabs has been investigated in laboratory.

The conclusions that can be drawn from this part of the project is the following:

- The overlapping concrete tongues can clearly resist the fatigue load as well as the static load according to EC 1-3.

- For a continuous bridge the joint tend to open over intermediate supports. The total horizontal opening during the test was 1.1 mm. The total vertical movement of the concrete slabs relatively each other, caused by a passing wheel, over an intermediate support was 1.4 mm during the test. This movement of the joint may cause problem for the waterproofing and surfacing and this has to be checked.

- Even if the elements in tension near the support will not be included in the design calculations, it is strongly recommended that they should be properly connected to the steel girders. The connection should be as strong as possible without exceeding the tensile strength of the concrete. The force on the connectors will vary with the traffic load and may cause fatigue failure. The laboratory test with 1 million load cycles caused three studs, but probably four, to
fail from fatigue. The calculated force on the studs of 44 kN is high compared to the expected resistance to fatigue according to EC 4-2 for 1 million cycles.

- Acoustic emission was measured and it was possible to detect the propagation of cracks of the headed studs.

4.5 References


5  **FATIGUE TESTS ON SPECIAL JOINTS OF BEAMS**

5.1  Preliminary investigations

5.1.1  *Description of the test procedure*

Four fatigue tests on special joints of beams have been tested in the present project. The joints tested are those needed between beams in laminated sections used in multi beam bridges. Test 1 and 2 concern connection in span and test 3 and 4 concern connection on support.

For the structural design of a concrete cross girder section (test 3 and 4), the general design, in order to limit the stress in the concrete, is shown in Fig. 5-1.

![Fig. 5-1: Concrete cross girder with thick cap plates](image)

To connect two girders of multi-beam bridges to each other with a bolted connection (Fig. 5-2) in span (test 1 and 2) there are several positions: these positions are limited on the one hand by the zero crossover of the maximum and on the other hand by the zero crossover of the minimum moment process.

If the connection is in the area of the positive moment distribution, the failure occurs in the bolts, the cover plates or the web of the girder. If the connection is in the area of the negative moment distribution, the failure may occur in the reinforcement of the concrete slab.

In the test program both possibilities have been tested.
5.1.2 Aims of the tests

The aims of the tests is to know the fatigue behaviour of the connection performed in large specimen with actual dimensions. The results should give information for the design of multi-beam bridges, and particularly the best position of the joint: in span or on support.

Tests 1 (Fig. 5-3) and 2 (Fig. 5-4), concerning connection in span, ones tested under positive moment (slab in compression)

and ones tested under negative moment (slab in tension),
shall give information about the fatigue behaviour in relation, if relevant, with slip of the cover plates.

Tests 3 (Fig. 5-5) and 4 (Fig. 5-5), concerning connection on support,

shall give information about the fatigue behaviour of the lower flange, in compression, for two designs of the welds on cap plate: fillet welds or full penetration welds.

5.1.3 Applied loads

The loads applied during the tests are described in the test performing of every test itself.

Test 1 and 2, where cover plates with prestressed bolts are applied only on the webs, two loads have been applied in order to produce bending in the connection without shear. The fatigue test has been performed under a range load as high as possible. The maximum load produced the elastic design moment in the connection in order to avoid slip in the connection.

In test 1, the concrete slab is always in compression and in test 2 the concrete slab is always in tension and cracked before starting the fatigue test.

At test 3 and 4 the maximum load corresponds to the infrequent load obtained in the reference bridge. The range load has been defined in order to produce the frequent load range obtained in the reference bridge.
5.1.4 Reference Project

In order to determine the geometry of the test specimens and the load conditions, a reference bridge has been calculated.

5.1.4.1 Type of bridge

The bridge with a total width of 11.5 m (Fig. 5-6) comprises two carriage lanes. The four steel beams are continuous on two spans of 28 meters length each. The span length of 28 meters has been chosen because it is the highest length compatible with the use of profiles HE 900A.

Fig. 5-6: Type of the reference bridge

5.1.4.2 Materials

5.1.4.2.1 Steel

Profiles HEA 900 have been used for the tests.

The characteristic of the profiles HE 900 A are:

- Sectional area : \( A = 321 \text{ cm}^2 \)
- Depth of section : \( h = 890 \text{ mm} \)
- Weight per meter : \( G = 252 \text{ kg/m} \)
- Moment of inertia : \( I_y = 422,100 \text{ cm}^4 \)
- Elastic section modulus : \( W_y = 9,480 \text{ cm}^3 \)
- Plastic section modulus : \( W_{pl} = 10,810 \text{ cm}^3 \)
The steel grade S 460 has been used with:

- Yield strength: \( f_{yk} = 460 \) N/mm\(^2\)
- Design strength: \( f_{yd} = 418 \) N/mm\(^2\)
- Ultimate strength: \( f_u = 550 \) N/mm\(^2\)
- Modulus of elasticity: \( E_s = 205,000 \) N/mm\(^2\)

Following EC 3, the fatigue resistance of a structural element is defined by an SN curve and the fatigue resistance under constant amplitude \( \Delta \sigma_c \) corresponds to \( N_C = 2 \times 10^6 \) cycles (Fig. 5-7). \( \Delta \sigma_D = 0.737 \Delta \sigma_c \) the fatigue limit under constant amplitude, corresponds to \( N_D = 5 \times 10^6 \) cycles.

In bridges which have been studied here, fatigue assessment is needed for five details:

- Rolled beams on each point: \( \Delta \sigma_c = 160 \) N/mm\(^2\)
- Bolted connections: \( \Delta \sigma_c = 112 \) N/mm\(^2\)
- Connections stiffener-flange: \( \Delta \sigma_c = 80 \) N/mm\(^2\)
- Connection profile-cap plate (full penetration weld): \( \Delta \sigma_c = 71 \) N/mm\(^2\)
- Connection profile-cap plate (fillet weld): \( \Delta \sigma_c = 36 \) N/mm\(^2\)
- Stud connectors: \( \Delta \sigma_c = 80 \) N/mm\(^2\)
5.1.4.2.2 Concrete

The concrete quality following EC 2 is C35/45 with:
- Compressive strength $f_{ck} = 35 \text{ N/mm}^2$
- Design strength $f_{cd} = 23.33 \text{ N/mm}^2$
- Modulus of elasticity : $33,500 \text{ N/mm}^2$
- Thickness $h_c = 200 \text{ mm}$

5.1.4.2.3 Reinforcement

The steel grade reinforcement is steel S500 with:
- Yield strength : $f_{sk} = 500 \text{ N/mm}^2$
- Design strength : $f_{sd} = 435 \text{ N/mm}^2$

5.1.4.2.4 Bolts

The bolts used are M27 grade 10.9, with:
- Yield strength : $f_{yb} = 900 \text{ N/mm}^2$
- Design strength : $f_{ub} = 1,000 \text{ N/mm}^2$
- Hole diameter $\phi = 30 \text{ mm}$
- Tensile stress area $A_s = 459 \text{ mm}^2$
- Gross cross-section area $A = 573 \text{ mm}^2$

5.1.4.2.5 Loads

5.1.4.2.5.1 Dead loads

Dead loads are calculated taking into account specific weight of material:
- Steel : 78.5 kN/m$^3$ (HE 900 A : 2.52 kN/m)
- Concrete : 25 kN/m$^3$ (concrete slab : 5 kN/m$^2$)
- Asphalt surface : 23 kN/m$^3$ (for a thickness of 0.09 m : 2.07 kN/m$^2$)

The characteristic traffic loads are those prescribed in the EC 1.3 - 4.3.2: Traffic loads on bridges:
- 9 kN/m$^2$ on lane 1 of 3 meters of width
- 2.5 kN/m$^2$ everywhere else
- One tandem of 300 kN on the lane 1
- Another tandem of 200 kN on lane 2

These traffic loads are applied on one span in order to know the maximum bending moment in span, but they are applied on the two spans in order to know the maximum bending moment on the middle support.
5.1.4.2.5.2 Fatigue loads

We consider as frequent loads the fatigue load model 1 (FLM1) given in EC 1.3 - 4.6.2.

This fatigue load model has the configuration of the main loading system (5.1.4.2.5.1). The values of the axle loads are equal to 0.7 $Q_{\text{ik}}$ and the values of the uniform distributed loads are equal to 0.3 $q_{\text{ik}}$.

If the stress range given by these loads is below the fatigue limit $\Delta \sigma_D$, no fatigue damage is expected.

In the other case, the equivalent load is given by the fatigue load model 3 (FLM3) given in EC 1.3 - 4.6.4. This load model allows the calculation of the fatigue life.

5.1.4.2.6 Bending moment in the middle beam

5.1.4.2.6.1 Joint in span

The Fig. 5-8 shows the envelope bending moment, without any partial safety factor, if the beams are continuous on the middle support (as test 1 and 2, joint in span). This figure shows also the design resistance moment of the sections.

The bending moments have been determined with the HE 900 A and the concrete slab in span, and with the HE 900 A and 1.96% of steel reinforcement in the slab (on the upper side of the concrete slab, there are $\phi 20$ with a spacing of 10 cm and on the lower side of the concrete slab there are $\phi 10$ with a spacing of 10 cm) on the support.

Fig. 5-8: Moment distribution for the joint in span
Considering the partial safety factors, the elastic resistance moment \( M_{Rd,elast} \) in span is larger than the applied moment \( M_{Sd} \). And on support it has to use the plastic design in order to obtain \( M_{Rd,plast} > M_{Sd} \), but following EC4 (Table 4.2), the section of HE 900 A (on middle support) corresponds to Class 3, where plastic design is not allowed. So, there is a problem to solve and the tests will help to define the right design moment.

The figure also shows that, if the joint of beams is between 17.5 meters and 24 meters, the joint is submitted to alternative bending moment. Beyond 24 meters, there is only negative bending moment.

The frequent moment range at 21 meters, according to FML1 [EC 1-3 - 4.6.2], is equal to:

\[
\Delta M_f = 0.7(Q_u) + 0.3(q_d) = 1569 \text{ kNm}
\]

(5–1)

5.1.4.2.6.2 Joint on middle support

The Fig. 5-9 shows the envelope bending moment, without any partial safety factor, if the beams are simply supported under the slab load (test 3 and 4, joint on middle support). This figure shows also the design resistance moment of the sections.

The bending moments have been determined with the HE 900 A and the concrete slab in span, and with the HE 900 A and 1.40\% of steel reinforcement in the slab (on the upper side of the concrete slab, there are \( \phi 16 \) with a spacing of 10 cm and on the lower side of the concrete slab there are \( \phi 10 \) with a spacing of 10 cm) on the support.

![Fig. 5-9: Moment distribution for the joint on middle support](image-url)
Considering the partial safety factors, the plastic design is acceptable in this case because following EC4 (Table 4.2), the section of HE 900 A (in span) corresponds to Class 2, where plastic design is allowed.

In this case elastic design and also plastic design are acceptable.

The frequent moment range on middle support, according to FML1 [EC 1.3 - 6.4.2], is equal to:

\[ \Delta M_t = 0.7 \left( Q_{k} \right) + 0.3 \left( q_{k} \right) = -1,155 \text{ kNm} \]  \hspace{1cm} (5–2)

\[ M_{\text{min}} = M_g = -608 \text{ kNm} \]

Under this frequent moment range, the frequent stress range in the steel rebar reaches:
- on the upper side of the concrete slab: \( \Delta \sigma_{\text{rebar}} = 80 \text{ N/mm}^2 \)
- on the lower side of the concrete slab: \( \Delta \sigma_{\text{rebar}} = 68 \text{ N/mm}^2 \)

The unfrequent moment on the support is equal to

\[ M_{uf} = M_g + 0.8 M_q = -2,620 \text{ kNm} \]  \hspace{1cm} (5–3)

5.1.4.2.6.3 Comparison

If the concrete load produced any moment on support, it appears that the maximum moment in span increases from 4,120 kNm (Fig. 5-8) to 5,030 kNm (Fig. 5-9) and the minimum moment on support increase from –4,797 kNm (Fig. 5-8) to –3,123 kNm (Fig. 5-9).

Comparing these values with the design resistance moments, it can be remarked that the second solution allows longer spans than the first solution if using the same steel section all along the span.

5.1.5 Calculation of test specimen

5.1.5.1 Materials

For the tests the same materials as in 5.1.4.2 have been used.

5.1.5.1.1 Steel

Profiles HE 900 A S460 have been used for the tests.

5.1.5.1.2 Concrete

The concrete quality following EC 2 is C35/45, with a thickness of 200 mm.

The steel grade reinforcement is steel S500.

5.1.5.1.3 Bolts

The bolts have been M27 (10.9) with a diameter of 30 mm.
5.1.5.1.4 Stud connectors

The stud connectors have been 22 x 125mm S235.

The design shear resistance of a headed stud automatically welded is the minimum of:

\[ P_{Rd} = 0.8 \frac{f_u}{\gamma_v} \left( \frac{\pi d^2}{4} \right) \quad [EC \ 4.2 \ - \ 6.13] \quad (5–4) \]

\[ \text{or} \quad P_{Rd} = \frac{0.29 \alpha d^2}{\gamma_v} \left( \frac{f_{ck}}{E_{cm}} \right) \quad [EC \ 4.2 \ - \ 6.14] \quad (5–5) \]

The resistance of one stud connector 22 x 125 results to \( P_{Rd} = 88 \) kN

5.1.5.2 Test N° 1: Joint in span, positive moment

\[ \begin{array}{c}
\text{Fig. 5-10: Joint in span, positive moment} \\
\end{array} \]

\[ \begin{array}{c}
\text{Fig. 5-11: Details of the joint} \\
\end{array} \]
5.1.5.2.1 Scope

The scope of the test concerns the analysis of the resistance of the bolted joint under positive moment, submitted to fatigue loads.

Therefore the static resistance of 3 sections (bolt, cover plate, web) has been calculated.

Following EC3 - 6.1.3 , the resistance of the connection shall be determined on the basis of the resistances of the individual fasteners or welds:

“6.3 Joints loaded in shear subject to vibration and/or load reversal
(1) Where a joint loaded in shear is subject to impact or significant vibration, either welding or else bolts with locking devices, preloaded bolts, injection bolts or other types of bolts which effectively prevent movement shall be used.

(2) Where slipping is not acceptable in a joint because it is subject to reversal of shear load (or for any other reason), either preloaded bolts in a slip-resistant connection (Category B or C), fitted bolts or welding shall be used.”

The design of the bolted connection loaded in shear shall conform to Category B : Slip-resistant at serviceability limit state.

“In this category preloaded high strength bolts with controlled tightening shall be used. Slip shall not occur at the Serviceability Limit State (SLS). The design serviceability shear load should not exceed the design slip resistance, obtained from 5.1.5.2.2.2. Slip is accepted in the Ultimate Limit State (ULS). The design ultimate shear load shall not exceed the design shear resistance nor the design bearing resistance, obtained from 5.1.5.2.2.1.” [EC 3 - 6.5.3.1]

5.1.5.2.2 Resistance of the bolts

5.1.5.2.2.1 Ultimate Limit State (ULS) : Design resistance of one bolt

Shear resistance

If the shear plane passes through the unthreaded portion of the bolt, the shear resistance [EC3 - 6.5.3] results to:

\[ F_{V,Rd} = n \left( \frac{0.6 f_{ub} A}{\gamma_{Mb}} \right) = 2 \left( \frac{0.6 \times 1000 \times 573}{1.25} \right) = 550 \text{ kN} \]  \hspace{1cm} (5–6)

\[ n = \text{number of the shear planes} \]

Bearing resistance

The bearing resistance [EC3 – 6.5.3] results to:

\[ F_{b,Rd} = \frac{2.5 \alpha f_u d t}{\gamma_{Mb}} = \frac{2.5 \times 0.778 \times 550 \times 27 \times 16}{1.25} = 370 \text{ kN} \]  \hspace{1cm} (5–7)
- $\alpha$ is the smallest of: \( \frac{e_1}{3d_0}; \frac{p_1}{3d_0}; \frac{1}{4}; \frac{f_{ub}}{f_u} \) or 1
- here: \( e_1 = 70 \) mm and \( p_1 = 100 \) mm \( \gamma \alpha = 0.778 \)
- \( t_w = 16 \) mm

5.1.5.2.2.2 Serviceability Limit State (SLS): High strength bolts in slip-resistant connections

Slip resistance

The design slip resistance of a preloaded high-strength bolt [EC3 - 6.5.8.1] shall be taken as:

\[
F_{s,Rd} = \frac{k_s \mu}{\gamma_{Ms}} F_{p,cd} = \frac{120.5}{1.1} = 321 = 292 \; \text{kN} \tag{5-8}
\]

- \( F_{p,cd} = 0.7 f_{ub} A_s = 0.7 \times 1000 \times 459 = 321 \; \text{kN} \) \[EC3 – 6.5.8.2\] \tag{5-9}

- \( \mu \) is the slip factor; here: \( \mu = 0.50 \) (class A surfaces: surfaces blasted with shot or grind, with any loose rust removed, no pitting) [EC3 – 6.5.8.3]

- \( n \) is the number of friction interfaces; here: \( n = 2 \)

5.1.5.2.2.3 Resistance moment of the bolts

The resistance once results of the slip resistance of the bolts (5.1.5.2.2.2). In calculation it can be supposed, that the neutral axis is in the centre of the concrete slab. Therefore the resistance of the concrete could be neglected.

---

**Fig. 5-12: Specifications for elastic design**
\[ M_{Rd,\text{elast},A} = F_{r,Rd} \sum y_i^2 \]
\[ y_{\text{max}} = 890 + \frac{200}{2} - 30 - 55 - 60 = 845 \text{ mm} \] (Fig. 5-11, Fig. 5-12)

\[ M_{Rd,\text{elast},B} = N_{Rd,\text{HEA}} y_{\text{HEA}} \]
\[ = 6,848 \times 0.545 = 3,732 \text{ kNm} \]

Once more the resistance results of the bearing resistance of the bolts (5.1.5.2.2.1). In calculation, it can be supposed that the neutral axis is located at the level of the upper steel reinforcement in the concrete slab. Therefore the resistance of the concrete could be neglected.

\[ M_{Rd,\text{plast}} = F_{b,Rd} n \sum y_i \]
\[ = 370 \times 2 \times (0.885 + 0.765 + 0.645 + 0.525 + 0.405 + 0.285) = 2,597 \text{ kNm} \]

\[ y_{\text{max}} = 890 + 140 - 30 - 55 - 60 = 885 \text{ mm} \] (Fig. 5-11, Fig. 5-13)

Also the resistance results of the slip resistance of the bolts (5.1.5.2.2.2).

\[ M_{Rd,\text{elast}} = F_{s,Rd} n \sum y_i \]
\[ = 292 \times 2 \times (0.885 + 0.765 + 0.645 + 0.525 + 0.405 + 0.285) = 2,050 \text{ kNm} \]
5.1.5.2.3 Resistance moment of the web

Here the failure should be expected (following EC 3) in the web on the profile.

Fig. 5-14: Specifications for the stresses in the web

A first assumption has been: \( t_{\text{covpla}} + 0.5 \ d_{\text{wa}} = 12 + 0.5 \ 50 = 37 \text{ mm} \) (5.1.5.2.4)

5.1.5.2.3.1 Full section, elastic design (no slip)

In calculation, it can be supposed that the neutral axis is located in the centre of the concrete slab, so that the resistance of the concrete could be neglected. Therefore, if there is no slip, the resistance moment for elastic design for the full section results to

\[
M_{\text{Rd, elast, f}} = \frac{f_{\text{yk}}}{\gamma_{M0}} \sum k \ A_y \ y_{\text{max}}^2 = \frac{f_{\text{yk}}}{\gamma_{M0}} w_f
\]

\[
= \frac{460}{1.1 \ 0.845} (1 \ \frac{0.6^3}{12} + 0.6 \ 0.545^2 + \frac{1}{\sqrt{3}} \ 0.17 \ (0.845^2 + 0.245^2)) = 2,155 \text{ kNm}
\]

- \( k = 1 \) in tension and \( k = \frac{1}{\sqrt{3}} \) in shear

\[
M_{\text{Rd, elast, f}} = \frac{f_{\text{yk}}}{\gamma_{M0}} \sum k \ A_y \ y_{\text{max}}^2 = \frac{f_{\text{yk}}}{\gamma_{M0}} w_f
\]

\[
= \frac{460}{1.1 \ 0.845} (1 \ \frac{0.674^3}{12} + 0.674 \ 0.545^2 + \frac{1}{\sqrt{3}} \ 0.207 \ (0.882^2 + 0.208^2)) = 2,564 \text{ kNm}
\]

5.1.5.2.3.2 Net section, plastic design (if slip)

In calculation, it can be supposed that the neutral axis is located at the level of the upper steel reinforcement in the concrete slab, so that the resistance of the concrete could be neglected. Therefore, if there is slip, the resistance moment for plastic design for the net section results to
$M_{Rd, plast,a} = \frac{f_{yk}}{\gamma_{M0}} \sum k A_i y_i = \frac{f_{yk}}{\gamma_{M0}} W_n$  \hfill (5–16)

\[
= \frac{460}{1.1} 16 \left(1 \ 5 \ 90 \ 0.585 + \frac{1}{\sqrt{3}} 125 (0.885 + 0.285) \right) = 2,326 \text{kNm}
\]

$M_{Rd, plast,b} = \frac{f_{yk}}{\gamma_{M0}} \sum k A_i y_i = \frac{f_{yk}}{\gamma_{M0}} W_n$  \hfill (5–17)

\[
= \frac{460}{1.1} 16 \left(1 \ 1 \ 67.4 \ 0.585 + \frac{1}{\sqrt{3}} 207 (0.922 + 0.248) \right) = 3,574 \text{kNm}
\]

Considering the reference bridge (5.1.4.2.6.1), this connection may be foreseen in a section beyond 21 meters (Fig. 5-8): $M_{Sd,21 \text{ m}} = 1842 \times 1.35 = 2,487 \text{kNm}$.

### 5.1.5.2.4 Thickness of the cover plates

The elastic moment results to $M_{sd, elast.} = 2,155 (2,564) \text{kNm}$ (5.1.5.2.3.1). In calculation it could be supposed, that the neutral axis is located in the middle of the concrete slab.

\[
Y \frac{f_{yk}}{\gamma_{M0}} = \frac{M_{Rd, elast}}{I_{cov, pla}} y_{max} \Leftrightarrow \frac{460}{1.1} = \frac{2,155 (2,564)}{0.905} \left[ \frac{0.720^3}{12} + \frac{2t \times 0.720 \times 0.545^2}{12} \right] \quad (5–18)
\]

\[
t = 9.5 (11.3) \text{ mm}
\]

- $t$ is the thickness of the cover plates

The plastic moment results to $M_{sd, plast} = 2,326 (3,574) \text{kNm}$ (5.1.5.2.3.2) and in calculation it could be supposed, that the neutral axis is located at the level of the upper steel reinforcement in the concrete slab.

\[
Y A_{net, cov, pla} \frac{f_{yk}}{\gamma_{M0}} y_{max} = M_{Rd, plast} \Leftrightarrow 2t \left(0.720 - 60.030\right) \frac{460}{1.1} 0.585 = 2,326 (3,574) \quad (5–19)
\]

\[
t = 8.8 (13.5) \text{ mm}
\]

- $t$ is the thickness of the cover plate.

Finally, the thickness of the two cover plates should be $t = 13.5 \text{ mm}$.

In the test the thickness of the cover plates has been 12 mm.
5.1.5.2.5 Connection between the concrete slab and the girder

5.1.5.2.5.1 Stud connectors [EC 4.2 - 6.3]

The design shear resistance of a headed stud, automatically welded, is calculated in 5.1.5.1.4.

The resistance of one stud connector 22 x 125 results to $P_{Rd} = 88 \text{ kN}$

The maximum tensile load in the connection is:

\[ F_c = F_{b,Rd} \times n = 370 \times 12 = 4,440 \text{ kN} \]  
\[ (5.1.5.2.2.1) (5– 20) \]

- $n$ is equivalent to the number of the bolts at one girder

$F_c$ should be introduced in the slab by the stud connectors: 3 lines of 24 stud connectors in each line are foreseen

\[ V_{Rd} = 3 \times 24 \times 88 = 6,336 \text{ kN} \geq F_c = 4,440 \text{ kN} \]  
\[ (5–21) \]

5.1.5.2.5.2 Transverse steel reinforcement [EC 4.2 - 6.6]

Following EC 4.2 - 6.6, the transverse reinforcement in the slab has been designed for the ultimate limit state so that premature longitudinal shear failure and longitudinal splitting are prevented:

“The design longitudinal shear per unit length $v_{Sl}$ for any potential surface of longitudinal shear failure within the slab (Fig. 5-15) shall not exceed the design resistance to longitudinal shear $V_{Rd}$ of the shear surface considered.

The length of the shear surface $b-b$ shown in Fig. 5-15 should be taken as equal to $2h$ plus the head diameter for a single row of stud shear connectors, or as equal to $2h + s_t$ plus the head diameter for stud shear connectors arranged in rows, where $h$ is the height of the studs and $s_t$ is the transverse distance centre-to-centre between the studs nearest to the two edges of the flange.

In absence of a more accurate calculation, the design resistance of any surface of potential shear failure in a flange or a haunch should be determined from:

\[ V_{Rd} = 2.5 A_{cv} \times \eta \times \tau_{Rd} + \frac{A_{e} \times f_{sk}}{\gamma_s} \]  
\[ \text{[EC 4.2 - 6.25]} \]  
\[ (5-22) \]
Whichever is smaller, where :

- $\tau_{Rd}$ is the basic design shear strength, to be taken as $0.25 f_{ck} \frac{0.05}{\gamma_c}$, or as zero in a region where the longitudinal shear is determined assuming the concrete to be cracked
- $f_{ck}$ is equal to 35 N/mm²
- $f_{sk}$ is equal to 500 N/mm²
- $\eta$ is equal to 1 for normal-weight concrete
- $A_{cv}$ is the mean cross-sectional area per unit length of beam of the concrete shear surface under consideration
- $A_e$ is the sum of the cross-sectional areas of transverse reinforcement (assumed to be perpendicular to the beam) per unit length of beam crossing the shear surface under consideration (Fig. 5-15) including any reinforcement provided for bending of the slab”

In this test, the steel section which is needed on the upper side of the concrete slab is 74 $\phi$10. On the lower side of the concrete slab, 74 $\phi$16 is needed.

5.1.5.2.6 Fatigue resistance of the test specimen

5.1.5.2.6.1 Loads

During the fatigue test, the bending moment should not be higher than minimum design moment:

$$M_{sd} = 1,406 \text{ kNm}$$

$$\text{Fig. 5-16: Test N}^01$$

The dead load of the specimen reaches 11.52 kN/m and the total dead load is 90 kN.

The bending moment under the dead load reach reaches $11.52 \cdot \frac{8^2}{8} = 92.2 \text{ kNm}$.

The load to apply on the specimen should not be higher than :
\[ \frac{P}{2} = \frac{M_{\text{max}}}{2.8} \iff P_{\text{max}} = \frac{2(1,406 - 92.2)}{2.8} = 940 \text{ kN} \] (5–24)

For a minimum value of \( P_{\text{min}} = 80 \text{ kN} \), \( M_{\text{min}} = 92.2 + 80 \cdot \frac{2.8}{2} = 200 \text{ kNm} \).

The range of the bending moment should reach: \((940 - 80) \cdot \frac{2.8}{2} = 1,200 \text{ kNm}\).

For a load range equal to \((940 - 80) = 860 \text{ kN}\), the moment range reaches 1,200 kNm. It is the maximum range possible to apply on the test specimen.

Considering the reference bridge (5.1.4.2.6.1), this moment range is close to the frequent moment range in the reference bridge \((\Delta M_f = 1,569 \text{ kNm})\).

5.1.5.2.6.2 Preliminary fatigue analyses

If the stress range, which is produced by frequent load, is less than fatigue limit corresponding to \( N_D = 5 \times 10^6 \text{ cycles} \) (Fig. 5-7), fatigue life is unlimited.

So if \( \Delta \sigma_f < \Delta \sigma_D = 0.737 \Delta \sigma_e \), no fatigue damage should occur.

If this is not the case, the fatigue life has to calculate.

If \( \Delta \sigma_e \) is the equivalent stress range produced by the equivalent load, the allowed number of cycles is deduced from the SN curve (Fig. 5-7):

\[ n_e = 5 \times 10^6 \left( \frac{\Delta \sigma_D}{\Delta \sigma_e} \right)^m \] (5–25)

- \( m = 3 \), if \( \Delta \sigma_e > \Delta \sigma_D \)
- \( m = 5 \), if \( \Delta \sigma_e < \Delta \sigma_D \)

The main fatigue problems of this test occur at:

- the lower flange of the steel girder
- the weld of stiffener at the lower flange
- the weld of stud connectors
- the web of the girder (no slip)
- the hole in the web of the girder (slip)
- the cover plate (no slip)
- the hole in the cover plate (slip)
5.1.5.2.6.3 Calculation of the stress range

5.1.5.2.6.3.1 In the beam (between the loads P/2, outside the connection)

**Detail category**: - Rolled beams: \( \Delta \sigma_c = 160 \text{ N/mm}^2 \), \( \Delta \sigma_D = 118 \text{ N/mm}^2 \)

- Flange (weld of stiffener): \( \Delta \sigma_c = 80 \text{ N/mm}^2 \), \( \Delta \sigma_D = 59 \text{ N/mm}^2 \)

- Top flange, weld of stud connectors: \( \Delta \sigma_c = 80 \text{ N/mm}^2 \), \( \Delta \sigma_D = 59 \text{ N/mm}^2 \)

\[\text{Fig. 5-17: Beam section}\]

The moment of inertia of the full composite section reaches \( I = 1,062,000 \text{ cm}^4 \) (the inertia is calculated with a coefficient \( E_s/E_c = 6 \)).

The neutral axis is situated in the web: 80 cm from the bottom of the girder.

Under a moment range of \( \Delta M = 1,200 \text{ kNm} \), the stress range reaches in the full section:

- at the lower flange of the steel profile: \( \Delta \sigma_c = \frac{1,200 \times 10^6 \times 800}{1,062,000 \times 10^4} = 90 \text{ N/mm}^2 < \Delta \sigma_D = 118 \text{ N/mm}^2 \)

  \( \Upsilon \) no fatigue damage should occur

- at the weld of stiffener (lower flange): \( \Delta \sigma_c = \frac{1,200 \times 10^6 \times 770}{1,062,000 \times 10^4} = 87 \text{ N/mm}^2 > \Delta \sigma_D = 59 \text{ N/mm}^2 \)

  \( \Upsilon \) the expected fatigue life should be: \( N = 5 \times 10^6 \left( \frac{59}{87} \right)^3 = 1.56 \times 10^6 \text{ cycles} \)

  (it should be possible to avoid this weld)

- in the top flange (weld of stud connectors): \( \Delta \sigma_c = \frac{1,200 \times 10^6 \times 90}{1,062,000 \times 10^4} = 10 \text{ N/mm}^2 < \Delta \sigma_D = 59 \text{ N/mm}^2 \)

  \( \Upsilon \) no fatigue damage should occur
5.1.5.2.6.3.2 In the connection section (Fig. 5-14)

Detail category: - Rolled beams: $\Delta \sigma_c = 160 \text{ N/mm}^2$, $\Delta \sigma_D = 118 \text{ N/mm}^2$
  - Hole in the web: $\Delta \sigma_c = 112 \text{ N/mm}^2$, $\Delta \sigma_D = 83 \text{ N/mm}^2$

Under a moment range of $\Delta M = 1200 \text{ kNm}$, the stress range reaches:

- in the web and without slip: $\Delta \sigma_e = \frac{1.200 \times 10^6}{6.132 \times 10^3} = 196 \text{ N/mm}^2 > \Delta \sigma_D = 118 \text{ N/mm}^2$

  - $W = \frac{16}{0.845} \left( \frac{0.674^3}{12} + 0.674 \times 0.545^2 + \frac{1}{\sqrt{3}} \times 0.207 \left( 0.882^2 + 0.208^2 \right) \right) = 6.132 \text{ cm}^3$

  (Failure zone b in 5.1.5.2.3, 5.1.5.2.3.1)

  $Y$ the expected fatigue life should be: $N = 1.10 \times 10^6$ cycles.

If the slip resistance is not verified during the fatigue test, a crack may occur in the web at the holes:

- at the hole in the web (net section) if slip: $\Delta \sigma_e = \frac{1.200 \times 10^6}{5.563 \times 10^3} = 216 \text{ N/mm}^2 > \Delta \sigma_D = 83 \text{ N/mm}^2$

  - $W = 16 \left( 1 + 0.585 + \frac{1}{\sqrt{3}} \times 125 \left( 0.885 + 0.285 \right) \right) = 5.563 \text{ cm}^3$

  (Failure zone a in 5.1.5.2.3, 5.1.5.2.3.2)

  $Y$ the expected fatigue life should be: $N = 0.28 \times 10^6$ cycles.

5.1.5.2.6.3.3 In the cover plate

Detail category: - Cover plate: $\Delta \sigma_c = 160 \text{ N/mm}^2$, $\Delta \sigma_D = 118 \text{ N/mm}^2$
  - Hole in the cover plate: $\Delta \sigma_c = 112 \text{ N/mm}^2$, $\Delta \sigma_D = 83 \text{ N/mm}^2$

![Fig. 5-18: Cover plate section](image_url)
The moment of inertia of the composite section (in the cover plate and with a thickness of the cover plate of 12 mm) reaches $I = 493,143 \text{ cm}^4$ (the inertia is calculated with a coefficient $\frac{E_s}{E_c} = 6$).

The neutral axis is situated in the web: 78.3 cm from the bottom of the cover plates.

Under a moment range of $\Delta M = 1,200 \text{ kNm}$, the stress range reaches:

- in the cover plate without slip: $\Delta \sigma_e = \frac{1,200 \times 10^6 \times 783}{493,143 \times 10^4} = 191 \text{ N/mm}^2 > \Delta \sigma_D = 118 \text{ N/mm}^2$

  \[ \text{the expected fatigue life should be: } N = 1.19 \times 10^6 \text{ cycles} \]

If the slip resistance is not verified during the fatigue test, a crack may occur at the holes in the cover plate:

- at the hole in the cover plate (with slip): $\Delta \sigma_e = \frac{1,200 \times 10^6 \times 723}{493,143 \times 10^4} = 176 \text{ N/mm}^2 > \Delta \sigma_D = 83 \text{ N/mm}^2$

  \[ \text{the expected fatigue life should be: } N = 0.53 \times 10^6 \text{ cycles} \]

The expected fatigue life in the cover plate is close to the fatigue life in the web. In order to avoid an earlier fatigue crack in the cover plate, the thickness of the cover plates has been enlarged up to 12 mm (5.1.5.2.4). In this case, the fatigue life in the cover plate without slip should reached $N=1.20 \times 10^6 \text{ cycles}$ and the fatigue life of a hole in the cover plate (with slip) should reached $N = 0.53 \times 10^6 \text{ cycles}$.

5.1.5.2.7 Conclusion

Table 5-1: Summary of the fatigue calculation

<table>
<thead>
<tr>
<th>Position</th>
<th>Expected fatigue life</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lower flange of the girder</td>
<td>No fatigue damage should occur</td>
</tr>
<tr>
<td>Weld of the stiffener of the lower flange</td>
<td>No fatigue damage should occur</td>
</tr>
<tr>
<td>Weld of the stud connectors</td>
<td>No fatigue damage should occur</td>
</tr>
<tr>
<td>Web of the girder (no slip)</td>
<td>$1.10 \times 10^6$</td>
</tr>
<tr>
<td>Hole in the web of the girder (slip)</td>
<td>$0.28 \times 10^6$</td>
</tr>
<tr>
<td>Cover plate (no slip)</td>
<td>$1.19 \times 10^6$</td>
</tr>
<tr>
<td>Hole in the cover plate (slip)</td>
<td>$0.53 \times 10^6$</td>
</tr>
</tbody>
</table>
Finally, if the slip resistance is verified, the fatigue crack is expected in the web at $1.10 \times 10^6$ cycles, and if slip occurs, the fatigue crack should be expected at $0.28 \times 10^6$ cycles.

5.1.5.3 Test N°2: Joint in span, negative moment

5.1.5.3.1 Scope

The scope of the test concerns the analysis of the resistance of the bolted joint under negative moment, submitted to fatigue loads.

The design of the bolted connection loaded in shear shall conform to Category B: Slip-resistant at serviceability limit state (5.1.5.2.1).

N.B.: The bottom flange of the test specimens should be fixed lateral in order to avoid lateral torsional buckling during the test.

5.1.5.3.2 Resistance of the bolts

5.1.5.3.2.1 Ultimate Limit State (ULS): Design resistance of one bolt

See 5.1.5.2.2.1.
5.1.5.3.2.2 Serviceability Limit State (SLS) : High strength bolts in slip-resistant connections

See 5.1.5.2.2.2.

Fig. 5-21: Cross section of Test N²2

The resistance of this section results of the resistance in tension of the rebars in the concrete slab and the bolts (the concrete is fully cracked). First a rebar section of 1,4% (ϕ16 + ϕ10), that is required on the support in the reference bridge (5.1.4.2.6.2), is considered. But due to the reduction of the width of the slab for the test, a higher rebar section has been considered, which corresponds to 1,96% (ϕ20 + ϕ16), in order to obtain \( N_{\text{rebar}} = N_{\text{bolts}} = 12 \times 370 = 4,440 \text{ kN} \). (5.1.5.2.5.1, 5.1.5.3.2.3: \( N_{\varphi 16} + N_{\varphi 20} = 1,573 + 2,457 = 4,030 \text{ kN} \)).

5.1.5.3.2.3 Resistance moment of the bolts

The resistance results once of the slip resistance of the bolts and the elastic resistance of the reinforcement. The neutral axis results of the maximum stress of the lower row of bolts and of the equilibrium of the normal forces. The forces in the reinforcement must be less then the area of the reinforcement multiply with the design strength \( f_{sd} \).

\[
M_{Rd,\text{elast}} = F_{s,Rd} h \sum \sigma_{si} A_{si} \sum y_{i} \tag{5-26}
\]

Fig. 5-22: Progress of stress for elastic design (negative moment)

- n row of bolts at one girder
- \( y_i \) distance of each bolt to the neutral axis
- \( F_s \) force in the reinforcement
- $A_s$ total area of the longitudinal reinforcement
- $f_{sd}$ design strength of the reinforcement

Table 5-2: Calculation of the negative elastic resistance moment

<table>
<thead>
<tr>
<th>Area (cm²)</th>
<th>$y_i$ (mm)</th>
<th>$f_{yd}$ (N/mm²)</th>
<th>$N_{el.}$ (kN)</th>
<th>$M_{el}$ (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M27 : row 1</td>
<td>437.50</td>
<td>-584.00 = 2 $F_s$,$R_d$</td>
<td>-255.50</td>
<td></td>
</tr>
<tr>
<td>M27 : row 2</td>
<td>317.50</td>
<td>-423.80</td>
<td>-134.56</td>
<td></td>
</tr>
<tr>
<td>M27 : row 3</td>
<td>197.50</td>
<td>-263.60</td>
<td>-52.06</td>
<td></td>
</tr>
<tr>
<td>M27 : row 4</td>
<td>77.50</td>
<td>-103.50</td>
<td>-8.02</td>
<td></td>
</tr>
<tr>
<td>M27 : row 5</td>
<td>-42.50</td>
<td>56.70</td>
<td>-2.41</td>
<td></td>
</tr>
<tr>
<td>M27 : row 6</td>
<td>-162.50</td>
<td>216.90</td>
<td>-35.25</td>
<td></td>
</tr>
<tr>
<td>18 $\phi$ 16</td>
<td>-377.50</td>
<td>503.90</td>
<td>-190.22</td>
<td></td>
</tr>
<tr>
<td>18 $\phi$ 20</td>
<td>-447.50</td>
<td>597.30</td>
<td>-267.29</td>
<td></td>
</tr>
</tbody>
</table>

$\sum H = 0 \text{ kN}$ $\sum M = -945 \text{ kNm}$

And: 1) $N_{el,upper \ rebars} = 597.30 \text{ kN}$ $# A_s f_{yd} = 56.52 \times 435 = 2,459 \text{ kN}$
   
   2) $N_{el,lower \ rebars} = 503.90 \text{ kN}$ $# A_s f_{yd} = 36.18 \times 435 = 1,574 \text{ kN}$

With 1.96% of steel reinforcement, the elastic resistance moment is equal to -945 kNm.

Once more the resistance results of the bearing resistance of the bolts and the plastic resistance of the reinforcement ($f_{sd}$). The normal forces must be in equilibrium.

$$M_{Rd,\ plast} = F_{b,Rd} n \sum y_i + A_{si} f_{sd} \sum y_i \quad (5–27)$$
Fig. 5-23: Progress of stress for plastic design (negative moment)

with:  
- \( n \) row of bolts at one girder  
- \( y_i \) distance of each bolt / layer of longitudinal reinforcement to the upper edge of the concrete slab  
- \( A_s \) total area of the longitudinal reinforcement  
- \( f_{sd} \) design strength of the reinforcement

<table>
<thead>
<tr>
<th>Area (cm²)</th>
<th>( y_i ) (mm)</th>
<th>( f_{yd} ) (N/mm²)</th>
<th>( N_{pl} ) (kN)</th>
<th>( M_{pl} ) (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M27: row 1</td>
<td>945</td>
<td>-740</td>
<td>2 ( F_{b,Rd} )</td>
<td>-699.30</td>
</tr>
<tr>
<td>M27: row 2</td>
<td>825</td>
<td>-740</td>
<td></td>
<td>-610.50</td>
</tr>
<tr>
<td>M27: row 3</td>
<td>705</td>
<td>-740</td>
<td></td>
<td>-521.70</td>
</tr>
<tr>
<td>M27: row 4</td>
<td>585</td>
<td>-740</td>
<td></td>
<td>-432.90</td>
</tr>
<tr>
<td>M27: row 5</td>
<td>465</td>
<td>-740</td>
<td></td>
<td>-344.10</td>
</tr>
<tr>
<td>M27: row 6</td>
<td>345</td>
<td>-330</td>
<td></td>
<td>-113.85</td>
</tr>
<tr>
<td>18 ( \phi ) 16</td>
<td>36.18</td>
<td>130</td>
<td>435</td>
<td>1,573</td>
</tr>
<tr>
<td>18 ( \phi ) 20</td>
<td>56.52</td>
<td>60</td>
<td>435</td>
<td>2,457</td>
</tr>
</tbody>
</table>

\[ \Sigma N = 0 \quad \Sigma M = -2,370 \text{ kNm} \]

With 1.96 % of steel reinforcement, the plastic resistance moment is equal to –2,370 kNm.
5.1.5.3.3 Connection between the concrete slab and the girder

5.1.5.3.3.1 Stud connectors [EC 4.2 - 6.3]

The design shear resistance of a headed stud, automatically welded, is calculated in 5.1.5.1.4. The resistance of one stud connector 22 x 125 results to \( P_{Rd} = 88 \text{ kN} \).

The maximum effort in the rebar is: \( F_{Rd} = A_s f_{sd} = (36.18 + 56.52) \times 435 = 4,032 \text{ kN} \).

\( F_{Rd} \) should be introduced in the connection by stud connectors: 2 lines of 24 stud connectors have been expected.

\[
V_{Rd} = 2 \times 24 \times 88 = 4,224 \text{ kN} \quad \$ \quad F_{Rd} = 4,032 \text{ kN}
\]

In order to harmonise the fabrication of the specimens, the same number of stud connectors are foreseen in test 2 as in test 1: 3 x 24 studs (5.1.5.2.5.1).

5.1.5.3.3.2 Transverse steel reinforcement. [EC 4.2 - 6.6]

Following EC 4.2 - 6.6, the transverse reinforcement in the slab has been designed for the ultimate limit state so that premature longitudinal shear failure and longitudinal splitting are prevented:

The same rebar section as calculated in 5.1.5.2.5.2 should be foreseen: 74 \( \phi 10 \) on the upper side of the concrete slab and 74 \( \phi 16 \) on the lower side of the concrete slab.

5.1.5.3.4 Fatigue resistance of the test specimen

5.1.5.3.4.1 Loads

During the fatigue test, the bending moment should not be higher than elastic design moment:

\[
M_{sd} = -945 \text{ kNm} \quad (5.1.5.3.2.3).
\]

\[Fig. 5-24: Test N^2\]

The dead load of the specimen reaches 11.52 kN/m, and the total dead load is 90 kN.

The bending moment under the dead load reaches 11.52 \( \left( -\frac{2.8^2}{2} + \frac{2.4^2}{8} \right) = -37 \text{ kNm} \).
The load to apply on the specimen should not be higher than:

\[
\frac{P}{2} = \frac{M_{\text{max}}}{2.8} \iff P_{\text{max}} = 2 \left( \frac{945 - 37}{2.8} \right) = 649 \text{ kN}
\]

(5–28)

For a minimum value of \( P_{\text{min}} = 120 \text{ kN} \), \( M_{\text{min}} = -37 - 120 \frac{2.8}{2} = -205 \text{ kNm} \).

The range of the bending moment should reach: \(-(649 - 120) \frac{2.8}{2} = -741 \text{ kNm}\).

For a load range equal to 529 kN, the moment range reaches -741 kNm. It is the maximum range possible to apply on the test specimen, if slip should be avoid. But before starting the fatigue test, the first static load must reach a value producing cracks in the concrete slab.

5.1.5.3.4.2 Preliminary fatigue analyses

Same procedure as in 5.1.5.2.6.2.

The main fatigue problems of this test occur at:
- the lower/upper flange of the steel girder
- the weld of stiffener at the lower/upper flange
- the weld of stud connectors
- the upper/lower web of the girder (no slip)
- the upper/lower hole in the web of the girder (slip)
- the upper/lower edge of the cover plate (no slip)
- the upper/lower hole in the cover plate (slip)

5.1.5.3.4.3 Calculation of the stress range

5.1.5.3.4.3.1 In the beam (between the supports)

Detail category:
- Rolled beams: \( \Delta\sigma_c = 160 \text{ N/mm}^2 \), \( \Delta\sigma_D = 118 \text{ N/mm}^2 \)
- Flange (weld of stiffener): \( \Delta\sigma_c = 80 \text{ N/mm}^2 \), \( \Delta\sigma_D = 59 \text{ N/mm}^2 \)
- Top flange, weld of stud connectors: \( \Delta\sigma_c = 80 \text{ N/mm}^2 \), \( \Delta\sigma_D = 59 \text{ N/mm}^2 \)
The moment of inertia of the full composite section if concrete is cracked reaches \( I = 646,442 \text{ cm}^4 \).
The neutral axis is situated in the web : 57 cm from the bottom of the girder.

Under a range moment of \( \Delta M = -741 \text{ kNm} \), the stress range reaches in the full section :

- at the lower flange of the steel profile : \( \Delta \sigma_e = \frac{741 \times 10^6 \times 570}{646,442 \times 10^4} = 65 \text{ N/mm}^2 < \Delta \sigma_D = 118 \text{ N/mm}^2 \)
  \( \Upsilon \) no fatigue damage should occur

- at the upper flange of the steel profile : \( \Delta \sigma_e = \frac{741 \times 10^6 \times 320}{646,442 \times 10^4} = 37 \text{ N/mm}^2 < \Delta \sigma_D = 118 \text{ N/mm}^2 \)
  \( \Upsilon \) no fatigue damage should occur

- at the lower flange (weld of stiffener) : \( \Delta \sigma_e = \frac{741 \times 10^6 \times 540}{646,442 \times 10^4} = 62 \text{ N/mm}^2 > \Delta \sigma_D = 59 \text{ N/mm}^2 \)
  \( \Upsilon \) the expected fatigue life should be : \( N = 4.32 \times 10^6 \) cycles

- at the upper flange (weld of stiffener) : \( \Delta \sigma_e = \frac{741 \times 10^6 \times 290}{646,442 \times 10^4} = 33 \text{ N/mm}^2 < \Delta \sigma_D = 59 \text{ N/mm}^2 \)
  \( \Upsilon \) no fatigue damage should occur

- in the top flange (weld of stud connectors) : \( \Delta \sigma_e = \frac{741 \times 10^6 \times 320}{646,442 \times 10^4} = 37 \text{ N/mm}^2 < \Delta \sigma_D = 59 \text{ N/mm}^2 \)
  \( \Upsilon \) no fatigue damage should occur

5.1.5.3.4.3.2 In the connection section (Fig. 5-14)
The moment of inertia of the composite section (resistance of the web and concrete slab cracked) reaches \( I = 224,474 \text{ cm}^4 \).

The neutral axis is situated in the web : 53.4 cm from the lower row of bolts.
Under a range moment of $\Delta M = -741$ kNm, the stress range reaches:

- upper edge of the web and without slip: $\Delta \sigma = \frac{741 \times 10^6 \times 126}{224,474 \times 10^3} = 42$ N/mm$^2 < \Delta \sigma_D = 118$ N/mm$^2$

  $\Rightarrow$ no fatigue damage should occur

- lower edge of the web and without slip: $\Delta \sigma = \frac{741 \times 10^6 \times 594}{224,474 \times 10^3} = 196$ N/mm$^2 > \Delta \sigma_D = 118$ N/mm$^2$

  $\Rightarrow$ the expected fatigue life should be: $N = 1.01 \times 10^6$ cycles

If the slip resistance is not verified during the fatigue test, a crack may occur in the web at the holes:

- at the upper hole in the web if slip: $\Delta \sigma = \frac{741 \times 10^6 \times 66}{224,474 \times 10^3} = 22$ N/mm$^2 < \Delta \sigma_D = 83$ N/mm$^2$

  $\Rightarrow$ no fatigue damage should occur

- at the lower hole in the web if slip: $\Delta \sigma = \frac{741 \times 10^6 \times 534}{224,474 \times 10^3} = 176$ N/mm$^2 > \Delta \sigma_D = 83$ N/mm$^2$

  $\Rightarrow$ the expected fatigue life should be: $N = 0.52 \times 10^6$ cycles

5.1.5.3.4.3.3 In the cover plate

Detail category: - Rolled beams: $\Delta \sigma_c = 160$ N/mm$^2$, $\Delta \sigma_D = 118$ N/mm$^2$

- Hole in the cover plate: $\Delta \sigma_c = 112$ N/mm$^2$, $\Delta \sigma_D = 83$ N/mm$^2$

![Fig. 5-26: Cover plate section](image)
The moment of inertia of the composite section (in the cover plate) reaches $I = 262,698 \text{ cm}^4$. The neutral axis is situated in the web: 55.4 cm from the bottom of the cover plates.

Under a moment range of $\Delta M = -741 \text{ kNm}$, the stress range reaches:

- upper edge of the cover plate (no slip): $\Delta \sigma = \frac{741 \times 10^6 \times 166}{262,698 \times 10^4} = 47 \text{ N/mm}^2 < \Delta \sigma_D = 118 \text{ N/mm}^2$

  $\nabla$ no fatigue damage should occur

- lower edge of the cover plate (no slip): $\Delta \sigma = \frac{741 \times 10^6 \times 554}{262,698 \times 10^4} = 156 \text{ N/mm}^2 > \Delta \sigma_D = 118 \text{ N/mm}^2$

  $\nabla$ the expected fatigue life should be: $N = 2.15 \times 10^6$ cycles

If the slip resistance is not verified during the fatigue test, a crack may occur at the holes in the cover plate:

- at the upper hole in the cover plate: $\Delta \sigma = \frac{741 \times 10^6 \times 106}{262,698 \times 10^4} = 30 \text{ N/mm}^2 < \Delta \sigma_D = 83 \text{ N/mm}^2$

  $\nabla$ no fatigue damage should occur

- at the lower hole in the cover plate: $\Delta \sigma = \frac{741 \times 10^6 \times 494}{262,698 \times 10^4} = 139 \text{ N/mm}^2 < \Delta \sigma_D = 83 \text{ N/mm}^2$

  $\nabla$ the expected fatigue life should be: $N = 1.06 \times 10^6$ cycles

\textbf{5.1.5.3.5 Conclusion}

\textit{Table 5-4: Summary of the fatigue calculation}

<table>
<thead>
<tr>
<th>Position</th>
<th>Expected fatigue life</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper flange of the girder</td>
<td>No fatigue damage should occur</td>
</tr>
<tr>
<td>Lower flange of the girder *</td>
<td>No fatigue damage should occur</td>
</tr>
<tr>
<td>Weld of the stiffener of the upper flange</td>
<td>No fatigue damage should occur</td>
</tr>
<tr>
<td>Weld of the stiffener of the lower flange *</td>
<td>$4.32 \times 10^6$</td>
</tr>
<tr>
<td>Weld of the stud connectors</td>
<td>No fatigue damage should occur</td>
</tr>
<tr>
<td>Upper edge of the web (no slip)</td>
<td>No fatigue damage should occur</td>
</tr>
<tr>
<td>Lower edge of the web (no slip) *</td>
<td>$1.01 \times 10^6$</td>
</tr>
<tr>
<td>Upper hole in the web of the girder (slip)</td>
<td>No fatigue damage should occur</td>
</tr>
</tbody>
</table>
Finally, if the slip resistance is verified, the fatigue crack is expected in the lower edge of the web of the girder at $1.01 \times 10^6$ cycles, and if slip occurs, the fatigue crack should be expected in the lower hole in the web of the girder at $0.52 \times 10^6$ cycles.

5.1.5.4 Tests N° 3 and 4: Joint on middle support, negative moment

\begin{center}
\begin{tabular}{|l|c|}
\hline
Lower hole in the web of the girder (slip) * & $0.52 \times 10^6$ \\
Upper edge of the cover plate (no slip) & No fatigue damage should occur \\
Lower edge of the cover plate (no slip) * & $2.15 \times 10^6$ \\
Upper hole in the cover plate (slip) & No fatigue damage should occur \\
Lower hole in the cover plate (slip) * & $1.06 \times 10^6$ \\
\hline
\end{tabular}
\end{center}

* These sections are under compression!

Fig. 5-27: Joint on middle support, negative moment

Fig. 5-28: Details of the joint
5.1.5.4.1 Scope

The scope of the tests concerns the analysis of the resistance of the joint on the middle support and the design of the welded connection.

For the welds there are two options for welding the profile on the cap plate. It is possible to make a fillet weld all around the profile and it is also possible to make a full penetration weld.

The fillet weld is the cheaper version but following EC 3, the expected fatigue life of the proper lower flange-to-cap-plate-weld is rather shorter as the fatigue life of the full penetration weld version.

Because of the cheaper option, it has been tested the fillet weld solution, to verify the resistance against fatigue.

5.1.5.4.2 Longitudinal steel reinforcement

Steel section needed in the reference bridge:
- on the upper side of the concrete slab 10 φ 16/m : 18 φ 16 = 36.18 cm²
- on the lower side of the concrete slab 10 φ 10/m: 18 φ 10 = 14.22 cm²

In total, there is \( A_s = 50.4 \text{ cm}^2 \) of steel reinforcement, that represents 1.4% of the concrete section.

Design tensile force in the rebars: \( F_{s,Rd} = 50.4 \times \frac{500}{1.15} = 2,192 \text{ kN} \)

Design compression in lower flange: \( F_{Rd} = b \ t_{f} \ f_{yd} = 30 \times 3 \times 460/1.1 = 3,762 \text{ kN} \) (\( > 2,192 \text{ kN} \))

The resistance moment results of the rebar resistance

In order to obtain the resistance moment available in the reference bridge where the width of the concrete slab reaches 3.40 m, the rebar section should be:
- on the upper side of the concrete slab : 18 φ 20 = 56.55 cm²
- on the lower side of the concrete slab : 18 φ 20 = 56.55 cm²

In total, there is \( A_s = 113.1 \text{ cm}^2 \) of steel reinforcement, that represents 3.1% of the concrete section.

5.1.5.4.3 Static calculation

The design resistance moment results of the rebar resistance.

\[
M_{Rd,i} = \left[ A_{s,i} \left( h_{HEA} + h_c - h_{s,i} - 0.5 \ t_f \right) \right] \frac{f_{sk}}{\gamma_s} \tag{5–29}
\]

\[
M_{Rd,i} = -\left[36.18 \ (1.075 - 0.06) + 14.22 \ (1.075 - 0.13)\right] \frac{500}{1.15} = -2,181 \text{ kN} \text{m} \tag{5–30}
\]

(for \( A_{s,i}/A_c = 1.4 \% \))
\[ M_{Rd,2} = \left[ 56.55 (1.075 - 0.06) + 56.55 (1.075 - 0.13) \right] \frac{500}{1.15} = -4.819 \text{ kNm} \]  
(5–31)

(for \( A_{s2}/A_c = 3.1\% \))

Considering the design moment in the reference bridge (\( M_{rd} = -3.123 \times 1.35 = -4.216 \text{ kNm} \); Fig. 5-9, 5.1.4.2.6.3), the third test could be compared with a real case if \( A_{s2}/A_c = 3.1\% \). The design action moment on the middle support is close to the resistance moment from this third test (\( M_{Rd} = -4.819 \text{ kNm} \)).

5.1.5.4.4 Connection between the concrete slab and the girder

5.1.5.4.4.1 Stud connectors [EC 4.2 - 6.3]

The design shear resistance of a headed stud, automatically welded, is calculated in 5.1.5.1.4.

The resistance of one stud connector 22 x 125 results to \( P_{Rd} = 88 \text{ kN} \)

The maximum load to report from the rebars to the profile section is equal to the maximum force in the rebars:

\[ F_{Rd} = A_s f_{sd} = 113.10 \times 500/1.15 = 4.920 \text{ kN} \]

\( F_{Rd} \) should be introduced in the connection by stud connectors: 2 lines of 33 stud connectors in each line are expected.

\[ Y V_{Rd} = 2 \times 33 \times 88 = 5.808 \text{ kN} > F_{Rd} = 4.920 \text{ kN} \]

5.1.5.4.4.2 Transverse steel reinforcement [EC 4.2 - 6.6]

Following EC 4.2 - 6.6, the transverse reinforcement in the slab has been designed for the ultimate limit state so that premature longitudinal shear failure and longitudinal splitting are (5.1.5.2.5.2).

In this test, steel section needed on the upper side of the concrete slab is \( \phi 10 \) with a space of 10 cm. On the lower side of the concrete slab, \( \phi 16 \) with a space of 10 cm is needed (Fig. 5-28, Fig. 5-32).

5.1.5.4.5 Design of the middle support

For the structural design of a concrete cross girder section, the general design, in order to limit the stress in the concrete, is a concrete cross girder with thick cap plates (Fig. 5-1)

5.1.5.4.5.1 Thick cap plate

In order to have a sufficient stress distribution in the concrete a thick cap plate with \( t \geq 60 \text{ mm} \) (here: 70 mm) is needed. The thickness of the cap plate has been determined following EC 2.1 - 6.7.
5.1.5.4.5.2 Maximum compression force in the bottom flange of the beam

In order to resist at the important forces in the concrete, a large steel reinforcement in the middle support is needed.

5.1.5.4.5.3 Weld of the profile

Two solutions are considered for welding the profile on the cap plate:

1. Full penetration weld: this is a more expensive solution, but the expected fatigue life should be longer

![Fig. 5-29: Full penetration welded solution](image)

2. Fillet weld all around the profile: this is a cheap solution, but following EC 3, the fatigue life is very short.

The thickness of the fillet weld in the flange results with EC3 to:

\[
t_f b f_{yd} \leq 2 \frac{\sqrt{2}}{\sqrt{3}} b f_{yd} a_w \Leftrightarrow a_w \geq 18 \text{mm}
\]

The thickness of the fillet weld in the web results with EC3 to:

\[
t_w d f_{yd} \leq 2 \frac{\sqrt{2}}{\sqrt{3}} d f_{yd} a_w \Leftrightarrow a_w \geq 10 \text{mm}
\]
But with the DIN18800 the thickness of the fillet welds results to:

$$a_w \geq \sqrt{\text{max } t - 0.5} \iff a_w \geq 8 \text{mm}$$

(5–34)

The 15 mm thick fillet weld results of the half thickness of the flange.

At the loadside there is the 8 mm and at the load averted side there is the 15 mm thick fillet weld.
5.1.5.4.5.4 Vertical force in the middle support

Following 5.1.5.4.6 the maximum vertical load during the test is equal to $P_{Sd} = P_{\text{max}} + P_G = 728 + 3.5 \times 11.52 = 770 \text{ kN}$. This load should be transmitted by stud connectors.

But in the reference bridge, the vertical load is larger (maximum vertical load $P_{Sd} = 1,500 \text{ kN}$). This results in more stud connectors.

The resistance of one stud connector $22 \times 125$ is $P_{Rd} = 88 \text{ kN}$ (5.1.5.1.4)

The maximum vertical load is: $F_{Rd} = 1,500 \text{ kN}$.

$F_{Rd}$ should be introduced in the concrete by 3 lines of 6 stud connectors.

$$
\gamma V_{Rd} = 3 \times 6 \times 88 = 1,584 \text{ kN} > F_{Rd} = 1,500 \text{ kN}
$$

5.1.5.4.6 Fatigue tests on the joints

5.1.5.4.6.1 Loads

The dead load of the specimen reaches 11.52 kN/m and the total dead load reaches 97.35 kN.

The bending moment under the dead load reaches $M_g = 11.52 \times \frac{3.5^2}{2} = 71 \text{ kNm}$.

In the reference bridge the difference between the maximum negative moment and the minimum negative moment is equal to $M_q = -3,123 + 608 = -2,515 \text{ kNm}$ (5.1.4.2.6.2 and 5.1.4.2.6.3) and the frequent moment range reaches $\Delta M_f = -1,155 \text{ kNm}$ (5.1.4.2.6.2).

The maximum moment should equal to the unfrequent moment in the reference bridge which results to $M_{uf} = -2,620 \text{ kNm}$ (5.1.4.2.6.2).

The load to apply on the specimen should not be higher than:

$$
P_{\text{max}} = \frac{2,620 - 71}{3.5} = 728 \text{ kN}
$$

This load should be increased during the first loading, if necessary, in order to obtain cracks in the concrete slab.

In order to reach a moment range equal to the frequent range moment, the load range should be:

$$
\Delta P = \frac{1,155}{3.5} = 330 \text{ kN}, \quad \text{and} \quad P_{\text{min}} = 728 - 330 \approx 400 \text{ kN}
$$

5.1.5.4.6.2 Expected fatigue life

5.1.5.4.6.2.1 Weld of lower flange

Considering full penetration welds in the lower flange, the fatigue limit is equal to $\Delta \sigma_D = 52 \text{ N/mm}^2$ (detail category $\Delta \sigma_c = 71 \text{ N/mm}^2$).
The moment of inertia of the composite section reaches to $I = 676,145 \text{ cm}^4$.
The neutral axis is situated in the web: 59 cm from the bottom of the girder.

Under a moment range of $\Delta M_f = -1,155 \text{ kNm}$, the stress range reaches at the weld in the lower flange of the steel section:

$$\Delta \sigma_e = \frac{1,155 \times 10^6 \times 590}{676,145 \times 10^4} = 101 \text{ N/mm}^2 > \Delta \sigma_D = 52 \text{ N/mm}^2$$

$\gamma$ the expected fatigue life should be: $N = 0.68 \times 10^6$ cycles

If the weld in the lower flange on the beam is a fillet weld, the fatigue limit is equal to $\Delta \sigma_D = 26.5 \text{ N/mm}^2$ (detail category: $\Delta \sigma_c = 36 \text{ N/mm}^2$) and the fatigue life should be very short:

$$\Delta \sigma_e = \frac{1,155 \times 10^6 \times 590}{676,145 \times 10^4} = 101 \text{ N/mm}^2 > \Delta \sigma_D = 26.5 \text{ N/mm}^2$$

$\gamma$ the expected fatigue life should be: $N = 90,300$ cycles

The upper flange is not relevant for the fatigue life, because of the less distance to the neutral axis.
The same obtains for the fillet welds in the concrete section (Fig. 5-33)
5.1.5.4.6.2.2 Fatigue of rebars

Following EC 2.1 - 6.8.4, for reinforcing steel, the fatigue resistance could be calculated with the following equation:

\[
\gamma_F \gamma_{sd} \Delta \sigma_{s,\text{equ}} (N^*) < \frac{\Delta \sigma_{Rsk} (N^*)}{\gamma_{s,fat}} \quad \text{[EC2.1 - 6.8.4]} \quad (5-35)
\]

- \( \gamma_F = 1.0 \)
- \( \gamma_{sd} = 1.0 \)
- \( \gamma_{s,fat} = 1.15 \)
- \( \Delta \sigma_{Rsk}(N^*) = 162.5 \text{ N/mm}^2 \) (stress range at 10\(^6\) cycles)
- \( \Delta \sigma_{s,\text{equ}}(N^*) \) is the stress range

In this case, under a range moment of \( \Delta M_f = -1,155 \text{ kNm} \), the stress range in the rebars reaches:

\[
\Delta \sigma_{s,\text{equ}} (N^*) = \frac{1,155 \times 10^6 \times 440}{676,145 \times 10^4} = 75 \text{ N/mm}^2 < \frac{\Delta \sigma_{Rsk} (N^*)}{\gamma_{s,fat}} = 141 \text{ N/mm}^2
\]

The expected fatigue life should be: \((141/75)^9 \times 10^6 = 2.9 \times 10^8\) cycles

5.1.5.4.7 Conclusion

<table>
<thead>
<tr>
<th>Position</th>
<th>Expected fatigue life</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Full penetration weld</td>
</tr>
<tr>
<td>Weld in the lower flange of the girder*</td>
<td>0.68 \times 10^6</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>2.9 \times 10^8</td>
</tr>
</tbody>
</table>

* These sections are under compression!

The fatigue crack is expected near the weld in the lower flange. If a full penetration weld is realised, the fatigue life amounts to \( N = 680,000 \) cycles and if it is a fillet weld, the fatigue life amounts to only \( N = 90,300 \) cycles.

In agreement with the other participants in the research, it has been decided that test \( N^0 3 \) should be performed first with fillet welds. Following the results of this test, it should be decided, if the weld of test \( N^0 4 \) has to be performed with:

- the same fillet welds (if the fatigue life is very longer than expected)
- full penetration welds (if the fatigue life is as short as expected)

Due to the fact, that test \( N^0 3 \) reaches 100 times more cycles than calculated (5.2.3) and much more than the calculated fatigue life of a full penetration weld, test \( N^0 4 \) has been performed also with fillet welds (15 mm thick at the lower flange of the averted load side and 8 mm thick at the load side) to validate the results of test \( N^0 3 \) (Fig. 5-31).

The norms to respect for the welds are:

- EN 287-1 : qualification of the welders
- EN 288-3 : instructions for operations
5.2 Evaluation of test results

5.2.1 Test 1

5.2.1.1 Summary of the test performed

All information about the dimensioning are given in 5.1.5.2.

The test sequence seems out as follows:

1) stat 11a 1st static test (a) to crack the concrete (0 cycles) with $P_{\text{max}}=1,044.28$ kN
2) stat 11b 1st static test (b) after the concrete is cracked (0 cycles) with $P_{\text{max}}=1,039.92$ kN
3) fat 11 1st fatigue test with $P_{\text{min}}=211$ kN and $P_{\text{max}}=1,040$ kN
   1,180,000 cycles; no slip has been occurred;
   crack at the lower side of the cover plate (Fig. 5-47)
4) stat 12 2nd static test with $P_{\text{max}}=1,602.08$ kN
5) Total amount of cycles: 1,180,000

5.2.1.2 Static test analysis

5.2.1.2.1 Arrangement of the strain gauges

Fig. 5-34: Strain gauges of Test N°1
Fig. 5-35: Strain gauges at the joint

Table 5-6: Distances of the strain gauges

<table>
<thead>
<tr>
<th>Detail</th>
<th>Distance [mm]</th>
<th>Detail</th>
<th>Distance [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load-introduction</td>
<td>0,000</td>
<td>Upper side of the concrete slab</td>
<td>0</td>
</tr>
<tr>
<td>Support</td>
<td>2,800</td>
<td>1\textsuperscript{st} row of strain gauges</td>
<td>100</td>
</tr>
<tr>
<td>Cross section A</td>
<td>3,400</td>
<td>2\textsuperscript{nd} row of strain gauges</td>
<td>230</td>
</tr>
<tr>
<td>Cross section B</td>
<td>3,750</td>
<td>3\textsuperscript{rd} row of strain gauges</td>
<td>345</td>
</tr>
<tr>
<td>Cross section C</td>
<td>3,875</td>
<td>4\textsuperscript{th} row of strain gauges</td>
<td>705</td>
</tr>
<tr>
<td>Cross section D</td>
<td>4,000</td>
<td>5\textsuperscript{th} row of strain gauges</td>
<td>945</td>
</tr>
<tr>
<td>Cross section E</td>
<td>4,125</td>
<td>6\textsuperscript{th} row of strain gauges</td>
<td>975</td>
</tr>
<tr>
<td>Cross section F</td>
<td>4,250</td>
<td>7\textsuperscript{th} row of strain gauges</td>
<td>1,005</td>
</tr>
<tr>
<td>Cross section G</td>
<td>4,600</td>
<td>8\textsuperscript{th} row of strain gauges</td>
<td>1,090</td>
</tr>
<tr>
<td>Support</td>
<td>5,200</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Load-introduction</td>
<td>8,000</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
5.2.1.2.2 Distribution of deformation 10 for the static tests

![Graph showing distribution of deformation 10](image)

**Fig. 5-36: Distribution of Deformation 10**

5.2.1.2.3 Results of the first static test stat 11b (Cross sections)

<table>
<thead>
<tr>
<th>Test</th>
<th>Stress [N/mm²]</th>
<th>Distance from the upper edge of the concrete [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>stat 11a</td>
<td>Test (A)</td>
<td></td>
</tr>
<tr>
<td>stat 11b</td>
<td>Test (G)</td>
<td></td>
</tr>
<tr>
<td>stat 12</td>
<td>Calculation (A &amp; G)</td>
<td></td>
</tr>
</tbody>
</table>

Cross section A and G
(first static test (b); P<sub>max</sub>=1039.92 kN)

![Graph showing stress in Cross Section A and G](image)

**Fig. 5-37: Progress of stress in Cross Section A and G (Remark 1)**
Fig. 5-38: Progress of stress in Cross Section B and F (Remark 1)

Cross section B and F
(first static test (b); $P_{\text{max}}=1039.92$ kN)

Fig. 5-39: Progress of stress in Cross Section D (Remark 1)

Cross section D
(first static test (b); $P_{\text{max}}=1039.92$ kN)
5.2.1.2.4 Results of the second static test stat 12 (Cross sections)

Cross section A and G
( second static test ; \( P_{\text{max}} = 1602.08 \) kN )

![Graph showing stress distribution for Cross section A and G](image)

Fig. 5-40: Progress of stress in Cross Section A and G (Remark 1)

Cross section B and F
( second static test ; \( P_{\text{max}} = 1602.08 \) kN )

![Graph showing stress distribution for Cross section B and F](image)

Fig. 5-41: Progress of stress in Cross Section B and F (Remark 1)
5.2.1.2.5 Longitudinal stress distribution of the first static test stat 11b

Fig. 5-42: Progress of stress in Cross Section D (Remark 1)

Fig. 5-43: Progress of stress of the lower edge of the lower/upper flange

Cross section D
(second static test; $P_{\text{max}} = 1602.08$ kN)

Stress distribution of the lower edge from the lower/upper flange
(first static test (b); $P_{\text{max}} = 1039.92$ kN)
Fig. 5-44: Progress of stress of the lower side of the cover plate (area of the lower row of bolts)

5.2.1.2.6 Longitudinal stress distribution of the second static test stat 12

Fig. 5-45: Progress of stress of the lower edge of the lower/upper flange
5.2.1.3 Fatigue test analysis

At this test no slip has been occurred and after 1,180,000 cycles cracks have been arised at the lower side of the cover plate (Fig. 5-47).

Fig. 5-46: Progress of stress of the lower side of the cover plate (area of the lower row of bolts)

Fig. 5-47: Failure of test 1
The S-N curve (\(i =\) test; line = EC3 S-N curve) of this test results to

![S-N curve](image)

*Fig. 5-48: S-N curve of test 1 (detail category 160 for the cover plate)*

Therefore it can be stated, that the fatigue resistance of this test is approximately as high as prescribed in EC 3.

<table>
<thead>
<tr>
<th>Detail category</th>
<th>Test 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>160</td>
<td>1.180</td>
</tr>
<tr>
<td>160</td>
<td>1.320</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Dp</th>
<th>Ds_e,wts*</th>
<th>Ds_e,ts*</th>
<th>N</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>[kN]</td>
<td>[N/mm²]</td>
<td>[N/mm²]</td>
<td>[cycles]</td>
<td></td>
</tr>
<tr>
<td>829</td>
<td>186</td>
<td>---</td>
<td>1.180 (10^6)</td>
<td></td>
</tr>
<tr>
<td>829</td>
<td>184</td>
<td>---</td>
<td>1.320 (10^6)</td>
<td></td>
</tr>
</tbody>
</table>

*\(\Delta P\) = load increment; \(\Delta \sigma_{e,wts}\) = without tension stiffening; \(\Delta \sigma_{e,ts}\) = with tension stiffening

Table 5-7: Summary of the S-N data of the cover plate

The tension stiffening effect has been calculated according to EC 4 [2].
5.2.2 Test 2

5.2.2.1 Summary of the test performed

All information about the dimensioning are given in 5.1.5.3.

The test sequence seems out as follows:

1) stat 21a  1st static test (a) to crack the concrete (0 cycles) with \( P_{\text{max}} = 681.50 \text{ kN} \)
2) stat 21b  1st static test (b) after the concrete is cracked (0 cycles) with \( P_{\text{max}} = 685.32 \text{ kN} \)
3) fat 21    1st fatigue test with \( P_{\min} = 213 \text{ kN} \) and \( P_{\max} = 683 \text{ kN} \)

4,120,000 cycles; no slip and no crack have been occurred

4) stat 22   2nd static test after 4,120,000 cycles with \( P_{\max} = 1,093.46 \text{ kN} \)
5) fat 22    2nd fatigue test with \( P_{\min} = 213 \text{ kN} \) and \( P_{\max} = 1,061 \text{ kN} \)

2,730,000 cycles; no slip has been occurred;

... crack at the lower and outer hole in the web of the girder (Fig. 5-76)

6) stat 23   3rd static test after 6,850,000 cycles with \( P_{\max} = 1,326.87 \text{ kN} \)
7) stat 24   4th static test after 6,850,000 cycles (with slip) with \( P_{\max} = 1,523.74 \text{ kN} \)
8) Total amount of cycles: 6,850,000

5.2.2.2 Static test analysis

5.2.2.2.1 Arrangement of the strain gauges

![Strain gauges of Test N°2](image)

Fig. 5-49: Strain gauges of Test N°2
Fig. 5-50: Strain gauges of the joint

Table 5-8: Distances of the strain gauges

<table>
<thead>
<tr>
<th>Detail</th>
<th>Distance [mm]</th>
<th>Detail</th>
<th>Distance [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load-introduction</td>
<td>0.000</td>
<td>Upper side of the concrete slab</td>
<td>0</td>
</tr>
<tr>
<td>Cross section A</td>
<td>1,800</td>
<td>1st row of strain gauges</td>
<td>230</td>
</tr>
<tr>
<td>Support</td>
<td>2,800</td>
<td>2nd row of strain gauges</td>
<td>345</td>
</tr>
<tr>
<td>Cross section B</td>
<td>3,400</td>
<td>3rd row of strain gauges</td>
<td>705</td>
</tr>
<tr>
<td>Cross section C</td>
<td>3,750</td>
<td>4th row of strain gauges</td>
<td>945</td>
</tr>
<tr>
<td>Cross section D</td>
<td>4,000</td>
<td>5th row of strain gauges</td>
<td>975</td>
</tr>
<tr>
<td>Cross section E</td>
<td>4,250</td>
<td>6th row of strain gauges</td>
<td>1,090</td>
</tr>
<tr>
<td>Cross section F</td>
<td>4,600</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Support</td>
<td>5,200</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cross section G</td>
<td>6,200</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Load-introduction</td>
<td>8,000</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
5.2.2.2.2 Distribution of deformation 10 for the static tests

![Graph showing distribution of deformation 10](image)

*Fig. 5-51: Distribution of Deformation 10 (stat 24: the strain gauge has been out of order)*

5.2.2.2.3 Results of the first static test stat 21b (Cross sections)

![Graph showing stress in Cross Section A and G](image)

*Fig. 5-52: Progress of stress in Cross Section A and G (Remark 1)*
Fig. 5-53: Progress of stress in Cross Section B and F (Remark 1)

Cross section B and F
(first static test (b); $P_{\text{max}} = 685.32$ kN)

Cross section C and E
(first static test (b); $P_{\text{max}} = 685.32$ kN)

Fig. 5-54: Progress of stress in Cross Section C and E (Remark 1)
5.2.2.2.4 Results of the second static test stat 22 (Cross sections)

Cross section D
(first static test (b); $P_{\text{max}} = 685.32 \text{ kN}$)

Fig. 5-55: Progress of stress in Cross Section D (Remark 1)

Cross section A and G
(second static test; $P_{\text{max}} = 1093.46 \text{ kN}$)

Fig. 5-56: Progress of stress in Cross Section A and G (Remark 1)
Fig. 5-57: Progress of stress in Cross Section B and F (Remark 1)

Cross section B and F
(second static test ; $P_{\text{max}} = 1093.46 \text{ kN}$)

Fig. 5-58: Progress of stress in Cross Section C and E (Remark 1)

Cross section C and E
(second static test ; $P_{\text{max}} = 1093.46 \text{ kN}$)
5.2.2.2.5 Results of the third static test stat 23 (Cross sections)

Cross section D
( second static test ; \( P_{\text{max}} = 1093.46 \text{ kN} \) )

Cross section A and G
( third static test ; \( P_{\text{max}} = 1326.87 \text{ kN} \) )

Fig. 5-59: Progress of stress in Cross Section D (Remark 1)

Fig. 5-60: Progress of stress in Cross Section A and G (Remark 1)
Cross section B and F
( third static test; $P_{\text{max}} = 1326.87$ kN )

Fig. 5-61: Progress of stress in Cross Section B and F (Remark 1)

Cross section C and E
( third static test; $P_{\text{max}} = 1326.87$ kN )

Fig. 5-62: Progress of stress in Cross Section C and E (Remark 1)
5.2.2.2.6 Results of the fourth static test stat 24 (Cross sections)

**Cross section D**
( third static test ; \(P_{\text{max}} = 1326.87 \text{ kN} \) )

![Graph showing stress distribution in Cross Section D](image)

**Cross section A and G**
( fourth static test ; \(P_{\text{max}} = 1523.74 \text{ kN} \) )

![Graph showing stress distribution in Cross Sections A and G](image)

Fig. 5-63: Progress of stress in Cross Section D (Remark 1)

Fig. 5-64: Progress of stress in Cross Section A and G (Remark 1)
Cross section B and F
( fourth static test ; $P_{\text{max}} = 1523.74 \, \text{kN} $)

![Graph of Cross section B and F](image)

**Fig. 5-65: Progress of stress in Cross Section B and F** *(Remark 1)*

Cross section C and E
( fourth static test ; $P_{\text{max}} = 1523.74 \, \text{kN} $)

![Graph of Cross section C and E](image)

**Fig. 5-66: Progress of stress in Cross Section C and E** *(Remark 1)*
5.2.2.2.7 Longitudinal stress distribution of the first static test stat 21b

Fig. 5-67: Progress of stress in Cross Section D (Remark 1)

Fig. 5-68: Progress of stress of the lower edge of the lower/upper flange
5.2.2.2.8 Longitudinal stress distribution of the second static test stat 22

**Fig. 5-69: Progress of stress of the lower row of bolts**

**Fig. 5-70: Progress of stress of the lower edge of the lower/upper flange**
5.2.2.2.9 Longitudinal stress distribution of the third static test stat 23

**Fig. 5-71: Progress of stress of the lower row of bolts**

**Fig. 5-72: Progress of stress of the lower edge of the lower/upper flange**

<table>
<thead>
<tr>
<th>Test (lower edge of the lower flange)</th>
<th>Calculation (lower edge of the lower flange)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BEAM (web)</td>
<td>Cover plate</td>
</tr>
<tr>
<td>Strain gauge 104/604</td>
<td>Strain gauge 203/503</td>
</tr>
<tr>
<td>Strain gauge 202/502</td>
<td>Strain gauge 304/504</td>
</tr>
<tr>
<td>Strain gauge 204/504</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test (lower edge of the upper flange)</th>
<th>Calculation (lower edge of the upper flange)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strain gauge 104/604</td>
<td>Strain gauge 203/503</td>
</tr>
<tr>
<td>Strain gauge 202/502</td>
<td>Strain gauge 304/504</td>
</tr>
<tr>
<td>Strain gauge 204/504</td>
<td></td>
</tr>
</tbody>
</table>
5.2.2.2.10  Longitudinal stress distribution of the fourth static test stat 24

Fig. 5-74: Progress of stress of the lower edge of the lower/upper flange

Fig. 5-73: Progress of stress of the lower row of bolts
5.2.2.3 Fatigue test analysis

At this test no slip has been occurred and after 6,850,000 cycles cracks have been arisen at the lower and outer hole in the web of the girder (Fig. 5-76).

lower and outer hole
in the web of the girder      crack

---

![Graph showing stress progress of the lower row of bolts](image)

**Fig. 5-75: Progress of stress of the lower row of bolts**

**Fig. 5-76: Failure of test 2**
The S-N curve (\(i = \text{test}; \text{line} = \text{EC3 S-N curve}\)) of this test results to

![S-N curve of test 2 (detail category 112 for a hole in the web of the girder)](image)

**Fig. 5-77: S-N curve of test 2 (detail category 112 for a hole in the web of the girder)**

Therefore it can be stated, that the fatigue resistance of this test is higher as prescribed in EC 3.

**Table 5-9: Summary of the S-N data of the hole in the web**

<table>
<thead>
<tr>
<th></th>
<th>(\Delta P) [kN]</th>
<th>(\Delta \sigma_{e,wts}^*) [N/mm(^2)]</th>
<th>(\Delta \sigma_{e,ts}^*) [N/mm(^2)]</th>
<th>(N) [cycles]</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Test</strong></td>
<td>470 / 848</td>
<td>---</td>
<td>124</td>
<td>(6.850 \times 10^6)</td>
</tr>
<tr>
<td><strong>Calculation</strong></td>
<td>470 / 848</td>
<td>224</td>
<td>---</td>
<td>(0.250 \times 10^6)</td>
</tr>
<tr>
<td></td>
<td>470 / 848</td>
<td>---</td>
<td>222</td>
<td>(0.257 \times 10^6)</td>
</tr>
</tbody>
</table>

*\(e,wts\): equivalent, without tension stiffening  *\(e,ts\): equivalent, with tension stiffening

The tension stiffening effect has been calculated according to EC 4 [2].
5.2.3 Test 3

5.2.3.1 Summary of the test performed

All information about the dimensioning are given in 5.1.5.4.

The test sequence seems out as follows:

1) stat 31a 1st static test (a) to crack the concrete (0 cycles) with $P_{\text{max}}=732.01$ kN
2) stat 31b 1st static test (b) after the concrete is cracked (0 cycles) with $P_{\text{max}}=730.16$ kN
3) fat 31 1st fatigue test with $P_{\text{min}}=400$ kN and $P_{\text{max}}=730$ kN
   97,000 cycles; no crack has been occurred
4) stat 32 2nd static test after 97,000 cycles with $P_{\text{max}}=733.93$ kN
5) fat 32 2nd fatigue test with $P_{\text{min}}=400$ kN and $P_{\text{max}}=730$ kN
   7,613,000 cycles; no crack has been occurred
6) stat 33 3rd static test after 7,710,000 cycles with $P_{\text{max}}=746.13$ kN
8) fat 33 4th fatigue test with $P_{\text{min}}=200$ kN and $P_{\text{max}}=730$ kN
   1,470,000 cycles; 1,380,000 cycles: crack in the weld of the lower flange (Fig. 5-101)
9) stat 34 4th static test after 9,180,000 cycles with $P_{\text{max}}=921.70$ kN
   (2.2 mm crack in the concrete over support, but no crack growth in the weld)
10) fat 34 5th fatigue test with $P_{\text{min}}=200$ kN and $P_{\text{max}}=730$ kN
   730,000 cycles; crack growth in the concrete
11) stat 35 5th static test after 9,910,000 cycles with $P_{\text{max}}=665.71$ kN
   loss of stability of the beam: too high deformation; $P_{\text{max}}$ could not loaded
   therefore it could be stated, that the reinforcement failed
12) Total amount of cycles: 9,910,000

5.2.3.2 Static test analysis

5.2.3.2.1 Arrangement of the strain gauges

![Cross section diagram]

*Fig. 5-78: Strain gauges of Test N°3 (thicknesses of the fillet welds according to Fig. 5-30)*
Fig. 5-79: Strain gauges of the joint

Table 5-10: Distances of the strain gauges

<table>
<thead>
<tr>
<th>Detail</th>
<th>Distance [mm]</th>
<th>Detail</th>
<th>Distance [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load-introduction</td>
<td>0.000</td>
<td>Upper side of the concrete slab</td>
<td>0</td>
</tr>
<tr>
<td>Cross section A</td>
<td>2.730</td>
<td>1&lt;sup&gt;st&lt;/sup&gt; row of strain gauges</td>
<td>230</td>
</tr>
<tr>
<td>Cross section B</td>
<td>3.230</td>
<td>2&lt;sup&gt;nd&lt;/sup&gt; row of strain gauges</td>
<td>630</td>
</tr>
<tr>
<td>Root of the weld</td>
<td>3.256</td>
<td>3&lt;sup&gt;rd&lt;/sup&gt; row of strain gauges</td>
<td>830</td>
</tr>
<tr>
<td>Flange of the cover plate</td>
<td>3.280</td>
<td>4&lt;sup&gt;th&lt;/sup&gt; row of strain gauges</td>
<td>1,060</td>
</tr>
<tr>
<td>Support</td>
<td>3.500</td>
<td>5&lt;sup&gt;th&lt;/sup&gt; row of strain gauges</td>
<td>1,090</td>
</tr>
<tr>
<td>Flange of the cover plate</td>
<td>3.720</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Root of the weld</td>
<td>3.744</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cross section C</td>
<td>3.770</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cross section D</td>
<td>4.270</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Support</td>
<td>7.000</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
5.2.3.2.2 Distribution of deformation 2 for the static tests

Fig. 5-80: Distribution of Deformation 2

5.2.3.2.3 Results of the first static test b stat 31b (Cross sections)

Fig. 5-81: Progress of stress in Cross Section A and D (Remark 1)
The difference in the stress of the lower edge of the lower flange and the upper edge of the lower flange results of the hot-spot-influence, because the strain gauges in cross section B and C are to close to the fillet weld.
5.2.3.2.4 Results of the second static test stat 32 (Cross sections)

Fig. 5-83: Progress of stress in Cross Section A and D (Remark 1)

Fig. 5-84: Progress of stress in Cross Section B and C (Remark 1)
The difference in the stress of the lower edge of the lower flange and the upper edge of the lower flange results of the hot-spot-influence, because the strain gauges in cross section B and C are too close to the fillet weld.

5.2.3.2.5 Results of the third static test stat 33 (Cross sections)

![Graph of Cross section A and D](image)

**Fig. 5-85: Progress of stress in Cross Section A and D (Remark 1)**
The difference in the stress of the lower edge of the lower flange and the upper edge of the lower flange results of the hot-spot-influence, because the strain gauges in cross section B and C are to close to the fillet weld.

**Fig. 5-86: Progress of stress in Cross Section B and C (Remark 1)**

The difference in the stress of the lower edge of the lower flange and the upper edge of the lower flange results of the hot-spot-influence, because the strain gauges in cross section B and C are to close to the fillet weld.
5.2.3.2.6 Results of the fourth static test stat 34 (Cross sections)

Cross section A and D
( fourth static test ; $P_{\text{max}} = 921.6952$ kN )

![Graph showing stress distribution in Cross Section A and D](image)

**Fig. 5-87: Progress of stress in Cross Section A and D (Remark 1)**

Cross section B and C
( fourth static test ; $P_{\text{max}} = 921.6952$ kN )

![Graph showing stress distribution in Cross Section B and C](image)

**Fig. 5-88: Progress of stress in Cross Section B and C (Remark 1)**
The difference in the stress of the lower edge of the lower flange and the upper edge of the lower flange results of the hot-spot-influence, because the strain gauges in cross section B and C are to close to the fillet weld.

5.2.3.2.7 Results of the fifth static test stat 35 (Cross sections)

![Graph showing stress distribution in Cross Section A and D](Image)

**Fig. 5-89: Progress of stress in Cross Section A and D (Remark 1)**
The difference in the stress of the lower edge of the lower flange and the upper edge of the lower flange results of the hot-spot-influence, because the strain gauges in cross section B and C are to close to the fillet weld.

Because of the failure of the concrete and the reinforcement the neutral axis moves to the lower flange.

Fig. 5-90: Progress of stress in Cross Section B and C (Remark 1)
5.2.3.2.8 Longitudinal stress distribution of the first static test stat 31b

Fig. 5-91: Progress of stress of the lower edge of the lower/upper flange

Fig. 5-92: Hot-Spot-Stress in close proximity to the fillet weld
The Hot-Spot-Stresses has been measured with 5 extra strain gauges on the centre of the lower edge of the lower flange with a distance of 5 mm to each other. The first strain gauge has been applied in a distance of 5 mm to the root of the fillet weld.

5.2.3.2.9 Longitudinal stress distribution of the second static test stat 32

![Stress distribution of the lower edge from the lower/upper flange](image)

**Fig. 5-93: Progress of stress of the lower edge of the lower/upper flange**
The Hot-Spot-Stresses has been measured with 5 extra strain gauges on the centre of the lower edge of the lower flange with a distance of 5 mm to each other. The first strain gauge has been applied in a distance of 5 mm to the root of the fillet weld.
5.2.3.2.10  Longitudinal stress distribution of the third static test stat 33

Fig. 5-95: Progress of stress of the lower edge of the lower/upper flange

Fig. 5-96: Hot-Spot-Stress in close proximity to the fillet weld
The Hot-Spot-Stresses has been measured with 5 extra strain gauges on the centre of the lower edge of the lower flange with a distance of 5 mm to each other. The first strain gauge has been applied in a distance of 5 mm to the root of the fillet weld.

5.2.3.2.11 Longitudinal stress distribution of the fourth static test stat 34

![Stress distribution of the lower edge from the lower/upper flange](image)

**Fig. 5-97: Progress of stress of the lower edge of the lower/upper flange**
Fig. 5-98: Hot-Spot-Stress in close proximity to the fillet weld

The Hot-Spot-Stresses has been measured with 5 extra strain gauges on the centre of the lower edge of the lower flange with a distance of 5 mm to each other. The first strain gauge has been applied in a distance of 5 mm to the root of the fillet weld.
5.2.3.2.12 Longitudinal stress distribution of the fifth static test stat 35

Fig. 5-99: Progress of stress of the lower edge of the lower/upper flange

**Course of stresses in close proximity to the fillet weld**

Fig. 5-100: Hot-Spot-Stress in close proximity to the fillet weld
The Hot-Spot-Stresses has been measured with 5 extra strain gauges on the centre of the lower edge of the lower flange with a distance of 5 mm to each other. The first strain gauge has been applied in a distance of 5 mm to the root of the fillet weld.

5.2.3.3 Fatigue test analysis

Because of the residual stress (no crack growth has been occurred) the fillet weld of the lower flange cracked after 9,090,000 cycles (Fig. 5-101). After 9,910,000 cycles there has been occurred loss of stability of the beam. Therefore it could be stated, that the reinforcement failed.

![Fig. 5-101: Failure of test 3](image-url)
The S-N curve (test line = EC3 S-N curve) of this test results to

![Fig. 5-102: S-N curve of test 3 (detail category 141 for the reinforcement)]

![Fig. 5-103: S-N curve of test 3 (detail category 36 for the fillet weld)]
Therefore it can be stated, that the fatigue resistance of the reinforcement of this test is approximately as high as prescribed in EC 2.1-6.8.4. The fatigue resistance of the fillet welds at the lower flange of this test is much higher as prescribed in EC 2.1-6.8.4.

**Table 5-11: Summary of the S-N data of the reinforcement**

<table>
<thead>
<tr>
<th></th>
<th>$\Delta P$ [kN]</th>
<th>$\Delta \sigma_{e,wts}^*$ [N/mm$^2$]</th>
<th>$\Delta \sigma_{e,ts}^*$ [N/mm$^2$]</th>
<th>$N$ [cycles]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test</td>
<td>330 / 530</td>
<td>(102)</td>
<td>115</td>
<td>$9.910 \times 10^6$</td>
</tr>
<tr>
<td></td>
<td>330 / 530</td>
<td>103</td>
<td>---</td>
<td>$16.880 \times 10^6$</td>
</tr>
<tr>
<td></td>
<td>330 / 530</td>
<td>---</td>
<td>116</td>
<td>$5.790 \times 10^6$</td>
</tr>
</tbody>
</table>

*e,wts: equivalent, without tension stiffening  *e,ts: equivalent, with tension stiffening

**Table 5-12: Summary of the S-N data of the fillet weld**

<table>
<thead>
<tr>
<th></th>
<th>$\Delta P$ [kN]</th>
<th>$\Delta \sigma_{e,wts}^*$ [N/mm$^2$]</th>
<th>$\Delta \sigma_{e,ts}^*$ [N/mm$^2$]</th>
<th>$N$ [cycles]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test</td>
<td>330 / 530</td>
<td>---</td>
<td>101/103</td>
<td>$9.090/9.910 \times 10^6$</td>
</tr>
<tr>
<td></td>
<td>330 / 530</td>
<td>120</td>
<td>---</td>
<td>$0.054 \times 10^6$</td>
</tr>
<tr>
<td></td>
<td>330 / 530</td>
<td>---</td>
<td>115</td>
<td>$0.061 \times 10^6$</td>
</tr>
</tbody>
</table>

*e,wts: equivalent, without tension stiffening  *e,ts: equivalent, with tension stiffening

The tension stiffening effect has been calculated according to EC 4 [2].
5.2.4 Test 4

5.2.4.1 Summary of the test performed

All information about the dimensioning are given in 5.1.5.4.

The test sequence seems out as follows:

1) stat 41a 1st static test (a) to crack the concrete (0 cycles) with $P_{max}=730.39$ kN
2) stat 41b 1st static test (b) after the concrete is cracked (0 cycles) with $P_{max}=733.068$ kN
3) fat 41 1st fatigue test with $P_{min}=400$ kN and $P_{max}=730$ kN
   2,300,000; no cracks have been occurred
4) stat 42 2nd static test after 2,300,000 cycles with $P_{max}=786.0152$ kN
5) fat 42 2nd fatigue test with $P_{min}=200$ kN and $P_{max}=730$ kN
   1,600,000 cycles: cracks at the 8 mm thick fillet weld in the lower edge of the lower flange
   1,655,000 cycles: the reinforcement failed and no crack growth in the flange has been occurred
6) Total amount of cycles: 3,955,000

5.2.4.2 Static test analysis

5.2.4.2.1 Arrangement of the strain gauges

![Cross section diagram](image)

Fig. 5-104: Strain gauges of Test N°4 (thicknesses of the fillet welds in Fig. 5-31)
Table 5-13: Distances of the strain gauges

<table>
<thead>
<tr>
<th>Detail</th>
<th>Distance [mm]</th>
<th>Detail</th>
<th>Distance [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load-introduction</td>
<td>0.000</td>
<td>Upper side of the concrete slab</td>
<td>0</td>
</tr>
<tr>
<td>Cross section A</td>
<td>2,730</td>
<td>1st row of strain gauges</td>
<td>230</td>
</tr>
<tr>
<td>Cross section B</td>
<td>3,200</td>
<td>2nd row of strain gauges</td>
<td>1,090</td>
</tr>
<tr>
<td>Root of the weld (a=8mm)</td>
<td>3,269</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flange of the cover plate</td>
<td>3,280</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Support</td>
<td>3,500</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flange of the cover plate</td>
<td>3,720</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Root of the weld (a=15mm)</td>
<td>3,741</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cross section C</td>
<td>3,800</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cross section D</td>
<td>4,270</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Support</td>
<td>7,000</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Fig. 5-105: Strain gauges of the joint**
5.2.4.2.2 Distribution of deformation 2 for the static tests

Fig. 5-106: Distribution of Deformation 2

5.2.4.2.3 Results of the first static test stat 41b (Cross sections)

Fig. 5-107: Progress of stress in Cross Section A and D (Remark 1)
5.2.4.2.4 Results of the second static test stat 42 (Cross sections)

Cross section B and C
(first static test (b) ; \( P_{\text{max}} = 735.068 \text{kN} \))

Fig. 5-108: Progress of stress in Cross Section B and C (Remark 1)

Cross section A and D
(second static test ; \( P_{\text{max}} = 786.0152 \text{kN} \))

Fig. 5-109: Progress of stress in Cross Section A and D (Remark 1)
Fig. 5-110: Progress of stress in Cross Section B and C (Remark 1)

5.2.4.2.5 Longitudinal stress distribution of the first static test stat 41b

Fig. 5-111: Progress of stress of the lower edge of the lower/upper flange
The Hot-Spot-Stresses has been measured with 5 extra strain gauges on the centre of the lower edge of the lower flange with a distance of 5 mm to each other. The first strain gauge has been applied in a distance of 5 mm to the root of the fillet weld.

*Fig. 5-112: Hot-Spot-Stress in close proximity to the fillet weld*
5.2.4.2.6 Longitudinal stress distribution of the second static test stat 42

Fig. 5-113: Progress of stress of the lower edge of the lower/upper flange

Course of stresses in close proximity to the fillet weld

Fig. 5-114: Hot-Spot-Stress in close proximity to the fillet weld
The Hot-Spot-Stresses has been measured with 5 extra strain gauges on the centre of the lower edge of the lower flange with a distance of 5 mm to each other. The first strain gauge has been applied in a distance of 5 mm to the root of the fillet weld.

5.2.4.3 Fatigue test analysis

Because of the residual stress (no crack growth has been occurred) the lower edge of the lower flange cracked in front of the root of the 8 mm thick fillet weld after 3,955,000 cycles.

The S-N curve ($\text{\textit{i} = test; line = EC3 S-N curve}$) of this test results to

![S-N curve of test 4 (detail category 141 for the reinforcement)](image)

*Fig. 5-115: S-N curve of test 4 (detail category 141 for the reinforcement)*
Therefore it can be stated, that the fatigue resistance of the reinforcement of this test is approximately as high as prescribed in EC 2.1-6.8.4. The fatigue resistance of the fillet welds at the lower flange of this test is much higher as prescribed in EC 2.1-6.8.4.

Table 5-14: Summary of the S-N data of the reinforcement

<table>
<thead>
<tr>
<th></th>
<th>$\Delta P$ [kN]</th>
<th>$\Delta \sigma_{e,wts}$ * [N/mm$^2$]</th>
<th>$\Delta \sigma_{e,ts}$ * [N/mm$^2$]</th>
<th>N [cycles]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test</td>
<td>330 / 530</td>
<td>(101)</td>
<td>114</td>
<td>3.955 $10^6$</td>
</tr>
<tr>
<td>Calculation</td>
<td>330 / 530</td>
<td>110</td>
<td>---</td>
<td>9.340 $10^6$</td>
</tr>
<tr>
<td></td>
<td>330 / 530</td>
<td>---</td>
<td>123</td>
<td>3.420 $10^6$</td>
</tr>
</tbody>
</table>

*e,wts: equivalent, without tension stiffening  * e,ts: equivalent, with tension stiffening

Table 5-15: Summary of the S-N data of the fillet weld

<table>
<thead>
<tr>
<th></th>
<th>$\Delta P$ [kN]</th>
<th>$\Delta \sigma_{e,wts}$ * [N/mm$^2$]</th>
<th>$\Delta \sigma_{e,ts}$ * [N/mm$^2$]</th>
<th>N [cycles]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test</td>
<td>330 / 530</td>
<td>---</td>
<td>118 / 119</td>
<td>3.460 / 3.955 $10^6$</td>
</tr>
<tr>
<td>Calculation</td>
<td>330 / 530</td>
<td>110</td>
<td>---</td>
<td>0.039 $10^6$</td>
</tr>
<tr>
<td></td>
<td>330 / 530</td>
<td>---</td>
<td>80</td>
<td>0.044 $10^6$</td>
</tr>
</tbody>
</table>

*e,wts: equivalent, without tension stiffening  * e,ts: equivalent, with tension stiffening

The tension stiffening effect has been calculated according to EC 4 [2].
**Remark 1:**

The stresses in the reinforcement are not the exactly existing stresses, because strain gauges were not attached on the rebars. Only the elongation in relation to a baseline of 300 mm has been measured. So the stress could be higher/lower in the reinforcement. Also it could be, that the elongation has not been measured. If this was the case, the stresses have been extrapolated in relation to the stresses on the top edge of the upper flange or have been equated to zero. The stresses are standard values!

---

**5.3 Conclusions**

For the connection in span with a bolted connection, for both tested cases, that means one connection with a negative action moment and one connection with a positive action moment, the tests have reached the same (Test 1, pos. Moment) respectively 27 times (Test 2, neg. moment) more cycles as calculated according to EC 3. Furthermore in both cases there has been no slip detected up to these number of cycles.

Due to the fact, that the lifetime of a bolted connection in span in the area of the negative moment is also round about 6 times higher than the lifetime of a bolted connection in span in the area of the positive moment, the connection should be performed nearer to an area where only negative moments occurred.

But it is important to keep the static resistance of the connection section in this area.

For the connection on support with a concrete cross girder, it could be concluded, that for a connection always in a negative moment area a fillet weld is the better solution to weld the profile on the cap plate. Advantages are the simple installation of the bending-, shear- and splitting-tensile reinforcement and the less reinforcement. Additionally a fillet weld solution is cheaper than a full penetration weld solution.

Furthermore it has been proved, that the fillet weld solution reaches a fatigue life which has been 183 times higher (Test 3) and 101 times higher (Test 4) than calculated according to EC concerning to the fillet welds. Also the fatigue life is much higher as for the full penetration weld has been calculated.

But it has to point out, that in Test 3 and 4 the reinforcement failed and in both tests the reinforcement did not reached the calculated lifetime.

Totalling it can be stated, that the connection on support with a concrete cross girder and a fillet weld to weld the profile on the cap plate, respectively to EC3, is the best solution, because this test reached the most cycles. Furthermore this should be the most effective solution with regard to lifetime and costs.

To calculate the stress range for the fatigue analysis a linear strain distribution over the whole section could be assumed. This means, that a continuous steel girder without a joint could be supposed.
5.4 References

[1] Eurocode 4: Design of composite steel and concrete structures
   Part 1-1: General rules and rules for buildings
   Part 2: Bridges
   Part 1: General rules and rules for buildings
   Part 3: Traffic loads on bridges
   Spans”; Annual Report – 1999
   Spans”; Annual Report - 2000
6  PLATE BUCKLING OF STOCKY AND SLENDER WEBS

6.1 Experimental Investigations

6.1.1 Scope

6.1.1.1 Aim of the tests

The aim was also to find out what influence the concrete slab will have on the resistance to vertical shear for girders with stocky as well as slender webs. Another objective was to investigate the shear resistance of beams of steel grade S 460.

The limitations for this work is: Only twelve beams will be tested in this study, four all steel beams and eight composite beams.
- 6 rolled girders of HEA 360 of grade S 460.
- 6 welded I-shaped girders with the same slenderness of the web for all girders, \( \lambda_w \approx 1.45 \)
- For the girders with slender webs the steel grade was S 420 for the web plate and S 500 for the flanges.
- The shear connectors were headed studs \( \phi 19 – 75 \) Nelson S137 3K.

6.1.1.2 Description of the test procedure

For the girders with stocky webs, test specimens TS 1 to TS 3, the load was applied at the end of the beams on the end plate, by a T-shaped loading bar. Two actuators, each with a capacity of 1000 kN were used in this test set up. The deformation was applied by constant rate of 0.02 mm/s.

![Fig. 6-1: Test set-up for specimen TS 1, TS 2 and TS 3. (TS 3 without concrete).]

Before each test the out of plane deformation of the web was measured. It was measured again when the maximum load was reached and the web began to buckle. LVDT gauges measured the deflection of the girders. The strains in the web were measured by rosette strain gauges, two on each side of the web, see Fig. 6-6 to Fig. 6-7.

For the girders with slender webs, test specimen TS 5 to TS 8, the supports in the middle of the span, at the actuators and at the support on the supported side of the beam were not fixed to the girder so they were free to move relatively to each other. The load was applied in the middle of the beams for TS 5 and TS 7 by a T-shaped loading bar. For the tests TS 6 and TS 8 the load was applied 1 meter from each end of the beams. Two actuators, each with a capacity of 1000 kN was
used in this test setup. The deformation was applied by constant rate of 0.02 mm/s. See chapter 6.1.4.

Before each test the out of plane deformation of the web was measured. It was measured during the tests at a several times.

LVDT gauges measured the deflection of the flange. The strains in the web were measured by rosette strain gauges, two on each side of the web. The supports at the end of the beams and at the actuators were not fixed to the girder so they were free to move relatively to each other. The girders were supported on roller bearings.

The location of the displacement gauges and the position of the strain gauges are shown in chapter 6.1.5.

### 6.1.2 Calculations

#### 6.1.2.1 Resistance to vertical shear and bending moment

The resistances to vertical shear were calculated according to EC 3 part 1-1 and EC 3 part 1-5 and EC 4 part 1-1. For the girders with stocky webs the resistances to bending moment are calculated for a plastic cross-section for specimen TS 1 to TS 3. The girders with slender webs, TS 5 to TS 8, were class four cross-sections and therefore it is necessary to use an effective cross-section according to EC 3 part 1-1. In Table 6-1 the calculated resistance is shown.

Table 6-1: Calculated resistance to vertical shear and bending moment.

<table>
<thead>
<tr>
<th>Test specimen</th>
<th>EC 3-1-1</th>
<th>EC 3-1-5, $\eta=1,2$</th>
<th>EC 4-1-1</th>
<th>$M_R$</th>
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<td>TS 1:2</td>
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<td>TS 3:2</td>
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<tr>
<td>Test specimens with slender webs</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
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<td>EC 3-1-1</td>
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<td>TS 6:2 B</td>
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<td>TS 8:1 B</td>
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<td>688</td>
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<td>1738</td>
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</tbody>
</table>

EC 4-1-1 is not formally official yet. The parameter $\eta$ is a factor that takes the steel grade into account when calculating the resistance to vertical shear according to EC 3-1-5. For steel grades up to S 355 $\eta=1,2$, but for steel grades S 420 and S 460 $\eta=1,05$.

The resistance to vertical shear according to EC 3 part 1-5 for the composite girders TS 7 and TS 8, is calculated in such a way that the largest steel flange is taken into account, according to EC 4 part
2. Bridges (4.4.3(3)). Index A and B referees to each web plate. After the first web plate had reach maximum load the other web plate were subjected to an increasing load until it reach maximum load.
6.1.2.2 Materials

The steel grade was S 460 for all rolled beams, which had an actual proof stress of 461 MPa for the web, 502 MPa and 496 MPa for the lower and the upper flange respectively. For the welded steel girders with slender webs the steel grade was S 420 for the web plate and S 500 for the flange plates. The actual proof stress of the steel plates was for the web 435 MPa, for the non-composite flange 531 MPa and for the composite steel flange 625 MPa. The reinforcement had an actual proof stress of 438 Mpa. Nominal yield strength was 400 MPa. The concrete had strength of 52 MPa in compression.

The shear connectors were headed studs φ 19 – 75 Nelson S137 3K. Their strength was not tested.

6.1.3 Details of the test specimens

The geometrical properties of the tested sections are presented in this chapter. The notations are shown in Fig. 6-2 to Fig. 6-3 and the geometrical properties are shown in Table 6-2 and Table 6-3.
Table 6-2: Geometrical properties for girders with stocky webs, TS 1-TS 3.

<table>
<thead>
<tr>
<th></th>
<th>TS 1:1</th>
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<th>TS 2:1</th>
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Table 6-3: Geometrical properties for girders with slender webs, TS 5-TS 8.

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<td>3</td>
<td>3</td>
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</tr>
</tbody>
</table>
6.1.4 **Applied loads**

![Fig. 6-4: Applied loads for test specimens without concrete.](image1)

Fig. 6-4: Applied loads for test specimens without concrete.

![Fig. 6-5: Applied loads for test specimen including concrete slab.](image2)

Fig. 6-5: Applied loads for test specimen including concrete slab.

6.1.5 **Measurements**

The locations of the LVDT-gauges are shown in Fig. 6-6 and Fig. 6-7. The positions of the strain gauges are marked by x in Fig. 6-6 and Fig. 6-7. In Fig. 6-8 to Fig. 6-9 the grid on the web is shown. The grid is used to measure the out of plane deformation of the web during the test. For the composite girders it was necessary to find out how the concrete slab will move relative to the steel girder.

The movement parallel and perpendicular to the steel girder was measured by LVDT gauges. The locations of the gauges are shown in Fig. 6-10 and Fig. 6-11. The vertical gauges are placed 50 mm from the end of the concrete slab. The horizontal gauges are placed 50 mm from the edge of the concrete slab.
Fig. 6-6: Location of the measuring instrument on the all steel girders
Fig. 6-7: Location of the measuring instrument on the composite girders.

Fig. 6-8: Grid on web for measuring web buckles. The size of the grid is 100 x 100 mm.

Fig. 6-9: Grid on web for measuring web buckles. The size of the grid is 100 x 100 mm.
Fig. 6-10: Instrumentation for measuring the movement of the concrete relative to the steel girder.

Fig. 6-11: Instrumentation for measuring the movement of the concrete relative to the steel girder for test specimen.
6.1.6 Results

6.1.6.1 Girders with stocky webs

The six beams tested showed very small variations in the behaviour during the test and the maximum load was almost the same for all the beams. The failure mode was web buckling in the strain hardening range in all tests. The maximum shear and the total deflection in the middle of the beam are shown in Table 6-4.

Table 6-4: Test results. Maximum shear and total deflection at maximum load.

<table>
<thead>
<tr>
<th>Test specimen</th>
<th>Maximum shear (kN)</th>
<th>Moment (kNm)</th>
<th>Total deflection at maximum load (mm)</th>
</tr>
</thead>
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<td>1330</td>
<td>1064</td>
<td>56</td>
</tr>
<tr>
<td>TS 3:1</td>
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<td>738</td>
<td>64</td>
</tr>
<tr>
<td>TS 3:2</td>
<td>1267</td>
<td>760</td>
<td>65</td>
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</tbody>
</table>

In the diagrams below the load versus the total deflection for the six beams is shown. What also can be seen in these diagrams are the limits for the resistance to vertical shear according to EC3-1-5, where more conservative maximum shear strength, \(0.58 f_{yd} \times (1.05 / \sqrt{3})\), are used and the resistance to vertical shear according to the less conservative, \(0.7 f_{yd} \times (1.2 / \sqrt{3})\), limit as the maximum shear strength, is also shown. All the six beams have a greater value for the maximum load than \(0.7 f_{yd}\). The resistance to vertical shear according to EC3-1-1(5.4.6) is marked in the diagrams as well. None of the tested beams reached that load level.

As a comparison, the elastic theoretical deflection due to bending and shear is also shown. At the end of the test the concrete were separated from the steel girder.
Fig. 6-12: Shear vs Deflection TS 1:1

Fig. 6-13: Shear vs Deflection TS 1:2
Fig. 6-14: Shear vs Deflection TS 2:1

Fig. 6-15: Shear vs Deflection TS 2:2
Fig. 6-16: Shear vs Deflection TS 3:1

Fig. 6-17: Shear vs Deflection TS 3:2
6.1.6.2 Girders with slender webs

The six girders showed small variations in the behaviour during the test and the maximum load for each one of the girders was almost the same.

The failure mode was web buckling in all the tests. The maximum shear and the total deflection under the load are shown in Table 6-5.

Table 6-5: Test results. Maximum shear and total deflection under the load at maximum load.

<table>
<thead>
<tr>
<th>Test specimen</th>
<th>Maximum Shear (kN)</th>
<th>Moment (kNm)</th>
<th>Total deflection at maximum load (mm)</th>
</tr>
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<tbody>
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In the diagrams the shear versus the total deflection under the load is shown. What also can be seen in these diagrams are the resistance to vertical shear according to EC 3-1-1 (5.6.3, Simple post-critical method) and EC 3-1-5. The largest steel flange is taken into account, according to EC 4, Part 2 Bridges (4.4.3(3)), for the four composite girders (TS 7:1 – TS 8:2). The resistance to vertical shear according to EC 4.1.1 (4.4.4) is also shown. None of the tested composite girders reached that load level. A and B represents web plate A and web plate B.
Fig. 6-18: Shear vs. deflection TS 5:1

Fig. 6-19: Shear vs deflection TS 6:1 A
Fig. 6-20: Shear vs. deflection TS 6:1 B

Fig. 6-21: Shear vs. deflection TS 7:1
Fig. 6-22: Shear vs. deflection TS 7:2

Fig. 6-23: Shear vs. deflection TS 8:1 A
Fig. 6-24: Shear vs deflection TS 8:1 B

Fig. 6-25: Shear vs. deflection TS 8:2 A
Fig. 6-26: Shear vs. deflection TS 8:2 B

Fig. 6-27: Interaction curve shear force and bending moment.
6.2 Numerical investigations

6.2.1 General

The aim of this FE-simulation is to generate an appropriate numerical model to perform sensibility- and parameter studies to extend the database to cases not tested. This has been done by a complete non-linear calculation, by considering the imperfections, large deformations and strains which have especially an important influence on stocky beams. In order to involve the physical non-linearity a complete stress-strain-curve was entered for the material law based on material tests.

The Finite-Element-Calculation is performed by the FE-Program *Marc* version 2000 and the graphic user interface *Mentat* version 2000 for pre- and post-processing.

6.2.2 Generating of proper models

6.2.2.1 Generation of the geometry

The generation of the steel beam has been done with an 8-node-shell-element which describes the behaviour of the buckled plate very well. The concrete plate is considered by an 8-node-volume-element. The joint between the steel beam flange and the concrete plate is discreted by links. These are elements with defined amounts of degrees of freedom between two nodes. It is known from the tests in Sweden and those documented in the literature that the significant influence of the concrete slab on the shear-bearing capacity can be considered by an increased slab stiffness. By considering all effects in the concrete slab, e.g. crack development and tension-stiffening, the model become to complicated and the buckling effect subordinated. Therefore no reinforcement is considered in the compression zone under sagging moment. Under hogging moment the reinforcement in the slab is considered only. To receive short running-times for the sensibility-and parameter study a simple

![FE-model](image.png)

Fig. 6-28: Half FE-model at the example of beam TS8

In a first step a generated test beam of a steel beam has been compared with the theoretic stiffness to determine the grade of the net and the types of elements until convergence. A further calibration has been performed by performing an Eigenvalue-analyses of a Navier-supported panel and a comparison over the Eigenvalues of the theoretic buckling solution.
The chosen 8-node-shell element “thick shell (22)” contains out of 4 Gauß-Integration points and has three displacement and rotation degrees of freedom. Two normal and three shear stresses are considered.

For the concrete 8-node-volume elements have been chosen which are connected to the steel flange by Links. Every node has three degrees of freedom – all six stress- and deformation components are considered. The differences between the linear trial function of the concrete and the non-linear trial function of the steel for the deformations have not shown any numerical problems.

![Fig. 6-29: Link-elements between the concrete and the upper steel flange](image)

Every FE-model starts with a pre-calculation to determine the lowest positive Eigenvalue which is compared with the real imperfection and the buckling shape of the test beam. In a second step the shape of this Eigenvalue is considered as a pre-imperfection with the measured values from the test beams. The following calculation steps consider big displacements and strains. The use of the big-strains-procedure is necessary due to the maximum strains are above 5%. Therefore the incremental solution of the equilibrium is necessary. The complete Newton-Raphson procedure has been chosen to obtain a quadratic convergence instead of a linear convergence of the modified Newton-Raphson procedure. By using a deformation-controlled calculation also maximums in the load-deformation-curve are obtained instead of using the arc-length procedure.

### 6.2.2.2 Material properties

The stress tensor can always be separated into two parts:

- spherical tensor (hydrostatic stress, volume dilatation)
- stress deviator (change of shape)
\[
\begin{bmatrix}
  s_{xx} & s_{xy} & s_{xz} \\
  s_{yx} & s_{yy} & s_{yz} \\
  s_{zx} & s_{zy} & s_{zz}
\end{bmatrix} =
\begin{bmatrix}
  \bar{s} & 0 & 0 \\
  0 & \bar{s} & 0 \\
  0 & 0 & \bar{s}
\end{bmatrix} +
\begin{bmatrix}
  s_{xx} - \bar{s} & s_{xy} & s_{xz} \\
  s_{yx} & s_{yy} - \bar{s} & s_{yz} \\
  s_{zx} & s_{zy} & s_{zz} - \bar{s}
\end{bmatrix}
\]

(6–1)

stress tensor \( \bar{S} \) = spherical tensor \( S_0 \) + stressdeviator \( \tilde{S} \)

\[
\bar{s} = \frac{1}{3} \left[ s_{xx} + s_{yy} + s_{zz} \right] = \frac{1}{3} \left[ s_{11} + s_{22} + s_{33} \right] = \frac{1}{3} I_1
\]

(6–2)

The sum of main and normal stresses does not depend on a transformation of the co-ordination system and is called the first invariant of the stress tensor. The three invariant of the stress tensor are:
\[ I_1 = \left[ s_1 + s_2 + s_3 \right] \]
\[ I_2 = -\left[ s_1 s_2 s_3 + s_3 s_1 \right] \]
\[ I_3 = \left[ s_1 s_2 s_3 \right] = \det \sigma \]

The spherical tensor is only an elastic value and changes only the volume but not the shape of the body. The changing of shape is described with the stress deviator.

By transforming the stress deviator on the main axis and showing the invariant of this tensor:

\[ \tilde{\sigma} = \begin{bmatrix} s_{xx} - \bar{\sigma} & s_{xy} & s_{xz} \\ s_{yx} & s_{yy} - \bar{\sigma} & s_{yz} \\ s_{zx} & s_{zy} & s_{zz} - \bar{\sigma} \end{bmatrix} = \begin{bmatrix} s_1 - \bar{\sigma} & 0 & 0 \\ 0 & s_2 - \bar{\sigma} & 0 \\ 0 & 0 & s_3 - \bar{\sigma} \end{bmatrix} = \tilde{\varepsilon} \]

\[ \tilde{I}_1 = 0 \]
\[ \tilde{I}_2 = \frac{1}{2} \left( \tilde{\varepsilon}_1^2 + \tilde{\varepsilon}_2^2 + \tilde{\varepsilon}_3^2 \right) = \frac{1}{6} \left[ (s_1 - s_2)^2 + (s_1 - s_3)^2 + (s_3 - s_2)^2 \right] \]
\[ \tilde{I}_3 = \left[ \tilde{\varepsilon}_1 \tilde{\varepsilon}_2 \tilde{\varepsilon}_3 \right] = \det \tilde{\sigma} \]

With \( \tilde{I}_2 \) an easy relationship between the three axial and the one axial stress state can be shown. With the assumption that the third invariant does not have any influence on material behaviour, the von Mises-Hypothesis is identified.

\[ \tilde{I}_2 = \frac{1}{3} f_\sigma^2 \]

The non-linear behaviour of steel is considered by using the von Mises-yielding condition. When reaching the yielding limit a „table“ is defined that approximates the stress-strain-curve as polygon known from the material tests. The program is using true stresses when calculating large strains.

\[ \sigma_w = \sigma (1 + \varepsilon) \]
\[ \varepsilon_w = \ln(1 + \varepsilon) \]

The ratio between the tension- and compression strength for the concrete is nearly 1:10. Furthermore the failure stress limit under compression depends significantly from the three-dimensional stress distribution. Therefore the linear Mohr-Coulomb-Law (also called Drucker-Prager-Model) is used – by linearisation of this parabolic approach convergence is reached. The formula for the yielding behaviour includes the dependence from the hydrostatic stress-state and from the deviator-stress-state in the following equation:

\[ \alpha \times I_1 + \sqrt{I_2} - \sigma_v \]

with \( a = \frac{s_{ck} - s_t}{\sqrt{3}(s_{ck} + s_t)} \) und \( s_v = \frac{2s_{ck} s_t}{s_{ck} + s_t} \)
Due to the material law maintains the stress after reaching the yielding limit instead of a cracking failure when exceeding the tensile strength the maximum tension strength is limited up to a value of the pure reinforcement in state II.

In order to consider the non-linear material behaviour of the incremental solution for the equilibrium the update Lagrange definition is chosen - after each increment not the original but the system of the last increment is used for the reference system.

### 6.2.2.3 Simulation of the tests

In order to keep the run-times as short as possible the large systems considers the symmetry by using the proper support (Eigenshapes have to be fulfilled). To be sure that the smallest possible Eigenvalue is calculated the Eigenvalue-analysis is checked at the entire system.

The geometry of the welded beams is simplified by considering the welding by an extra area of the web and flange. For the compact rolled profiles the rounded part is about ~ 20% of the shear-force area. Therefore a larger thickness is used for the elements nearby this area and smaller thickness is used for the rest of the web to obtain the same moment of inertia by keeping the area of the cross section.

### 6.2.3 Calibration

In order to obtain good results with the FE-calculation the simulations have been generated as realistic as possible by keeping simple FE-model. The calibration has been performed by comparing the load-deformation-curve, the load-strain-curve and the maximal buckling ordinate. The following restrictions have been made for the simulation:

- The slip-effects occurred in the supports, composite joint, etc. is not considered
- 7 different models will be generated:
  - 3 models for the 6 tests with compact webs (TS1, TS2, TS3)
  - 4 models for the 6 tests with slender webs. (TS5, TS6, TS7, TS8)
- The average values have been considered for pre-imperfections, geometry and materials
The simulation of the compact test beams with stocky webs and the concrete slab in compression is problematical due to the concrete cracked after leaving the linear path (1000kN). Furthermore no symmetry could be used because of the static system is a double-span beam with cantilever arm which led to very long calculating times.

By considering the loading situation the system is symmetric due to the support. However no symmetric deformation can occur.

During the tests the deflection of the cantilever beam was very high – therefore the concrete (in the compression zone) reached a level of tension stresses on the upper surface which led to a cross fracture in the slab. In the FE-calculation this effect of a sudden fracture of the whole slab could not be simulated. Test series TS2 did not fail due to shear buckling of the composite beam but yielding of the steel.

The composite beams under hogging moment behave similar to pure steel beams. A direct comparison is impossible due the dimensions of the flanges from the composite and the steel beam are different. Therefore the positive contribution of the reinforcement in the composite beam could not been clearly evaluated.
The composite beams under sagging moment have also different dimensions of the flanges comparing with the pure steel beams. Therefore no direct comparison is possible. Nevertheless the positive influence of the concrete slab is obviously - after reaching the maximum load the load capacity stays stable with a ductile load-deformation-curve. The maximum deformation is reached when plastic moments are formed in the composite beam and the static system becomes kinematic. By analysing the buckling-deformation curve the sudden buckling and stabilisation of the post-critical bearing capacity can be seen.

**Fig.6-33: FE buckling-deformation curve**

The load-deformation-curves as well as the load-strain-curves have shown a good accordance with the test results. In almost every test the bearing capacity of the FE-calculations is slightly higher than the real system. This is a typical effect from the FE-method due to the model will always remain an approximation to the reality no matter how exact the model is discretized. In the test set-up the assumed ideal conditions can never be accomplished. Therefore the difference of ~ 4% is acceptable - the models can be used for further studies.
6.2.4 Sensibility analysis

6.2.4.1 Pre-imperfection

The geometrical pre-imperfections, obtained from rolling and welding, have a significant influence on the bearing capacity of the buckling panel. The different standards (e.g. ZTVK 88, DIN 18800 T3, EC 3, etc.) include mostly the same value of $l_0/250$ for the maximal size for the bow for the buckling panel.

There are three different possibilities of considering these pre-imperfections in FE-calculations:

1. extra load which has no influence on the global general carrying behavior
2. entering of the imperfect system
3. loading the system by entering the imperfections according to the standards respectively

Eigenvalue-analysis without or by considering the measured shape of the.

For this FE-simulation the Eigenvalue-analysis has been chosen - the shapes were calibrated with the values measured from the test beams. For test specimens with stocky webs the influence of pre-imperfections is negligible. Therefore for those beams no Eigenvalue-analysis is necessary.

For the sensibility analysis the influence of pre-imperfections for the three test beams (TS5, TS7, TS8) with slender web have been examined. The bow for the buckling has been varied from 0 mm up to 10 mm.

Fig. 6-34: Bearing capacity of the web panel by considering different pre-imperfections, beam TS5

For all test specimen are the bearing capacities of the perfect system above the imperfect panel. The pure steel beams have shown this effect significantly. By reaching the critical load the system suddenly failed due to buckling. After reaching the maximum capacity the perfect system gets closer to the imperfect system. A small pre-imperfection of only 0.5 mm decreases the maximum bearing capacity of 25% - a further imperfection of 3 mm decreases the capacity of only 8%. The bearing capacity for such a perfect system is only a theoretic value.
The different curve-characteristics for the three different types of beams are typical and have been already obtained during the calibration. As expected no influence of the concrete slab on the pre-imperfection has been gained. The differences between bearing capacity of the perfect and the imperfect system are almost the same.
6.2.4.2 Material laws and strengths

Two independent material tests have been carried out in advance in Luxembourg (Arbed) and Sweden. The results of these tests show differences of up to 5% in the yielding strength.

Due these differences a sensibility analysis of the influence of the steel strength on the bearing capacity of specimen has been performed. Because of these values are pure steel parameters this sensibility analysis is only carried out on beams TS3 and TS5.

Fig. 6-37 shows a comparison on the bearing capacity for different steel strength for specimen TS 3. A decreasing of the material strength of 5, 10 and 15% decreases also the bearing capacity on the same percentage value. By considering a bilinear material law without any strain hardening the bearing capacity is as expected lower. For steel structures these values the highest bearing capacity is not relevant due to the deformations exceed the serviceability limits.

![Comparison on the bearing capacity for different steel strengths](image)

Fig. 6-37: Comparison on the bearing capacity for different steel strength for TS3

The comparison on the bearing capacity of the beam with a slender web TS5 the deformations are much smaller. (the load maximum is at 4mm, corresponding with L/250). Therefore the stress-strain-curves of the complete and bilinear material law are almost the same. The influence of the steel strength on the bearing capacity is also proportional - 10% less strength results in 10% less bearing capacity.
### 6.2.4.3 Uncertain parameters

The used FE-models are just approximations to the reality. A simulation, even if the calibration is extremely exact, can always only contribute the test series. Only the phenomena and tendencies are shown that have occurred during the tests.

The following parameters are uncertain:

- The iterations are as exact as 1.0%.
- The material strengths taken from the depend on the locations where the material was taken from
- The grade of exactly measurements to determine pre-imperfections depend on the measuring method (Photogrammetry, etc.)
- Slip effects (supports, composite joint) can not be simulated
- The model for the concrete slab can only simulate the global load-carrying behaviour. The local damage mechanism e.g. cracks, friction or cross section failures can not be simulated.

### 6.2.5 Parametric study

#### 6.2.5.1 Slenderness of the web, b/t-ratio

The slenderness of the web is one of the essential influencing parameters on the shear load capacity. The cross section can not develop the full plastic design load over a certain b/t-ratio due to buckling of the web. In addition the strength, the aspect ratio and the support condition of the buckling panel are further important parameters influencing the shear resistance. All these parameters are summarised in the web slenderness $\bar{\lambda}_w$ (compare to chapter 2):
\[
\bar{\lambda}_w = \frac{\tau_y}{\sigma_{crit}} = \frac{b/t}{37.4\varepsilon \sqrt{k_t}} \quad (6.1)
\]

Based on the theoretical solution a cross section is not buckling if:
\[
\sigma_{crit} \leq f_y \quad (6.2)
\]

Substituting this relationship into expression (6.1) and solving for b/t results in
\[
\lim b/t = \frac{37.4}{\sqrt{3}} \varepsilon \sqrt{k_t} = 28.42\varepsilon \sqrt{k_t} \quad . \quad (6.3)
\]

The values for non-stiffened beams are resulting from the limiting value of
\[
k_t = 5.34 + 4.0 \frac{b_w^2}{a^2} \quad (6.4)
\]

which is 5.34 for large a/b-ratios. Comparing these requirements with the standards the values in the standards are slightly higher than the theoretical values.

**Table 6-6: Comparison of the limiting b/t-values**

<table>
<thead>
<tr>
<th></th>
<th>Limiting (\bar{\lambda}_w)</th>
<th>limiting b/t</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Stiffened beam</td>
<td>Non-stiffened beam</td>
</tr>
<tr>
<td>Theory</td>
<td>0.76</td>
<td>28.42 (\varepsilon \sqrt{k_t})</td>
</tr>
<tr>
<td>EC3 T1.1</td>
<td>0.802</td>
<td>30 (\varepsilon \sqrt{k_t})</td>
</tr>
<tr>
<td>EC3 T1.5 (\eta=1.05)</td>
<td>0.83/(\eta)=0.793</td>
<td>29.52(\varepsilon \sqrt{k_t})</td>
</tr>
<tr>
<td>EC3 T1.5 (\eta=1.20)</td>
<td>0.83/(\eta)=0.694</td>
<td>25.83 (\varepsilon \sqrt{k_t})</td>
</tr>
</tbody>
</table>

These slightly higher limiting values can be explained by the fact, that over-critical reserves are neglected in the theoretical result. Further exists a semi-rigid clamping of the web due to the flanges which is not considered.

In the research report of the performed tests it is determined, that the plastic bearing capacity of EC3 part1.1 is significantly higher than the capacity of the tested beams. Therefore the limiting b/t-ratio and the behaviour of the beam in the plastic region was investigated more closely.

The FE-Calculation is based on the beam TS5. Generally 250 comparable calculations have been performed for the parametric study. Therefore, the calculation time was minimised. Some simplifications have been made to receive comprehensible results and to exclude the influence of unwanted parameters:

- The geometry is rounded to integers
- The steel has got a unified yield strength (S235, S355, S420, S690)
- A bilinear stress-strain relation is used (elastic- ideal-plastic)
- The load-deflection path is only plotted up to 10 mm deflection

In the parametric study the web thickness is varied between 3 and 16 mm for the different steel grades and the load-deflection curves are plotted. A specified overview of the calculated beams is given in the Annex. Table 6-6 shows the different geometry of the tested beam TS5 and the beams with the similar geometry (bt707 and bt706) which are used for the parametric study.

<table>
<thead>
<tr>
<th></th>
<th>bw</th>
<th>tw</th>
<th>bfl,o</th>
<th>bfl,u</th>
<th>tfl,o</th>
<th>tfl,u</th>
<th>bw/tw</th>
<th>Av</th>
<th>lambda</th>
</tr>
</thead>
<tbody>
<tr>
<td>TS5</td>
<td>698.25</td>
<td>6.58</td>
<td>250.78</td>
<td>249.6</td>
<td>15.21</td>
<td>15.29</td>
<td>106.12</td>
<td>4594.49</td>
<td>1.4297</td>
</tr>
<tr>
<td>bt707</td>
<td>700.00</td>
<td>7.00</td>
<td>250.00</td>
<td>15.00</td>
<td>100.00</td>
<td>4900</td>
<td>1.3230</td>
<td>4900</td>
<td>1.3230</td>
</tr>
<tr>
<td>bt706</td>
<td>700.00</td>
<td>6.00</td>
<td>250.00</td>
<td>15.00</td>
<td>116.67</td>
<td>4200</td>
<td>1.5435</td>
<td>4200</td>
<td>1.5435</td>
</tr>
</tbody>
</table>

Table 6-7: Overview of the geometry

Fig. 6-39 shows, that the curve of the tested beam TS5 is exactly between the two curves of the parametric study (bt707 and bt706). This fits exactly to the web area ratio.

Thus compact beams with a web thickness of 16 mm (b/t = 43.75) should behave plastically. Because of the simplification of the stress-strain relationship the bearing capacity could be lower for these beams. Therefore, a comparing calculation between the complete stress-strain curve to the bilinear approximation is carried out. For the comparing calculation the strain area is additionally enlarged to estimate the increase of bearing capacity.
Fig. 6-40: *Simplification of the stress-strain relationship*
It is obvious, that the run of the complete strain-deflection curve is equivalent to the bilinear approximation in the area of strain up to 10 mm. For 3 times higher deflections (30 mm = L/66), which is much higher than the serviceability limit, the bearing capacity is only increased about 1.15%. This increase is negligible.

In the following the curves for the beams bt700 are introduced. The geometry of the presented beams is equivalent to the specification in Table 6-7. The web thickness is varying between 3 and 16 mm.

Fig. 6-41 shows, that beams with small b/t-ratios have got a ductile run of the load-deformation curve while slender beams develop a maximum peak. The load increases until buckling occurs rapidly. Therefore the load drops, and the systems stabilises due to over-critical bearing capacity of the plate on a lower level. The limits for this behaviour are between a b/t-ratio of 700/9 = 77.8 (bt709) up to 700/10 = 70 (bt710). If the b/t-ratio is further increased, the output forces will be too high and no maximum peak will be developed. The change to this behaviour is between 700/6 = 116.7 (bt706) up to 700/5 = 140 (bt705).

![Load-deflection curve for various b/t-ratios](image)

Fig. 6-41: Load-deflection curve for the different b/t-ratios of the beams bt700

Relating the bearing capacity to the web area the run of the curves could be compared. The results are presented in Fig. 6-42.
Fig. 6-42: Related load-deflection curve of the test series bt700

Beams with compact webs are not reaching their plastic design bearing capacity, as it is obvious in Fig. 6-42, although the run of the load-deflection curve is ductile. The reason for this behaviour is, that with increasing absolute shear capacity the moment bearing capacity of the flanges utilised and further parts of the web are drawn on the moment bearing resistance. Keeping similar geometry the thickness of the flanges are increased up to 40 mm in a second calculation.

Fig. 6-43: related load-deflection curve of a beam with a flange thickness of 40 mm

The plastic bearing capacity is reached. The run of the curve with the flange thickness of 40 mm is affin to the curves of Fig. 6-41 and Fig. 6-42
Influence of the flanges

Fig. 6-44: Comparison of the FE-Calculation to the theoretical curves

The result of the FE-calculation with a constant flange thickness of 40 mm is opposed to the run of the theoretical curves of the idealised shear panel and the modified respectively the rotated stress panel in Fig. 6-44. The theory of the idealised shear panel uses absolute rigid flanking elements. The 40 mm thick flanges represent more and more rigid flanking elements for decreasing thickness of the web. Thus, the run of the FE-curve for very slender webs could be explained. The run of the curve for the modified stress panel theory is added for comparison additionally in Fig. 6-44. The curve grasps the post critical behaviour of the beam due to the turnaround of the principle stresses and the linked stress redistribution in the web (see chapter 2). The capacity of non-stiffened and stiffened web plates can therefore be described properly. As it is shown in Fig. 6-44 the differences between the two run of the curves is extending with increasing slenderness. Additionally the shear stress curve of EC3 Part 1.5 is given with $\eta=1.05$. Generally the run of the curve of the EC3 matches to the FE-calculation curve for small and large slenderness of the webs. In the medium areas of slenderness the run of the EC3-curve is higher.

In a further calculation the thickness of the flange is varied in the way, that the bearing capacity of the flanges is completely utilised at shear failure. Thus, no additional shear panel can be anchored to the flanges. The pure shear bearing capacity of the web is received. Referring to Fig. 6-45 also the beams with the variable flange thickness reach the shear bearing capacity $\tau_{plast}$. Concerning to the slenderness of the web, the changes between plastic behaviour and failure due to buckling are analog to the previous test serial.
The trends in a $\tau$-$\lambda$-diagram of the FE-calculation of 3 different flange thickness’ are presented in Fig. 6-46. Comparing these 3 curves of the different test serials the influence of the flange becomes obvious. The curve for the variable flange thickness and the curve for the flange thickness of 40 mm are reaching the level of full plastification. The graph of the test serial with the 15 mm flange thickness reaches the maximum at a slenderness of $\sim$1 but drops with decreasing slenderness. Simultaneously increases the bearing capacity in the area of larger slenderness due to the flanges alone (idealised shear panel).
For the investigation of the limit for the b/t-ratio the test serial with the variable flange thickness is calculated for various steel grades (S690, S420, S355 and S235). The particular related curve ($\tau_u/\tau_y$ compared to $\lambda_w$) is plotted. The related curve is independent of the steel strength. But the ration of $f_u$ to $f_y$ is different. Therefore, it is possible to reach a varying plastic bearing capacity.

Fig. 6-47 presents the related curve for the steel grades S690, S420, S355 and S235.

The FE-calculation shows, that the related curve is independent of the steel grades. The curves of the steel grades S690, S420, S355 and S235 are nearly the similar. The change to the plastic behaviour (intersection of the curve with the ordinate $\tau_u/\tau_y =1$) is at a $\lambda$-value between 0.8 to 0.9. This represents a limiting b/t-ration of $29.92 \varepsilon \sqrt{k_t}$ respectively $33.7 \varepsilon \sqrt{k_t}$. This means, that the FE-calculation represents a good approximation of the existing standards (see Table 6-6) considering that the FE-calculation is not applying any safety factors.

![related ultimate stress curve for various steel grades without Interaction of the flanges](image)

Fig. 6-47: Related ultimate stress curve for various steel grades

Fig. 6-48 shows the ultimate shear stress curve of the EC3 Part1.5 for $\eta = 1.05$ respectively for $\eta = 1.20$ for comparison with the FE-calculation. The run of the ultimate capacity in relation to the slenderness fits nicely with the FE-calculation. The EC3-curve runs on a lower level due to the concept of the partial safety factor.

The additional plastic plateau with a 20 % increased shear bearing capacity ($\chi = \eta = 1.2$) out of EC3 Part1.5 can not reconstructed in the strain range of the FE-calculation. Although the full material concept is applied for the compact cross-sections. Like mentioned before, a triplication of the strain range ends only up to a 1.15 % increased ultimate shear capacity. A further increase of the strains results into a higher ultimate shear capacity.
Fig. 6-48: Comparison of the EC3 Part1.5 to the calibration factors of the FE-calculation

There are two models in the EC3 Part1.1 “Background document“ to capture the plastic reserves of the cross section. The first one is a plate model, which assumes the slenderness to be $\lambda \leq 0.2$. It is presumed that the flanges can only fail after the web is fully plastified. The second model is a truss system, which acts on the assumption, that the flanges develop plastic hinges and fail similar to the Vierendeel mechanism. Both models include, that the web plastifies before the bearing capacity of the flanges is fully utilised. This means, that parts of the flanges act to the disposal of the shear bearing capacity. This fact is considered by increasing the area of shearing force in the EC3 Part1.1.

The run of the von Mises stresses with increasing load of the parametric study $f$ and $q$ are presented in the following Table 6-8. The beam of the test serial $f$ is reaching the bearing moment of the flange $M_{fl,Rk}$ after the web is fully plastified. The flange of the beam in the test serial $q$ has got a 50 mm flange and consequently a 40 % higher flange bearing moment. The beams have got the following geometry:

Table 6-8: Parameters of the parametric study $f$ and $q$

<table>
<thead>
<tr>
<th>Parametric study $f$</th>
<th>Parametric study $q$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$t_w$</td>
<td>16.00</td>
</tr>
<tr>
<td>$\lambda$</td>
<td>0.57</td>
</tr>
<tr>
<td>$V_{pl}$</td>
<td>2736.0</td>
</tr>
<tr>
<td>$t_{flange}$</td>
<td>35.14</td>
</tr>
</tbody>
</table>
It is possible to notice the differing failure mechanism in Fig. 6-49 and Fig. 6-50. The beam with the thinner flange is partly plastified. The bright areas represent the parts where the local plastification limit is exceeded. Therefore plastic moments establish in the chords before the whole web is plastified.

The behaviour of the beam with the 50 mm flange is different. The web is fully plastified before the chords are reaching the plastification limit.
Both load-deflection curves are additionally given to compare the bearing behaviour of the two beams. The bearing capacity of the beam $q$ is 7.6% higher than the capacity of beam $f$.

Fig. 6-50: Run of the von Mises stresses for the parametric study $q$
Influence of the flange on the plastic bearing resistance
Comparison of test serial q and f

Fig. 6-51: Influence of the flanges on the plastic bearing capacity
6.2.5.2 Effect of the concrete slab on the shear buckling behaviour

As shown in the previous chapter the shear bearing behaviour of steel beams is effected by the geometry of the flanges. Therefore, it can be supposed that an increase of the flange stiffness by a concrete slab will enlarge the shear capacity. However, it has to be differentiated between the cracked concrete slab in regions with negative bending moments where only the reinforcement participates in the resistance and the fully effective concrete slab under a positive bending moment. Different models will therefore be used in the FE-simulations for the two different load mechanisms. The investigations are based on the parametric study for the b/t-ratio.

6.2.5.2.1 Concrete Slab in areas of negative bending moments

Based on the calibrated beam TS7 a reference will be done to the steel beam TS5. To be able to compare the two beams, the flange, which is connected to the concrete slab, will be designed equal to the flange of the pure steel beam. A change of the bearing capacity can then be ascribed to the reinforced concrete slab.

The effect of the flanges changes depending on the web slenderness. Therefore, the resistance behaviour of the composite beams will again be investigated for a web slenderness between 16/700 and 3/700.

For test series TS7 8 Ø 12 /160 has been chosen for the upper and lower reinforcement. This results in 2*7.07 cm²/m = 14.14 cm²/m. For an effective width of 0.375 m the chargeable reinforcement area can then be calculated to 5.03 cm². Related to the flange area the following values are obtained (Table 6-9):

<table>
<thead>
<tr>
<th>TS7 test</th>
<th>TS7 parametric study</th>
</tr>
</thead>
<tbody>
<tr>
<td>A_flange</td>
<td>A_reinf / A_flange</td>
</tr>
<tr>
<td>0.8*20=16 cm²</td>
<td>5.03/16 = 31.4%</td>
</tr>
</tbody>
</table>

As the ratio between A_reinf/A_flange is rather small, the effect of the reinforcement on the shear capacity will also be small. A complete notation of all beams and the geometry of the considered beams is presented in the Annex.

Fig. 6-52 presents the curves of the related shear capacity for the steel and for the composite beam. Both beams have a flange thickness of 15 mm.
It can be seen from Fig. 6-52 that only for small slenderness ($\lambda \sim 1$) an effect is visible. The composite beam can reach a larger shear capacity than the pure steel beam, even when the pure steel beam, due to interaction, does not reach the plastic shear resistance. This can be explained by the fact that the reinforcement contributes to the resistance of the normal forces and thus, the web of the composite beam has to resist less normal stresses. The effect of the reinforcement will increase for an increased ratio of the reinforcement area to the flange area.

Fig. 6-53: Related load-deflection curves for the test series i700 with variable flanges
Fig. 6-53 shows the related load-deflection curves of the beams with variable flanges for a varying web slenderness. It can be seen that the curves have the same run as the curves of Fig. 6-45. Furthermore, the maximum shear capacity of 2736 kN for the steel beam and 2734 kN for the composite beam are nearly the same. The curves of Fig. 6-54 show again the related shear capacity of the steel and the composite beam with variable flange. The run of the curves is nearly identical.

**Related ultimate shear stresses curve**

**Comparison between TS5 and TS7**

![Graph showing comparison between TS5 and TS7](image)

**Fig. 6-54: Comparison between steel and composite beam for a variable flange thickness**

Thus, it is shown that the reinforcement does only effect the shear capacity when the resistible moment is smaller than the acting moment $M_{sd}$, thus only parts of the web are needed for the resistance of the moment. If $M_{flange}$ is larger than $M_{sd}$, an increase of the shear capacity will not be obtained by further reinforcement.

This is verified by another parametric study (m), where the area of the reinforcement is equal to the area of the flange. Again the flange thickness is variable.

**Table 6-10: Related reinforcement for TS7 with variable flange**

<table>
<thead>
<tr>
<th>TS7 parametric study d</th>
<th>TS7 parametric study i</th>
<th>TS7 parametric study m</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_{flange}$</td>
<td>$A_{reinf}$</td>
<td>$A_{flange}$</td>
</tr>
<tr>
<td>1.5*25</td>
<td>5.03/16</td>
<td>variable</td>
</tr>
<tr>
<td>= 37.5 cm²</td>
<td>= 13.4%</td>
<td></td>
</tr>
</tbody>
</table>
Deviations of 0.5% occurred in the numerical evaluation. However, these are below the accuracy of the numerical calculations which was set to 1%. Thus, it can be said that the same result is obtained for the two test series.

Hence, the total effect of the reinforcement on the shear behaviour is very small. An increase of the shear capacity can be shown when the flange of the steel beam is fully used by the moment loading and additionally the web is used for the resistance of the normal stresses. However, this is merely an interaction problem.

### 6.2.5.2.2 Concrete slab in positive moment areas

The influence of the concrete slab in a more ductile load-deflection behaviour has already been shown in the calibrating curves in chapter 5. Referring again to the calibrated beam (TS8) it is necessary to adjust the flanges to beam TS5 in order to enable a reference to the pure steel beam. For this beam series the load deflection curves are then again calculated for different b/t ratios. The following beams have been considered:

**Table 6-11: Examined composite beams for sagging moment**

<table>
<thead>
<tr>
<th>t_{flange} [mm]</th>
<th>e</th>
<th>g</th>
<th>n</th>
<th>p</th>
</tr>
</thead>
<tbody>
<tr>
<td>b_{w} [mm]</td>
<td>15</td>
<td>variable</td>
<td>variable</td>
<td>variable</td>
</tr>
<tr>
<td>t_{w} [mm]</td>
<td>700.00</td>
<td>680</td>
<td>680.00</td>
<td>680.00</td>
</tr>
<tr>
<td>h_{c} [mm]</td>
<td>16</td>
<td>16</td>
<td>16</td>
<td>16</td>
</tr>
<tr>
<td>b_{c} [mm]</td>
<td>100</td>
<td>100</td>
<td>150</td>
<td>100</td>
</tr>
<tr>
<td>b/t</td>
<td>570</td>
<td>570</td>
<td>570</td>
<td>855</td>
</tr>
<tr>
<td></td>
<td>43.75</td>
<td>42.5</td>
<td>42.5</td>
<td>42.5</td>
</tr>
<tr>
<td>max V [kN]</td>
<td>112</td>
<td>108.8</td>
<td>108.8</td>
<td>108.8</td>
</tr>
<tr>
<td></td>
<td>2636.7</td>
<td>2887.1</td>
<td>3134.3</td>
<td>2931.9</td>
</tr>
</tbody>
</table>
Fig. 6-56 shows the related load-deflection curve for parameter series e. It can be seen that for the beams with a small b/t-ratio in contrast to those steel and composite beams with the concrete slab in the tension zone (see Fig. 6-42 and Fig. 6-52) reach a plastic level, although the flange thickness is only 15 mm.

![Related load-deflection curves for various b/t-ratios](image)

Fig. 6-56: Related load deflection curve for the parametric study of series e

![Related load-deflection curve](image)

Fig. 6-57: Comparison between steel and composite beam in area of positive moments
Comparing the pure steel beam with a composite beam in areas of positive bending moments, it becomes obvious that for large slenderness ratios the bearing capacity remains constant from a special slenderness on. This is similar to the theory of the ideal stress field. Naturally, this decrease of the bearing capacity is significantly smaller for smaller slenderness ratios because of the interaction. However, it can be seen that the beams reach the plastic level.

When the steel and the composite beam with variable flange, which then has no contribution on the different slenderness ratios, are compared, the effect of the concrete slab can be realised.

![related ultimate shear stress curve](image)

**Fig. 6-58: Comparison of the shear resistance without a contribution of the flanges**

It can be seen from Fig. 6-58 that for small slenderness ratios the curve for the composite beam is parallel to that of the steel beam but on a higher level. With increasing slenderness ratios the curves diverge, the influence of the concrete slab grows. The post-critical resistance reserves increase with increasing slenderness, hence, the width of the tension field is enlarged with a raise of the stiffness of a flange. Again, an approach to the ideal tension field occurs.

For parameter series g the ultimate limit state of the beams g03 with a web thickness of 3 mm and of beam g16 with a web of 16 mm thickness is presented in the following. For beam g16 it can be seen that no plastic hinges develop in the flanges and the web is fully plastified.

Due to its large slenderness beam g03 shows the typical buckling behaviour with a tension-fold and two half-cycles. Locally the stresses reach the yield strength.
Fig. 6-59: Composite beam g03 with a web thickness of 3 mm and a variable flange thickness

Fig. 6-60: Composite beam g16 with a web thickness of 16 mm and a variable flange thickness
As it is known from former models of steel and composite beams and as it can be derived from the failure mechanism of the plastic hinge method, the effect of the flanges on the shear capacity depends on its flexural rigidity. Thus, the size of the concrete flange will be varied in the following calculations. At first the thickness of the concrete flange will be increased from 100 mm up to 150 mm (series \( n \)). Secondly the width will be increased by the factor 1.5 (series \( p \)), ensuring this way that the area remains constant with regard to the enlarged thickness. The width is with 855 mm still below the effective width according to EC4 \( (b_{\text{eff}}=875 \text{ mm}) \), cf. chapter 4.2.5.3). The results are presented by Fig. 6-55. Additionally the related shear capacity curve for a steel beam \( (f) \) and for the composite beam \( (g) \) with the original concrete geometry are presented as a reference.

**Comparison of the various types of composite beams**

All the curves for the composite beams have a similar run. They are displaced with regard to the basic curve (series \( g \)) depending on the slenderness. The curve of series \( n \) with the 150 mm high concrete flange lies significantly above the other curves whereas the curve of series \( p \) with the width enlarged by the factor 1.5 and the same concrete area than series \( n \) is placed only negligible above the basic curve. It can be seen clearly that the influence of the concrete slab is not that significant for small slenderness ratios as for larger ones.

Comparing the calculated results with the bearing capacity curves presented by the different Eurocodes, it can be seen that the raised shear capacity curve according to EC4 enlarges the applicable shear force for special ranges, however, the run of the bearing behaviour is different as the one from the FE-simulations. The raised shear capacity curve according to EC4 has been calculated only for composite beams without stiffeners subjected to uniform loads. Hence, a comparison between the standard curve and the FE-simulation is difficult. In addition the bearing capacity curve for the contribution of the web according to EC3 Part1.5 is presented. In the EC3 the contribution of the flanges is calculated depending on the geometry of the flanges but independent of the web geometry. This way a dependence between the effect of the flange geometry and the web slenderness is obtained.

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**Fig. 6-61: Comparison of the different composite beams**

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**Final Report**
Comparison of the composite beams with the standards

Fig. 6-62: Comparison between the parametric study and the standard curves
6.3 Evaluation of results and conclusions

6.3.1 Analysis of the experimental tests

6.3.1.1 Discussion

All the six beams with stocky webs shows the same behaviour during the test. The presence of the concrete slab does not make any difference. The inclination of the elastic part of the load-deflection curves was almost the same for all three test specimen. Therefore the flexibility in the girders was almost the same. The concrete slab did not at all influence it, which is a little bit surprising because the concrete was expected to give some decrease of the flexibility.

The theoretic elastic shear deflection was about 70 % and the bending deflection was about 30 % of the total theoretic deflection for test specimens TS 1:1, TS 1:2, TS 3:1 and TS 3:2. That relationship was 65 % in bending deformation and 35 % in shear deformation for TS 2:1 and TS 2:2. Test specimens TS 2:1 and TS 2:2 is much longer than the other, and therefore the bending deformation has a greater influence.

It is quite obvious that the less conservative shear strength, $0.7f_{yd}$ can be used even for steel of grade S 460. As can be seen in Figure 6-12 to 6-17, the maximum loads for all the six rolled beams are greater than the resistance to vertical shear calculated with $0.7f_{yd}$ as the maximum shear strength.

Furthermore it seems like the method presented in EC3-1-1 (5.4.6) for rolled sections, gives a too high resistance to vertical shear, since none of the tested beams reached that load level. There will be a big difference between the calculated resistance to vertical shear according to the two methods in EC3-1-1. The method in clause 5.4.6 gives almost twice as high resistance to vertical shear as the one according to EC3-1-1 (5.6).

In Figure 6-27 the interaction between the shear force and the bending moment is shown and all the test beams are above the limit for resistance to vertical shear according to the less conservative method of EC3-1-5.

There can be at least two different possible explanations to the fact that the maximum load was greater than expected according to the codes. The first thing is that the strain hardening can be greater. In the tests there were no distinct yield plateau. The strain-hardening started after the elastic part of the curve. The strain at the $0.7f_{yd}$ limit was quite small, around 0,2 % – 0,6 % for test specimen TS 1:1, TS 1:2, TS 3:1 and TS 3:2 and at the maximum load the strain was around 2,5 % so the material did yield for quite long time before the web buckle.

For TS 2:1 and TS 2:2 the strain had a completely different behaviour. The strain decreases at a level where the shear was about 1200 kN. The second possibility is that there is a positive influence from the flanges. It is likely that it is a combination of these two effects. The slenderness of the web is about 32 and according to EC3-1-5 there is no contribution from the flanges at all when the web is as stocky as it is in this case. As can be seen from the test results, the maximum load is greater than what is the case for only taking $0.7f_{yd}$ into account, therefore it is possible that the flanges can give a positive effect to the resistance to vertical shear after all.

The concrete slab does not show any influence, what so ever, on the resistance to vertical shear, not for the test specimens in hogging bending anyway. The test specimens in sagging bending on the other hand indicated that there could be some influence compared to the specimens in hogging bending, the maximum load was about 5 % higher. Maybe it is so that the test specimens where designed in such a way that the effect from the concrete slab was not possible to be seen in these
tests. But in order to get failure caused by the shear force the dimensions of the test specimens has to be as chosen. It was possible to perform the test in either of two ways. One way was to apply the load on top of the concrete slab, which would have been too favourable for the effect from the slab. This would make it impossible for the concrete slab to lift and slip relative the steel beam.

The other way to perform the test was to apply the load on the end plate of the steel beam, which was actually done in the tests, but this would probably be too unfavourable for the effect from the concrete slab. The reason is that the slab is too short for transferring all horizontal shear. As could be seen in the test results there was no influence from the concrete and maybe the reason was that this test method in fact was too unfavourable.

For tests TS 1:1 and TS 1:2 the concrete slab cracked initially over the intermediate support. At a load level about 1100 kN a second failure mode occur, a block of concrete come loose at the end of the slab and become separated from the rest of the concrete slab. This failure mode was the result from a combination of longitudinal shear in the concrete and tension forces from the shear studs caused by the bending in the beam. The magnitude of this tensile stress is 0.5 MPa, which is a rough estimation.

The movement of the concrete slab relative to the steel beam was quite small and for all four composite beams the vertical as well as the horizontal movement was around 0.20 mm at the termination of measurements at approximately 1000 kN. After that, the cracking of the concrete made the measurements useless.

The maximum shear for all the six girders with slender webs was almost the same, even so for the composite girders, which have one of the steel flanges smaller than the other one.

According to EC 3-1-5 the smallest flange is supposed to be taken into account when calculating the resistance to vertical shear. This will lead to a lower resistance to vertical shear. From the tests it is quite clear that the concrete will give some positive effect on the resistance to vertical shear, hence the maximum shear is about the same as for the test specimens without concrete. The size of the both flanges for the test specimens without concrete were the same as the biggest one for the composite girders. This indicates that the concrete slab will give some positive effect on the anchorage of the extra tension field carried by the flanges.

Earlier test performed by Allison et. al. have shown that the concrete slab will have some influence on the resistance to vertical shear for slender webs. The reinforcement was yielding at ultimate load and the tensile stress in the steel flange was lower than predicted. This gave a positive effect on the resistance to vertical shear, which was interpreted as an increase of the flange contribution.

The effect from the concrete slab might be greater for a more slender web. The reason is that the extra tension field caused by the flanges has a greater influence on the resistance to vertical shear and therefore the size of the flanges will be more important when the web is slender. The concrete slab will make it possible to anchor the tension field over a greater length and that will be positive for the resistance to vertical shear.

What can be seen from the shear versus deflection diagram and the shear resistance versus slenderness diagram, is that the method described in EC 4-1-1 (4.4.4) gives a much too high resistance to vertical shear for composite girders. It might be argued that it is not applicable to S460 and neither to a girder with one concentrated load. However it is so large deviation that the reason should be investigated.

The method described in EC 4-2 Bridges meaning that the method in EC 3-1-5 can be used for composite bridges and taking the largest steel flange into account, gives a much better result but a bit on the safe side.
Table 6-12: Relationship between the shear and the bending part of the total theoretic deflection.

<table>
<thead>
<tr>
<th></th>
<th>Shear (%)</th>
<th>Bending (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TS 5:1</td>
<td>80</td>
<td>20</td>
</tr>
<tr>
<td>TS 6:1</td>
<td>45</td>
<td>55</td>
</tr>
<tr>
<td>TS 7:1</td>
<td>80</td>
<td>20</td>
</tr>
<tr>
<td>TS 7:2</td>
<td>80</td>
<td>20</td>
</tr>
<tr>
<td>TS 8:1</td>
<td>66</td>
<td>34</td>
</tr>
<tr>
<td>TS 8:2</td>
<td>66</td>
<td>34</td>
</tr>
</tbody>
</table>

Test specimens TS 6:1 is much longer than TS 5:1, TS 7:1 and TS 7:2 and therefore the bending deformation has a greater influence. TS 8:1 and TS 8:2 the effect from bending is smaller than for TS 6:1 and the reason is that second moment of area is greater because of the compressed concrete.

6.3.1.2 Conclusions of the experimental tests

- The effect of the concrete slab on the resistance to vertical shear is negligible for composite beams with stocky webs. There could be some effect when the concrete is in sagging bending.
- The resistance to vertical shear may safely be taken from EC3-1-5 with $\eta = 1.20$ also for S 460
- The resistance according to EC3-1-1 for rolled beams seems to be too high.
- There is an effect from the concrete slab on the resistance of slender webs to vertical shear, for both the cases with the concrete in sagging as well as in hogging bending. This can be taken into account by using the resistance according to EC 3-1-5, using the non-composite flange for calculating the contribution from the flanges.
- The resistance to vertical shear according to EC 4-1-1 seems to be much too high.

6.3.2 Conclusions of the simulation

The buckling bearing capacity is proportional to the steel strength. An increasing of 10% in the steel strength lead to a higher buckling bearing capacity of the same percentage. The strain hardening has a significant influence on the stocky webs due to large deformations and plastifications whereas the bilinear material law is sufficient for the slender webs.

The pre-imperfection has an enormous influence on the load-deflection curve and on the maximum bearing capacity. By using an unrealistic perfect system the maximum bearing capacity is 25% - 30% higher than considering an imperfection of l/2000 for the panel. After reaching this theoretical buckling load the curve converge to the same level of the imperfected buckling curve.

The parameter study over the b/t-relation has considered the steel strength S690, S420, S355 and S235 (with a fixed ratio for $f_u/f_y = 1.2$). The determined limit b/t-relations in the FE-simulation for all steel strength are slightly higher than the given values in EC3 – therefore the EC3 is safe-sided.
The limit value for considering buckling in EC3 Part 1.5 is given to 0.83/η. For steel strength higher S355 η is given to 1,05 - therefore the limit b/t-relation is b/t ≤ 0.79. The increasing of the plastic level of 5% lead to a higher shear resistance of the same percentage. All FE-calculations without interaction confirm this bearing capacity of 1,05.

A positive influence of a composite action with the concrete slab under tension has not been observed in the FE-simulation. The bearing capacity for these composite beams under hogging moment have reached the same level as the pure steel beams. The only increasing of the bearing capacity has been noticed under interaction (M_{Fl} < M_{Sd}).

The composite action under sagging moment has shown a significant increasing of the shear bearing capacity. Comparing with the steel beams the load-deflection behaviour of the composite beams are more ductile. The increasing of the bearing capacity is depending on the slenderness. In the plastic part and a slenderness λ < 1.5 an increasing of 20% according to EC3 Part 1.5 has been reached. Above a slenderness of λ =1.5 this profit is growing above 20% depending on the slab thickness. Fig. 6-63 shows the profit of the slab geometry on the shear bearing capacity. A proposal for a considering of the concrete slab on the shear bearing capacity for composite beams under positive moment could be the following formula. This proposal should be verified by further test series.

\[ ?_{\text{concrete}} = k \, b_c \, t_c^2 \]  \hspace{1cm} (6–10)

The test series performed in Sweden have shown that the measured maximum bearing capacity for stocky webs was 8-10% below the calculation according to EC3 Part 1.1 (5.6.3) which allow to increase the effective shear area for rolled beams. This calculation lead to unsafe results in the design – the results in this FE-study can accompany further investigations to solve this problem.

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Fig. 6-63: Influence of the concrete slab on the shear bearing capacity
6.4 References


7 Effects from the Interaction with the Concrete Slab of the Bridge

7.1 Cracking of the slab and of the joints

Cracks in concrete slabs influence the stiffness of composite beams. The effects and limitations of the crack characteristics are described by an elastic model.

The behaviour of composite beams subjected to tension in the concrete slab is highly non-linear, due to the formation and propagation of cracks. The reinforced concrete slab acts as a tension member whose axial capacity arises mainly from the reinforcement, and to some extent from the concrete. The contribution of the concrete may be related to the formation of cracks after the tensile strength of the concrete is exceeded. The crack propagation and the crack width growth can be described using an elastic model.

Two phases define the crack behaviour. In the initial phase, the cracks are spaced at a minimum distance equal to the anchorage length of the reinforcing steel. The characteristic material property is the tensile strength of the concrete. In the following phase, increased tensile forces cause the formation of more closely spaced cracks. The main parameter for the spacing is the shear transfer between steel and concrete.

The density of the distribution of cracks, based on probability assumptions, leads to mean values for the stiffness and the stresses in the concrete slab and the reinforcement.

7.1.1 Scope

Due to the fact that the actual design code for composite structures - Eurocode 4 – only refer to Eurocode 2 concerning design methods for the calculation of the crack width of composite members under negative moment bending or tension load, this report will clarify which specific calculation methods can be used concerning composite members and especially concerning composite bridges.

The following steps were done:

- Literature research to find actual calculation methods for the calculation of the crack width in the concrete slab
- Examination of the found calculation methods
- Verification of the main parameters of the calculation methods
- Extensive parameter study of the main parameter
- Literature research to find tests on composite members and measured crack widths
- Comparison of the results of the calculation methods with test results
- Result of the comparison and proposals for the calculation of the maximum crack width of composite bridges
7.1.2 Methodology

7.1.2.1 Literature research

After an extensive literature research four calculation methods could be found. Two of them were developed only for the calculation of the crack widths in concrete members but the last two methods were developed for composite members.

Table 7-1: Available calculation methods

<table>
<thead>
<tr>
<th>Author</th>
<th>Calculation method developed for....</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>P. Noakowski</td>
<td>Concrete members</td>
<td>“Verformungstheorie zur Ermittlung der Zwangbeanspruchung bei gleichzeitiger Lastbeanspruchung”, Januar 1986</td>
</tr>
</tbody>
</table>

7.1.2.2 Examination of the found calculation methods and verification of the main parameters of these methods

The full description and discussion of the found calculation methods are given in [1]. For example the formula for the calculation of the maximum crack width according to Prof. Hanswille is shown in formula (7-1):

$$w_{\text{max}} = \frac{f_{ct} \cdot d_s}{2 \cdot \tau_{\text{sm}} \cdot \rho_s} \cdot \left[ \frac{\sigma_s}{E_s} - \beta \cdot \frac{f_{ct}}{\rho_s \cdot E_s} \cdot \left( 1 + n_0 \cdot \rho_s \right) \cdot \left( 1 - \beta \right) \cdot \varepsilon_{\text{cr,eff}} \right]$$  

(7-1)

with

- $f_{ct}$ tensile strength of the concrete
- $d_s$ diameter of the reinforcement
- $\tau_{\text{sm}}$ mean bond stress
- $\rho_s$ ratio of reinforcement
- $\sigma_s$ mean stress of the reinforcement
- $E_s$ modulus of elasticity of the reinforcement
The formula is valid in the second phase that means in the state of stabilised crack formation and was derived from the principal equilibrium of the crack width calculation:

\[ w_{\text{max}} = 2 \cdot L_{\text{es}} \cdot (\varepsilon_{\text{sm}} - \varepsilon_{\text{cm}}) \]  

with

- \( L_{\text{es}} \): transmission length (see Fig. 7-1)
- \( \varepsilon_{\text{sm}} \): mean strain of the reinforcement
- \( \varepsilon_{\text{cm}} \): mean strain of the concrete

\[ (7-2) \]

**7.1.2.3 Extensive parameter study of the main parameter**

An extensive discussion of the main parameter of the calculation of the crack width are given [1]. The results of this parameter study can be summarised as follows:

- **in reference on the crack distance**
  - \( f_{\text{ctm}} \): the crack distance increase with higher values of the tensile strength
  - \( d_s \): the crack distance increase with higher diameters of the reinforcement
  - \( \rho_s \): the crack distance decrease with a higher reinforcement ratio
  - \( \tau_{\text{sm}} \): the crack distance decrease with higher mean bond stress

- **in reference on the mean value of the strain**
  - \( f_{\text{ctm}} \): the mean value of the strain decrease with higher values of the tensile strength
  - \( \rho_s \): the mean value of the strain decrease with a higher reinforcement ratio
  - \( n_0 \): the mean value of the strain decrease with a higher ratio of the young’s modulus
  - \( \beta \): the mean value of the strain increase with higher values of \( \beta \)
  - \( \varepsilon_{\text{cs,eff}} \): the mean value of the strain increase with a higher influence of shrinkage
7.1.2.4 Literature research to find tests on composite members and measured crack widths and comparison of the results of the calculation methods with these test results

The literature research leads to 21 test which could be used for a verification of the four calculation methods. Most of these test are taken from the background document to Eurocode 4 [2]. The full description of these test and the comparison of the measured values of the maximum crack width with the calculated values can be taken from [1]. Fig. 7-2 shows exemplary the comparison of four test with calculated values according to all of the investigated calculation methods.

Fig. 7-2: Comparison of the measured values of 4 tests with the calculated values of the maximum crack width

7.1.3 Conclusion according to the cracking of the slab and of the joints

A comparison of 21 test results found in literature with the calculated values of the crack width according four different calculation methods and the interpretation of this comparison was carried out in the frame of this research project. From that the following conclusion can be drawn:

- The result was that the calculation methods according to Prof. Hanswille and Dr. Maurer work very well.
- The ENV 1994-2 : 1997 gives simplified design rules to avoid non-permissible cracks, for example values of minimum reinforcement, a table of maximum bar diameter and maximum bar spacing are given.
- These simplified design rules are in a good accordance with the results evaluated with the calculation methods according to Prof. Hanswille.
- In conclusion the simplified design rules are applicable to composite bridges in small and medium spans made of normal or high strength steel and in-situ or prefabricated concrete slabs. Only the influence of creep and shrinkage of partially prefabricated concrete slabs could not clarified yet because of the lack of test results.
7.1.4 List of references according to cracking of the slab and of the joints


7.2 Concrete technology for prefabricated slabs and in situ concrete

The competitiveness of composite bridges depends on several circumstances such as site conditions, local costs of material and staff and the contractor’s experience. One major advantage compared to concrete bridges is that the steel girders can carry the weight of the formwork and the wet concrete, which means that there is no need for temporary structures. Another advantage are the savings in construction time, which saves some money for the contractor but even more for the road users; a fact that is usually neglected when considering advantages and disadvantages of bridge designs.

Different construction methods have been developed to improve either one or both of these advantages. Generally the methods can be divided up into three main groups:

- the full in-situ concrete method: the whole slab is poured in place
- the full precast concrete method: the whole slab is a precast unit
- a combination of both: a precast unit is used as formwork and supplemented by an in-situ concrete layer, so that a two parted slab is created

The above-mentioned methods are varied slightly regarding special properties in order to emphasize certain advantages, so that they can be divided up into subgroups.

In the following unit the construction methods are described and their advantages and disadvantages are mentioned.

7.2.1 The full in-situ concrete method

The steel girders are laid over the gap that has to be bridged. If it is a two span bridge and planed as a continuous beam the first girder, which is ~15% longer than the span, is laid from one end to the support. The next steel beam is laid from the other end of the bridge to the end of the first girder. The connection is carried out as hinge so that an articulated beam is created. The bending moment is supposed to be zero exactly in the location of the connection. There after the formwork for the concrete slab is erected on the steel girders. If statically necessary the horizontal rigidity is improved by integrating reinforcing braces. Now the shear connectors are welded onto the steel girder and the reinforcement is installed. Thereafter the concrete is poured. After the time of after treatment the surface is sealed. The formwork often makes a complicated static necessary that in complexity might surpass the one of the bridge itself.
On the other hand it enables the architect to design forms that are not possible to build with precast units. Except for the welding, which also can be done by the steel company, there is nothing to do on site that requires especially skilled workers so that the work can be executed by a large number of companies without a special preparation. At least for the customer this has the advantage, that a great competition between the executing companies on the marked exists and so drops the prices and makes it easy to find an adequate firm.

This construction method results in that the steel girders carry the whole dead load of the bridge’s superstructure and in addition the weight of the formwork during construction time. So extra horizontal reinforcing braces are almost essential and the steel girders must be dimensioned bigger. The total amount of steel is the highest of all methods. Due to the formwork the execution time is very high and man-hours are numerous. The whole shrinkage of the concrete affects the bridge. Fore the whole term of concreting it is essential that the environmental temperature is above 5°C what makes a construction in winter terms impossible in colder tempered zones such as Northern Europe.

However, since the required techniques are quite easy to execute it has become the most popular solution to set up a composite bridge. According to the consulting engineers “Müller und Winter, Wunstorf” almost 70% of all composite bridges in Germany are constructed that way.

Table 7-2 gives a brief summary of the advantages and disadvantages of this concreting method.
Table 7-2: Summary of the advantages and disadvantages of the full in-situ concrete method

<table>
<thead>
<tr>
<th>conventional method</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
</table>
|                     | • almost each design of the slab is possible  
|                     | • large number of capable firms cause competition and low prices  
|                     | • most popular method  | • no composite action for dead load  
|                     |                            | • long construction time  
|                     |                            | • large amount of man-hours  
|                     |                            | • large amount of steel is used  
|                     |                            | • impossible in winter terms with temperatures below 5°C  |

7.2.2 The concrete slab as full precast unit

The example given here describes the construction of a single span bridge as composite bridge with a precast concrete slab carried out in the north of Sweden to bridge the river Edslan (see Fig. 7-6).

Eleven precast units of 300x1800x7650 mm were laid on rolled steel joist with pre-mounted shear connectors. As it was not planed to fix in mortar to the joints afterwards, a dry construction method was developed. In order to receive a good fit between the elements one finished element was used as formwork for the next one during production, what made a numeration of the elements necessary. The connection between the elements was made using special dowels. Within one day the units were mounted, while the doweling shown in Fig. 7-5 fit exactly.

![Fig. 7-5: Doweling of the elements](image)

Afterwards the elements were prestressed by tightening four M24 bolts at each end of the girders. Thus the steel girders were used as tension bars. The construction was made during winter at temperatures about –10°C. So in order to concrete the connections between the slab and the girders the environmental temperature had to be increased by at least 15°C. To achieve this raise in temperature the bridge was covered by plastic film and beneath the bridge gas heatings were placed. As the temperature rose to 5°C the concrete was injected to the shear connectors through holes with a diameter of 20mm in the precast units.

While using precast units has the advantage of high quality concrete and gives the opportunity of surveillance it also requires a high accuracy producing the units. Obviously very exact working is required producing the precast units and mounting the shear connectors so that later when positioning the elements no problems occur due to shear connectors that get into conflict with the reinforcement of the slab.
As said before one advantage of composite bridges is the possible short construction time. The most favourable way of constructing composite bridges in order to save time is the above mentioned one. In Sweden an investigation was carried out in order to determine the costs that might be saved by a shorter construction phase. Based on the time-schedule shown in Table 7-3 for three alternative concepts for a three span road bridge the saveable costs have been calculated.

**Table 7-3: Comparison of the time-schedule for three alternative concepts**

---

### a) in place concreted composite bridge

<table>
<thead>
<tr>
<th>weeks</th>
<th>1</th>
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### b) precasted units with wet joints

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### c) precast units with dry joints

<table>
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<th>Weeks</th>
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The table shows a possible time saving of about ten weeks not yet considered the possible disturbance of cold weather when concreting is impossible. The possible savings where calculated referring to a real carried out project. On the assumption that the time saving of each road user, using the bridge, is 5 minutes, that daily 2000 cars pass the bridge and that one hour values approximately 10 EURO for the road user, the sum of savings after 10 weeks is 117000 EURO. On the other hand, the problem is that no customer considers this fact what is quite understandable as nobody who is concerned with the decision how to build the bridge has to pay these expenses. So this argument might be a political question.

The following table gives the advantages and disadvantages in short:

**Table 7-4: Summary of the advantages and disadvantages of the concrete method using the concrete slab as full precast unit**

<table>
<thead>
<tr>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>high and equal concrete quality</td>
<td>high technical know-how is necessary</td>
</tr>
<tr>
<td>shortest construction time of all methods</td>
<td>this includes high costs</td>
</tr>
<tr>
<td>best solution for the road users</td>
<td>limited design possibilities</td>
</tr>
<tr>
<td>reduces man-hours due to work in fabrics</td>
<td>steel girders carry the whole dead load</td>
</tr>
<tr>
<td>set-up is possible in freezing periods</td>
<td>only very few precast unit factories are able to the job</td>
</tr>
<tr>
<td>reduced forces from shrinkage as the elements get to the site older</td>
<td>only few companies have experience with that method</td>
</tr>
</tbody>
</table>

### 7.2.3 Precast units supplemented with in-situ concrete

This method has to be divided up into two major groups. It can be carried out so, that the precast unit is just a permanent formwork. It is also possible to give the precast unit a static function in longitudinal direction. This method was improved by the SSF company in Germany.

The basic idea using these precast units is to simplify the formwork. Prefabricated elements are laid as formwork onto the steel girders that are mounted equal to the other construction methods. Next the reinforcement is installed and the concrete is poured from the middle of the bridge to the ends. The direction of concreting is quite important in order to avoid cracking. Starting in the middle lets deflect the bridge first so that when concreting the locations with negative moments and deflections, they already have their final forms. So the just poured concrete does not get subject to tension. To ensure the transmission of shear stress, the shear connectors are so long that they pass through the precast unit and enter into the in-situ concrete slab. Also shear garlands are mounted that connect the precast unit and the in-situ concrete. This sequence of construction results in that the whole dead load of the bridge is carried in longitudinal direction by the steel girders. As already mentioned other ways of building have been developed in order to give the prefabricated elements also in longitudinal direction a static function.
Fig. 7-7: A composite bridge with precast units as formwork

One possibility is to fix in mortar to the transverse joints before pouring the in-situ concrete. This however requires an extra work step and extra costs. The advantage is that the main dead load of the bridge, the load of the reinforcement and the in-situ concrete, is carried by the partly composite girder. The precast units act as compression boom and give the construction a better horizontal stiffness while concreting. Horizontal braces are not a must. In every case the amount of construction steel required is reduced.

Table 7-5 sums up the advantages and disadvantages of the described method.

Table 7-5: Summary of the advantages and disadvantages of the concrete method using precast elements as formwork

<table>
<thead>
<tr>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>• no additional formwork is needed</td>
<td>• more complicate technique that is not executable for every company</td>
</tr>
<tr>
<td>• short construction time</td>
<td>• restricted design possibilities</td>
</tr>
<tr>
<td>• possibility of reducing effects from shrinkage</td>
<td>• long transportation ways as only a few factories can produce the elements</td>
</tr>
<tr>
<td>• especially if composite action is activated first the amount of steel that is necessary is reduced as the dead load is carried by the composite girder</td>
<td></td>
</tr>
</tbody>
</table>

The SSF engineering society improved this method again. They developed a type of composite girder that can be almost completely prefabricated in the factory, see Fig. 7-8. These have an upper boom which acts as an economical cross section under compression, as a shell element for the bridge floor slab and as a horizontal stabilisation. Constructing and tilting braces are not longer needed for concreting the in-situ concrete plate. The amount of steel required is once again considerably reduced, since this is determined by the amount needed in construction and not the amount contained in the bridge structure. Simple connection rod links in the support area allow the girders to be coupled in the construction phase, in order to achieve a continuous effect. After the installation and reinforcing stages, the in-situ concrete can be pored continuously and therefore more economically. The already executed projects include large bridges over valleys as well as double-span bridges over motor ways and show that also this very special method which is only
supported by a few precast unit factories can be favoured by certain customers. However, it remains a very special solution and includes a very complicate transportation as the girders can not be piled up. Due to the fact that only a few factories support the method, long transportation ways are almost sure, this might be the most serious problem for this method to become more popular.

![Image](image.jpg)

**Fig. 7-8:** A VFT® girder by SSF

The advantages and disadvantages using this invention are given in Table 7-6.

**Table 7-6: Summary of the advantages and disadvantages using the above mentioned invention**

<table>
<thead>
<tr>
<th>VFT© girders by SSF</th>
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<tbody>
<tr>
<td><strong>Advantages</strong></td>
<td><strong>Disadvantages</strong></td>
</tr>
<tr>
<td>• The high quality of the finished structure keeps maintenance costs low, due to the strong design with easily accessible steel girders and a monolithic in-situ concrete slab</td>
<td>• The tall girder-slab composition is not pile able and makes huge trucks necessary for transportation.</td>
</tr>
<tr>
<td>• Extensive prefabrication of the girders reduces the amount of work on the building site, so that construction deployment is very economical with efficient construction time</td>
<td>• High costs due to low competition</td>
</tr>
<tr>
<td>• The concrete flanges that form the upper boom make buckling braces in the steel girders unnecessary, considerably reduce the amount of steel used in construction, since the composite material is mounted in advance, and increase the tilt resistance, i.e. no assembly braces are needed.</td>
<td>• The rare available precast unit fabrics make far transportation necessary.</td>
</tr>
</tbody>
</table>

**7.2.4 Conclusion according to the concrete technology**

Regarding all the mentioned arguments it is impossible to favour one of the above mentioned construction methods. However it is obvious that the main disadvantage of the more developed
methods, which all include the use of precast units, is the required know-how. Another disadvantage is that only a few precast unit fabrics are able to produce the required elements. If it would be possible to raise political interest in the costs caused by traffic disturbance the fastest method would always be the most favourable. Only the dry construction method as tested in the pilot project in Sweden allows construction during freezing period. Since these arguments only apply to very few bridges or to anyone, the use of pre-concreted elements as formwork seems to be the most reasonable method that should be developed and over all promoted. Consequently it is essential to develop a method that enables more companies to carry out the work, like it is the aim of the European project “Composite Bridge Design for Small and Medium Spans”. The very special solution of SSF’s VFT© is only reasonable for very few opportunities since transportation costs are too high.

7.3 Creep and shrinkage effects

As mentioned in chapter 7.1.4 one of the possible construction methods for composite bridges is to use precast units as formwork and compression boom. When the pre-concreted elements are given a static function, it is necessary to consider their different material properties. Mostly the precast units are of a higher concrete quality and are some weeks older than the in-situ concrete when the composite girder is charged. Also creep and shrinkage effects, as already said in chapter one, differ according to age and concrete quality. Thus it is of interest to examine the long-term behaviour of such composite girders.

7.3.1 Posing the problem

The construction method examined is the combination of pre-concreted elements with supplemented in-situ concrete. Before concreting the upper slab, mortar is fixed in the joints between the precast units.

The investigation focuses on the maximum of deflection and the range of cracked concrete in the state of serviceability. The ultimate limit state is not of interest because long term effects do not affect it.

The Eurocode does not provide a calculation method for the above described problem. However it gives sufficient basic information on material behaviour so that a particular calculation can be developed. Due to the leak of experience with three parted composite girders all generally used simplifications for two parted composite girders must first be queried.

In particular the following facts must be doubted:

- the creep multipliers $\Psi_l$ from EC4-2 [3], because the slab is two parted
- the assumption that 15% of the span crack and only in the uncracked range shrinkage and creep have to be considered, because the construction progress leads to two slab that crack under different conditions
### 7.3.2 Material properties

Shrinkage, creep and tension stiffening are the most significant properties of the used materials and are shortly explained in the following chapter. For further information the study of the Betonkalender 2000 is recommended. It gives a good background knowledge.

#### 7.3.2.1 Shrinkage

The property of concrete to reduce its volume in the course of time without stress is called \textit{shrinkage}. The Betonkalender [1] shows the most recent calculation method for shrinkage. According to a study by Müller / Hilsdorf based on a large data base the following formulas were set up:

Shrinkage consists in two parts:

\[ \varepsilon_{cs}(t, t_s) := \varepsilon_{cas}(t) + \varepsilon_{cds}(t, t_s) \]  
(7-3)

The first part is the autogenous shrinkage:

\[ \varepsilon_{cas}(t) := \varepsilon_{cas0}(f_{cm}) \cdot \beta_{as}(t) \]  
(7-4)

\[ \varepsilon_{csaso}(f_{cm}) := - \alpha_{as} \left( \frac{f_{cm}}{f_{cm0}} \right)^{2.5} \cdot 10^{-6} \]  
(7-5)

\[ \beta_{as}(t) := 1 - \exp \left( -0.2 \cdot \left( \frac{t}{t_1} \right)^{0.5} \right) \]  
(7-6)

The second is the dry shrinkage:

\[ \varepsilon_{cds}(t, t_s) := \varepsilon_{cdso}(f_{cm}) \cdot \beta_{RH}(RH) \cdot \beta_{ds}(t - t_s) \]  
(7-7)

\[ \varepsilon_{ds0}(f_{cm}) := \left( 220 + 110 \cdot \alpha_{ds1} \right) \cdot \exp \left( -\alpha_{ds2} \frac{f_{cm}}{f_{cm0}} \right) \cdot 10^{-6} \]  
(7-8)

\[ \beta_{RH}(RH) := -1.55 \left[ 1 - \left( \frac{RH}{RH_0} \right)^3 \right] \]  
(7-9)

for

\[ RH < 99 \% \cdot \beta_{s1} \]

\[ \beta_{RH}(RH) := 0.25 \]

for

\[ RH \geq 99 \% \cdot \beta_{s1} \]

with:
\[ t_h := t - t_s \]

\[
\beta_{ds(t_h)} := \frac{\left( \frac{t_h}{t_1} \right)}{\left( \frac{350}{t_1} \right)^2 \left( \frac{h}{h_0} \right)^2 + \left( \frac{t_h}{t_1} \right)} \]

\[ \beta_{s1} := \left( \frac{f_{cm0}}{f_{cm}} \right)^{0.1} \]

Variables as shown:

<table>
<thead>
<tr>
<th></th>
<th>( \alpha_{as} )</th>
<th>( \alpha_{ds1} )</th>
<th>( \alpha_{ds2} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Z 35 L</td>
<td>800</td>
<td>3</td>
<td>0.13</td>
</tr>
<tr>
<td>Z 35 F + Z 45 L</td>
<td>700</td>
<td>4</td>
<td>0.11</td>
</tr>
<tr>
<td>Z 45 F + Z 55</td>
<td>600</td>
<td>6</td>
<td>0.12</td>
</tr>
</tbody>
</table>

\[ f_{cm} := f_{ck} + 8 \frac{N}{\text{mm}^2} \]

average concrete compression strength

\[ f_{cm0} := 10^0 \frac{N}{\text{mm}^2} \]

t time

\[ t_1 := 1 \cdot \text{d} \]

RH humidity

\[ \text{RH}_0 := 100 \% \]

h effective construction unit thickness

\[ h_0 := 100 \text{ mm} \]

\( \beta_{s1} \) coefficient that considers internal drying

The given calculation method results in a variation-coefficient of 33% and is also suitable for high strength concrete.

### 7.3.2.2 Creep

Concrete shows a viscous-elastic-behaviour. Permanent charge results in an immediate deformation and a deformation that increases in the course of time. This property is called \textit{creep}. 
The total strain of concrete is the sum of elastic deformation, creep and shrinkage. The results of the following formula are shown in Fig. 7-9.

\[ \varepsilon_c(t) = \varepsilon_{cel}(t) + \varepsilon_{cc}(t, t_0) + \varepsilon_{cs}(t, t_s) \]  

(7–10)

Fig. 7-9: Deformation components of concrete

An important fact is that for

\[ |\sigma_c| \leq 0.45 f_{ck}, \]

the relation between the increase of deformation due to creep and the stress is linear.

The formula

\[ \varepsilon_{cel}(t) + \varepsilon_{cc}(t, t_0) \cdot J(t, t_0) \cdot \sigma_c \]  

(7–11)

gives the sum of elastic- and creep-deformation as product of stress and the creep-function \( J(t, t_0) \) which is

\[ J(t, t_0) = \left( \frac{1}{E(t_0)} + \frac{\phi(t, t_0)}{E(28 \text{ d})} \right) \]  

(7–12).

Model-code 90 gives the advice to use the following equation:

\[ J(t, t_0) = \left( \frac{1}{E_{cm}(t_0)} + \frac{\phi(t, t_0)}{1.05 \cdot E_{cm}(28 \text{ d})} \right) \]  

(7–13)

where

\[ \phi(t, t_0) \]

is the creep number and material specific.

*The creep number:*
Many attempts to develop a mathematical function that describes this creep-number were made. The most recent one was developed by Müller/Hilsdorf.

The presented method is an enhancement of the method in EC2.

\[
\phi(t, t_0) := \phi_0 \cdot \beta(t, t_0)
\]

(7–14)

The first factor is the basic creep number and the second is a function that describes the development by time.

\[
\phi_0 := \phi_{RH} \beta(f_{cm}) \beta(t_0)
\]

(7–15)

with:

\[
\phi_{RH} := \left[ 1 + \frac{1 - \frac{RH}{RH_0}}{3 \left( \frac{h}{h_0} \right)^{0.1}} \right]^{\alpha_1 \cdot \alpha_2}
\]

\[
\beta(f_{cm}) := \frac{5.3}{f_{cm} / f_{cm0}}
\]

\[
\beta(t_0) := \frac{1}{0.1 + \left( \frac{t_0}{t_1} \right)^{0.2}}
\]

\[
\alpha_1 := \left( \frac{f_{cm0}}{f_{cm}} \right)^{0.7}
\]

and

\[
\alpha_2 := \left( \frac{f_{cm0}}{f_{cm}} \right)^{0.2}
\]

and with the variables

\[
f_{cm0} := \frac{10^N}{\text{mm}^2}
\]

\[
RH_0 := 100\%
\]

\[
h_0 := 100\text{mm}
\]

the other variables are the same as for the shrinkage function.
The time-function is:
\[ \beta_c(t, t_0) := \left[ \frac{(t - t_0)}{t_1} \right]^{0.3} \left[ \frac{(t - t_0)}{t_1} \right] \]

with:
\[ \beta_H := 150 \left[ 1 + \left( \frac{1.2 \cdot RH}{RH_0} \right)^{18} \right] \frac{h}{h_0} + 250 \cdot \alpha \cdot 3 \leq 1500 \cdot \alpha \cdot 3 \]
\[ \alpha := \left[ 3.5 \cdot \left( \frac{f_{cm0}}{f_{cm}} \right) \right]^{0.5} \]

The so calculated values have an variation-coefficient of 32.8%.

The elastic modulus:
For the creep-function it is also necessary to know the development of the elastic modulus by time. MC90 provides the following formulas.

The compression strength:
\[ f_{cm}(t) := \beta_{cc}(t) \cdot f_{cm} \]

with
\[ \beta_{cc}(t) := \exp \left[ s \cdot \left( 1 - \left( \frac{28}{t} \right)^{1/2} \right) \right] \]

is basis for the elastic modulus:
\[ E_c(t) := \beta_E(t) \cdot E_{c28} \]

with:
\[ \beta_E(t) := (\beta_{cc}(t))^{0.5} \]
\[ E_{c28} := \alpha \cdot E \cdot E_{c0} \cdot \left( \frac{f_{cm}}{f_{cm0}} \right)^{1/3} \]
\[ E_{c0} = 9500 \frac{MN}{m^2} \]
The result is:

$$E_{cm}(t) = 9500 \left( f_{cm28d} \right)^{\frac{1}{3}} \left[ s \cdot \left( 1 - \left( \frac{28}{t} \right)^{0.5} \right) \right]$$

(7–19)

$\alpha_e$ is a multiplier to consider the type of aggregate. As this is often not known to the designing engineer it is not considered in the formula.

The variable $s$ is taken from Table 7-7.

**Table 7-7: Variable $s$**

<table>
<thead>
<tr>
<th></th>
<th>$Z\ 35\ L$</th>
<th>$Z\ 35\ F\ and\ Z\ 45\ L$</th>
<th>$Z\ 45\ F\ and\ Z\ 55$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S$</td>
<td>0.38</td>
<td>0.25</td>
<td>0.2</td>
</tr>
</tbody>
</table>

**The tensile strength:**

The tensile strength is derived from the compression strength as follows:

$$f_{cm} := f_{ck} + 8$$

(7–20)

$$f_{ctkom} := \frac{N}{mm^2}$$

(7–21)

$$f_{cmo} := 10 \frac{N}{mm^2}$$

(7–22)

the tensile strength results in:

$$f_{ctm} := f_{ctkom} \left( \frac{f_{ck}}{f_{cko}} \right)^{\frac{2}{3}}$$

(7–23)

For

$$f_{cm} > 80 \frac{N}{mm^2}$$

another formula is used:

$$f_{ctm} := f_{ctmo} \ln \left( 1 + \frac{f_{cm}}{f_{cm0}} \right)$$

(7–24)

with


\[ f_{ctm0} = 2.12 \frac{N}{mm^2} \]

\[ f_{cm0} = 10 \frac{N}{mm^2} \]

These formulas are used in the program for crack calculation.

**Superposition of creep:**

As remarked above, for

\[ |\sigma_c| \leq 0.45 f_{ck}, \]

the relation between the increase of deformation due to creep and the stress is linear. Regarding permanent loads this is no restriction since EC2 demands this condition anyway. This gives the possibility of considering changes in stress and their effects on strain due to creep.

The superposition-principle of Boltzmann allows the following transformation. Shen writes in \[5\] how the following formulas were developed.

\[ \varepsilon_{cel} (t_0) + \varepsilon_{cc} (t, t_0) = J(t, t_0) \cdot \sigma_c \]

is transformed to:

\[ \varepsilon_{cel} (t_0) + \varepsilon_{cc} (t, t_0) = J(t, t_0) \cdot \sigma_c + \int_{t_0}^{t} J(t, \tau) \frac{\delta \sigma(\tau)}{\delta \tau} d\tau \]

Written as finite sum:

\[ \varepsilon_{cel} (t_0) + \varepsilon_{cc} (t, t_0) = \sum_{i=0}^{n-1} \sigma(ta_i) \cdot \left( J(t, t_i) - J(t, t_{i+1}) \right) + \sigma(ta_{n-1}) \cdot J(t, t_n) \]

\[ \sigma(ta_i) \] is the average tension in the interval \( t_i < ta_i < t_{i+1} \).

In Eurocode 2 are made the following assumptions:

- creep and shrinkage are independent
- existence of a linear relation between strain due to creep and the provoking stress
- neglecting influences from irregular distributed temperature and humidity
- validity of the superposition system

**7.3.2.3 Tension stiffening**

In the cracks of cracked reinforced concrete units the tensile stress is carried only by the reinforcement. Between the cracks the composite action transmit the forces back to the concrete.
Regarding the whole unit the concrete participates carrying the load and increases the tensile stiffness. This effect is called tension stiffening.

EC4-2 [3] says in annex L how tension stiffening has to be considered in slabs of composite girders subjected to tension.

Fig. 7-10 shows normal force and mean strain $\varepsilon_{sm}$ for reinforced concrete tension members.

$$\varepsilon_{sm} = \frac{N_s}{E_s A_s} - \frac{0.2 f_{ctm}}{A_s E_s}$$

(7–25)

Regarding a three parted cross-section like the one resulting from the construction method that is investigated, the precast unit must also be considered. Since the reinforcement in the precast units is not continuous from one element to the next it is neglected. This results in that no tension stiffening for the precast unit has to be considered.

From the construction method and the cross-section itself result different possibilities for the state of cracking of the composite girder. In general it has to be considered that while the in-situ concrete slab is in state (a), (b) or (c) as shown in Fig. 7-10, the precast unit might be cracked or not. Consequential not only three states exist but six. Consequences are shown in chapter 7.3.3.3.

### 7.3.3 The program ‘CASE’ (Creep and Shrinkage Effects)

As mentioned in chapter 7.3.1 the deflection and the range of cracked concrete has to be calculated. The high complexity of the problem led to the decision to develop a computer-program to do the calculation.
The program is written in Visual Basic 6.0® as plug-in for Microsoft Excel 9.0® part of Microsoft Office2000®. This provides the possibility to use the calculated values and diagrams in other applications.
7.3.3.1 Features of ‘CASE’

The software calculates the long-term behaviour of a composite girder. The cross-section may consist in:
- a construction steel cross-section
- a precast unit as upper compression boom
- an in-situ concrete slab as upper compression boom
- two layers of reinforcement in the in-situ concrete

The results are shown in:
- a deflection curve diagram
- a bend curve diagram
- a crack range diagram
- a table of internal forces

A single-span and a symmetric two-span girder can be calculated. Prestressing by support jacking can also be considered.

The program also provides an analysis tool and export possibilities.

7.3.3.2 Facts that are considered by ‘CASE’

The following facts and material properties are considered:
1. creep according to the calculation method by Müller/Hilsdorf
2. shrinkage according to the calculation method by Müller/Hilsdorf
3. non-linear material behaviour due to cracking
4. tension stiffening
5. construction methods described under 7.2.3

7.3.3.3 Description of the calculation

The girder is divided up into 100 cross-sections (see Fig. 7-11). For each cross-section the curvature due to creep, shrinkage, cracking and the acting bending moment are calculated. The curvature is integrated twice to get the deflection. The term between 0 days and 10000 days is divided up into 70 time steps. A higher resolution did not show any effects to the results. For each step the internal forces and curvatures are calculated. In an iteration process the end moment of a two-span bridge is determined.
In the process of construction exist three different static systems, namely:

1. steel girder charged with the dead load of itself and the precast units

2. composite girder of precast unit and construction steel charged with the dead load of the in-situ concrete slab

3. complete composite girder as shown in figure 13 charged with the extension load

The systems 1) and 2) are described with the conventional methods and are not explained here. System 3) cannot be calculated using the formulas given in EC 4-2 due to the two parted slab. The function for curvature is determined differently. This static system has to be used after composite action between the two concrete slabs starts.

**7.3.3.4 Function of curvature for the three parted cross-section**

The following assumptions equal to step 1 and 2 are made:

- Creep and shrinkage are independent
- Elastic and linear material behaviour as long as no cracks occur
- Bernoulli hypothesis
- Full composite action between steel and concrete as well as between in-situ concrete and precast concrete

The assumption of full composite action between the two concrete was confirmed by the Institute of Solid Construction. They carried out tests and reported that a girder consisting of two concrete subject to a bending moment shows a monolithic behaviour.
In [8] Mainz only examined joints that are subject to a normal force and at the same time subject to shear stress. These results cannot be considered in this case.

In order to get a function for curvature, the internal forces $N_{sta}$, $M_{sta}$, $N_{ba}$, $M_{ba}$, $N_{bn}$, $M_{bn}$, $N_{st1}$, $N_{st2}$ are determined in an equation system. The equation system consists in 8 equations:

\[ N_{sta} + N_{ba} + N_{bn} + N_{st1} + N_{st2} = 0 \]
\[ M_{sta} + M_{ba} - N_{ba} \cdot y_{banet} + M_{bn} - N_{bn} \cdot y_{bn} - N_{st1} \cdot y_{st1} - N_{st2} \cdot y_{st2} = M_{ges} \]
\[ \kappa_{ba} = \Delta \kappa_{sta} \]
\[ \kappa_{bn} = \Delta \kappa_{sta} \]
\[ \varepsilon_{st2} = -\Delta \kappa_{sta} \cdot y_{st2} + \Delta \varepsilon_{sta} \]
\[ \varepsilon_{st1} = -\Delta \kappa_{sta} \cdot y_{st1} + \Delta \varepsilon_{sta} \]
\[ \varepsilon_{ba} = -\Delta \kappa_{sta} \cdot y_{banet} + \Delta \varepsilon_{sta} \]
\[ \varepsilon_{bn} = -\Delta \kappa_{sta} \cdot y_{bn} + \Delta \varepsilon_{sta} \]

Due to the joint construction between the pre-concreted elements it is necessary to reduce the thickness of the elements by the height $h_f$ so that $y_{banet}$ etc. must be used in the formulas.

The strains $\varepsilon_{ba}$ and $\varepsilon_{bn}$ are calculated according to chapter 7.3.2.2 and equal i.e. for $\varepsilon_{ba}$:

\[ \varepsilon_{ba}(t) = \frac{N_{ba}(t) \cdot J_{ba}(t,t_{n-1})}{A_{banet}} + \sum_{i=0}^{n-2} \frac{N_{ba}(t_i) + N_{ba}(t_{i+1})}{2 \cdot A_{banet}} \cdot (J_{ba}(t_i) - J(t_i,t_{i+1})) + \varepsilon_{sba}(t) \]

The $\Delta \Delta$ in front of a strain variable means the difference between the momentary strain and the strain from construction step 1) and 2).

It does not matter if a one or a two span bridge is calculated, the system is statically indeterminate due to composite action. So the calculated period is divided up logarithmically in order to get a good approximation while calculating only the necessary.

By adding the equation:

\[ \frac{N_{bn}(t)}{A_{bn}} - \left| \frac{M_{bn}(t)}{l_{bn}} \cdot \frac{h_{bn}}{2} \right| = f_{ctmbn}(t) \]

it is possible to get the Moment $M_{banet\,cr}$. It is the bending moment at which the in-situ concrete slab starts cracking. The same is done for the precasted concrete.

\[ \frac{N_{ba}(t)}{A_{banet}} - \left| \frac{M_{ba}(t)}{l_{banet}} \cdot \frac{h_{banet}}{2} \right| = f_{ctmba}(t) \]
Considering the three states of cracking (chapter 7.3.2.3) six different functions for curvatures are created. By controlling the stress in the outer fibre or the force \( N_s \) of the slabs it is possible to decide when which curvature-function is valid. The resulting equation systems are given in annex A according to [11].

The tensile strength \( f_{t,\text{max}}(t) \) of a cross-section is set to zero if the corresponding slab has cracked!

If the different curvature-functions are named \( \chi_{(pc),(is)} \) with the indices “pc” for the state of cracking of the precast concrete slab and ‘is’ for the state of cracking of the in-situ-concrete slab the following six curvatures exist:

\[ \chi_{11}, \chi_{12}, \chi_{22}, \chi_{23}, \chi_{13}, \chi_{21} \]

The indices 1,2,3 correspond to the states of cracking (a), (b) and (c) according to (7.3.2.3).

If the bending moments are named according to the states of cracking they divide, the following names are possible:

\( M_{1c2} \) means the precast concrete remains uncracked while the in-situ concrete changes from state (a) to (b) according to 7.3.2.3.

\( M_{c21} \) means the precast concrete cracks while the in-situ concrete remains uncracked.

According this scheme also exist:

\( M_{2c2}, M_{c22}, M_{1c3}, M_{c23}, M_{2c3}, M_{c22} \)

The calculation of the curvatures and bending moments \( M_{iz}, \chi_{iz} \) is given in annex A according to [11].

Fig. 7-12 shows the state of cracking and the corresponding curvature according to the acting bending moment \( M \).

\[ \begin{align*}
\kappa_{11} & \quad n \quad \kappa_{21} \\
\kappa_{22} & \quad n \quad \kappa_{23} \\
\kappa_{12} & \quad j \quad \kappa_{13} \\
\kappa_{23} & \quad j \quad \kappa_{23}
\end{align*} \]

\[ \begin{align*}
M_{1c2} & \quad n \quad \kappa_{11} \\
M_{c21} & \quad j \quad \kappa_{12} \\
M_{2c2} & \quad j \quad \kappa_{23} \\
M_{c22} & \quad j \quad \kappa_{23}
\end{align*} \]

Fig. 7-12: Scheme of acting moment and state of cracking

The values \( M_{iz} \) are calculated in the program for every cross-section for every time-step.
For further information the source code of the program is contained in annex C according to [11]. It is commentated and the used variables are named according to their static function as far as possible.

### 7.3.3.5 Program manual

First of all it has to be mentioned that the diagrams show no units. The convention for this program is that all lengths are in meters and all forces in meganewton. The therefrom derived units are in the same units; i.e. stress in MN/m².

**Installation of the program:**

The Program is written as Microsoft Excel 9.0® plug-in. It consequently requires Microsoft Excel 9.0®, part of Microsoft Office2000®. To install it the file ‘CompositeBridgeV2.2.xla’ has to be copied to the plug-in directory on the hard disk drive. Usually it is C:\WINDOWS\Anwendungsdaten\Microsoft\AddIns (for the German version).

The plug-in is installed by the choosing ‘Add-in manager’ from the menu ‘Extras’ and activating the checkbox for ‘CompositeBridgeV2.2’, see Fig. 7-13.

Alternatively the program can be installed by double-clicking the file ‘CompositeBridgeV2.2.xla’ on the CD and following the online instructions.

![Fig. 7-13: Installed by the choosing ‘Add-in manager’](image)

**The program menu ‘CASE’:**

After the installation ‘CASE’ is added to the command-bar. The following figure shows the menu.
Fig. 7-14: Command-bar after installation of ‘CASE’
The menu consists in 14 objects. They are detailed explained in [11] while calculating an example.

It has to be remarked that for no input an error check is made. This means that entering wrong values results in an error warning and the interruption of the running process or simply in wrong results. The entering numbers must compromise with the system settings (i.e. 1.5 or 1.5). Therefore this program should only used by experts which are able to identify ‘wrong results’!

**Preparing the calculation:**

From the menu ‘CASE’ the position ‘new workbook’ is chosen. It asks for a name for the created file and gives the opportunity of choosing a directory where the file is saved. The following pictures show the process:

![Fig. 7-15: Directory where the file is saved](image)

The name of the file should refer to the calculated problem, because the filename is added to the diagram-titles.

**Data input:**

Exemplary the input of the cross-section data is described. The description of the input of the in-situ concrete data, the precast concrete data and the bridge design and construction data could be taken from [11].

From the menu ‘CASE’ the position ‘input cross section data’ is chosen and the appearing form is fillet in according to the following picture.
It has to be remarked, that the effective width simplifying assume for the whole girder. The value $h_f$ is the thickness of the joint construction that serves to hold the mortar while it is fixed in. After the hydration a gap of some millimetres remains in this region between the precast elements. The dimension of the precast unit is reduced by this value.

The asked lever arms always refer to the main-axis of the construction steel.

Starting the calculation:

From the menu ‘CASE’ the position ‘start calculation’ is chosen.

As seen in the figure above, while calculating the program shows the determined deflection curves.
In the status bar, in the down left corner (see figure right-hand side), four values are displayed:
- time step
- day
- end moment
- error of iteration process

The error of the iteration process is given in radiant. It is the remaining rotation of the girder-end and should be close to zero. If the value remains too high the calculation is aborted with an error alert.

**Results:**

The calculation creates three more sheets, see Fig. 7-18 to Fig. 7-20:

1. The deflection curve diagram
2. The bend curve diagram
3. The crack diagram

The calculation table is fillet with the internal forces of all cross-sections and for all calculated times.

![Deflection Curve Diagram](image)

**Fig. 7-18:** The deflection curve diagram
The crack diagram shows for each time-step a double coloured line. The upper line stands for the in-situ concrete and can have three colours: green = uncracked, yellow = state of single cracking, red = state of stabilized cracking. The down line can be green = uncracked or red = cracked. Setting the cursor on the white line between the time steps displays the location in the command line. Placing the cursor right of the coloured lines shows the day of the time-step in the command line.
Fig. 7-21: The calculation table

The window is split while the table ‘calculation’ is displayed. The first column contains the description of the rows. Column 2 contains for each time step in row 1 the number of the time-step, in row 2 the day, in row 3 the end moment and in row 6 the load [MN/m²]. Row 1 contains the distance from the left end of the bridge. The down-left part of the split window contains for each time-step 15 rows with the internal forces of the 100 cross-sections. It can be scrolled horizontal and vertically to view the cross-sections at the different time steps while always controlling the location and time.

In order to save the diagrams as ‘*.gif’ file (compuserve bitmap), from the menu ‘CASE’ the positions ‘save ~curve’ can be chosen. This provides the possibility to use the diagrams in other documents.

Modification of input data:

Therefore the program provides the possibility to open a new workbook with the old input data. Only the values that are modified have to be changed. After the calculation two independent workbooks exist which enables comparison of the results. This process can be repeated.

From the menu ‘CASE’ the position ‘new workbook with old input data’ is chosen. It asks for a name for the created file and gives the opportunity of choosing a directory where the file is saved.

Saving calculations:

The normal ‘save file’ options can be used. The saved file only contains the calculated data and input data. It can be used for every purpose. Recalculating however requires the plug-in ‘CASE’.

Opening existing calculations:

The normal ‘open file’ options can be used. Calculation can only be done if the plug-in ‘CASE’ is installed.

Analysis:
The program provides an analyse-tool to examine the stress distribution in the composite girder, see Fig. 7-22. It was developed for the investigations described in 7.3.6. Among other things it displays a diagram with the ratio-curves of the internal-forces in the cross-section. This diagram shows the bending moment, caused by the concrete and the steel, in % of the hole bending moment or, in case of shrinkage, just the bending moments, see Fig. 7-23.

![Analyse-tool of the program ‘CASE’](image1)

**Fig. 7-22:** Analyse-tool of the program ‘CASE’

![Analysis diagram with the ratio-curves of the internal-forces in the cross-section](image2)

**Fig. 7-23:** Analysis diagram with the ratio-curves of the internal-forces in the cross-section

### 7.3.3.6 Verification of the program

Due to the difficulty to find a Software, that does a similar calculation, hand-formula are used to verify the results of the program. For that the long-term behaviour of the precast concrete is neglected or regarded to be the same as the behaviour of the in-situ concrete.

So the verification had to be done stepwise. The used algorithms were controlled separated from each other.
As described above, the program calculates the deflection by integrating the curvature twice. In step 1) the curvature is simply the one of a steel girder. So the result from step one can be calculated manually.

Manual calculation of the maximum of deflection of a single span girder:

\[ w_{\text{max}} = \frac{ql^4}{76.8EI} \]

for

\[ q=0.02313560; \ l=20; \ E=210000; \ I=0.006447 \]

results in:

\[ w_{\text{max}}=3.56010 \times 10^{-2} \]

In comparison the program ‘CASE’ calculates:

\[ w_{\text{max}}=3.56010 \times 10^{-2} \]

For a two-span bridge the end moment is determined in an iteration. The end moment can be calculated manually:

\[ M=-\frac{ql^2}{8} \]

For the example above this results in \( M=-1.15678 \)

‘CASE’ calculates \( M=-1.15678 \)

The resulting maximum of deflection calculated manually:

\[ w_{\text{max}} = \frac{ql^4}{184.6EI} = 1.481126 \]

‘CASE’ calculates:

\[ w_{\text{max}}=1.481126 \]

From these results it can be derived that the integration and iteration modules work correctly. The same modules are used for all three steps.

Now the functions \( \chi_{xx}(M) \) were verified. This was done recalculating the elastic modulus from a calculated internal force and the corresponding curvature. If for every member of the cross-section the elastic modulus is correct recalculated it can be derived that the functions (see annex A) work correctly. For manual control the following formulas were used:

\[ \frac{N_{bn}}{\left(\kappa y_{bn} A_{bn}\right)} = E_{bn} \]

\[ \frac{N_{ba}}{\left(\kappa y_{banet} A_{ba}\right)} = E_{ba} \]
\[
\frac{M_{ba}}{\kappa I_{ba}} = E_{ba}
\]
\[
\frac{M_{sta}}{\kappa I_{sta}} = E_{sta}
\]
\[
\frac{N_{st1}}{A_{st1}(\kappa \cdot y_{st1})} = E_{st1}
\]
\[
\frac{N_{st2}}{A_{st2}(\kappa \cdot y_{st2})} = E_{st2}
\]

This control was done for all curvature functions. All functions work correctly.

Naturally this can only be done for \( t=0 \) the long term behaviour cannot be controlled like this due to the changing elastic modulus. To verify this, the creep multipliers were calculated as explained in 7.3.6. From this it could be concluded that also the long-term behaviour was considered correctly. Further more in [6] the same principles are used.

Cracking was controlled by checking the stress in the outer fibers of the calculated cross-sections.

The crack moments were checked by calculating the stress in the outer fibers when they were set into the curvature functions \( \chi(M) \).

The calculated examples in annex B according to [11] proof a lot of logical and estimated effects and no contradiction to material properties occur.

7.3.3.7 Probability of advancement

The structure of the program permits the following advancements:

- The integrated creep and shrinkage functions can be substituted by others only by changing the module “funktionen” in the program. The VBE\textsuperscript{®} programming also allows the parallel use of two or more modules. So non-linear creep could be considered.

- A calculation of the ultimate limit state is possible as for composite girders the Eurocode only allows a elastic-calculation. It is already implemented in the program. This makes the consideration of different loading conditions possible.

- The calculation of asymmetric double-span bridges

- The calculation of bridges with different cross-sections

- The stress distribution in one cross-section at a specific time. This information is already contained in the table ‘calculation’ and must only be displayed

- Implementation of proofs according to the EC

7.3.4 Results of carried out calculations

The carried out tests show the following facts:
- Because of the construction progress the in-situ concrete participates only in carrying the extension load. This leads to a very low strain due to stress compared to the volume reduction due to shrinkage. As result the in-situ layer is subject to tension. This tension causes cracking. Support jacking can avoid this effect.

- The increase in deformation due to creep is very low because the internal forces change from the precast unit to the in-situ concrete and not to the steel girder.

- Using high compression strength concrete for the precast elements causes strong cracking to the in-situ concrete, but reduces deflection.

- Cracking is a problem regarding the serviceability limit state, because the superstructure loose its waterproofness and the substructure is then subject to corrosion. This fact shows the significance of an exact calculation.

The diagrams of the carried out tests are found in annex B [11].

7.3.5 Proposal for the enhancement of EC4-2

According to EC4-2 the elastic section properties of a composite cross-section with concrete in compression should be expressed as those of an equivalent steel cross-section by dividing the contribution of the concrete component by the relevant modular ratios. This is the principle known as ‘cross-section method’, described in [8].

\[ n_L = n_0 (1 + \Psi_L \phi) \]

is the modular ratio.

The creep-multipliers \( \Psi_L \) depend on the creep and ageing coefficient and the cross-section properties of the steel and composite section. For two-parted composite girders within the scope given in EC4-2 time constant creep multipliers are given.:

<table>
<thead>
<tr>
<th>Loading Conditions</th>
<th>( \Psi_L )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent loads</td>
<td>1.1</td>
</tr>
<tr>
<td>Shrinkage and time dependent hyperstatic effects</td>
<td>0.55</td>
</tr>
<tr>
<td>Prestressing by imposed deformations (e.g. jacking of supports)</td>
<td>1.5</td>
</tr>
</tbody>
</table>

If these creep multipliers are known, it is possible to calculate the stress distribution of a composite girder by the ‘cross-section method’. Haensel writes in [9] how to determine these creep multipliers analytically but also says that it is impossible to find some for the investigated cross-section. As result he gives an equation for two-parted composite girders with one concrete layer and remarks, that also these equations are not exact.

In order to use the ‘cross-section method’ it is essential to know the creep multipliers.

7.3.6 ‘CASE’ and the creep multipliers

7.3.6.1 Developing the formulas

The idea is to get the creep multipliers by recalculating them using the stress distribution that is calculated by the program ‘CASE’. It is intended to find cross-section independent values for the different loading conditions.

First of all, it is necessary to separate the loading conditions and then the following formulas can be set up.

For the dead load:
The compatibility conditions:

\[ \kappa_{ba}(t) = \kappa_{sta}(t) ; \quad \kappa_{bn}(t) = \kappa_{sta}(t) \]

\[ \varepsilon_{ba}(t) = \frac{-M_{sta}(t)}{E_{sta} I_{sta}} y_{banet} + \frac{N_{sta}(t)}{E_{sta} A_{sta}} , \quad \varepsilon_{bn}(t) = \frac{-M_{sta}(t)}{E_{sta} I_{sta}} y_{bn} + \frac{N_{sta}(t)}{E_{sta} A_{sta}} \]

with

\[ \kappa_{ba}(t) = \frac{M_{ba}(t)}{E_{ba}(t_0) I_{ba}} \cdot \frac{n_{bal}}{n_0} ; \quad \kappa_{bn}(t) = \frac{M_{bn}(t)}{E_{bn}(t_0) I_{bn}} \cdot \frac{n_{bnl}}{n_0} \]

\[ \varepsilon_{ba}(t) = \frac{N_{ba}(t)}{E_{ba}(t_0) A_{ba}} \cdot \frac{n_{baA}}{n_0} ; \quad \varepsilon_{bn}(t) = \frac{N_{bn}(t)}{E_{bn}(t_0) A_{bn}} \cdot \frac{n_{bnA}}{n_0} \]

and the modular ratios:

\[ \frac{n_{baA}}{n_{ba0}} = (1 + \Psi_{baAL} \phi_{ba}(t,t_0)) \]

\[ \frac{n_{bnA}}{n_{bn0}} = (1 + \Psi_{bnAL} \phi_{bn}(t,t_0)) \]

\[ \frac{n_{bnI}}{n_{bn0}} = (1 + \Psi_{bnIL} \phi_{bn}(t,t_0)) \]

\[ \frac{n_{bal}}{n_{bal0}} = (1 + \Psi_{bal} \phi_{bal}(t,t_0)) \]

result in:

\[ \frac{M_{sta}(t)}{E_{sta} I_{sta}} \cdot \frac{M_{bn}(t)}{E_{bn}(t_0) I_{bn}} = \Psi_{bnI} \]

\[ \frac{M_{sta}(t)}{E_{sta} I_{sta}} \cdot \frac{M_{ba}(t)}{E_{ba}(t_0) I_{banet}} = \Psi_{bal} \]

\[ \frac{-M_{sta}(t)}{E_{sta} I_{sta}} y_{banet} + \frac{N_{sta}(t)}{E_{sta} A_{sta}} \cdot \frac{A_{banet}}{E_{ba}(t_0) N_{ba}(t)} = 1 \]

\[ \frac{-M_{sta}(t)}{E_{sta} I_{sta}} y_{bn} + \frac{N_{sta}(t)}{E_{sta} A_{sta}} \cdot \frac{A_{bn}}{E_{bn}(t_0) N_{bn}(t)} = 1 \]

For shrinkage:
The strain for the concrete must be changed to:

\[
\varepsilon_{bn}(t) = \frac{N_{bn}(t)}{E_{bn}(t_{0})A_{bn}} + \varepsilon_{sbn}(t)
\]

\[
\varepsilon_{ba}(t) = \frac{N_{ba}(t)}{E_{ba}(t_{0})A_{bn}} + \varepsilon_{sba}(t)
\]

The results are as shown:

\[
\frac{M_{sta}(t)}{E_{sta}I_{sta}} \cdot \frac{E_{bn}(t_{0})I_{bn}}{M_{bn}(t)} - 1 = \psi_{bnA}
\]

\[
\frac{M_{sta}(t)}{E_{sta}I_{sta}} \cdot \frac{E_{banet}(t_{0})I_{banet}}{M_{ba}(t)} - 1 = \psi_{baI}
\]

\[
\frac{-M_{sta}(t)}{E_{sta}I_{sta}} \cdot y_{banet} + \frac{N_{sta}(t)}{E_{sta}A_{sta}} - \varepsilon_{sba}(t) = \psi_{baA}
\]

\[
M_{sta}(t) - M_{staold} \cdot \frac{E_{bn}(t_{0})I_{bn}}{M_{bn}(t)} = \psi_{bnA}
\]

\[
M_{sta}(t) - M_{staold} \cdot \frac{E_{banet}(t_{0})I_{banet}}{M_{ba}(t)} = \psi_{baI}
\]

\[
\frac{-M_{sta}(t) + M_{staold}}{E_{sta}I_{sta}} \cdot y_{n} + \frac{N_{sta}(t) - N_{staold}}{E_{sta}A_{sta}} - \varepsilon_{sba}(t) = \psi_{baA}
\]

\[
\frac{-M_{sta}(t) + M_{staold}}{E_{sta}I_{sta}} \cdot y_{n} + \frac{N_{sta}(t) - N_{staold}}{E_{sta}A_{sta}} - \varepsilon_{sba}(t) = \psi_{bnA}
\]

### 7.3.6.2 The tool ‘Analysis’ and its calculation of the creep-multipliers

The analyse tool of the Program ‘CASE’ does this calculation. The tool contains the following userform:
In order to calculate the creep-multipliers for permanent load, the cross section to be calculated and the time of first load $t_0$ are chosen. The check box for ‘new calculation’ must be enabled. Then the command button ‘dead load’ is pressed. The program now changes some settings and starts a new calculation. This will delete all former results. Afterwards for the chosen cross-section the ratio curves are drawn and the creep multipliers for the chosen $t_0$ and $t=10000d$ are calculated. The table ‘Analyse Tab’ contains all the information to calculate also the creep multipliers for other times than $t=10000d$.

If the creep-multipliers for shrinkage, that are also valid for time-changing forces shall be calculated, only the command button ‘shrinkage’ has to be pressed and for the time $t_0$ the time of after treatment of the in-situ concrete must be chosen.

If the creep-multipliers for imposed deformations shall be determined the button ‘imposed deformation’ is pressed. The textbox for $t_0$ must contain the moment of sagging.

If the checkbox ‘new calculation’ is disabled only the creep-multipliers and ratio curves are calculated again. This should not vary the coefficients or curves. The ratio curves can also be calculated from every stress distribution resulting from a former calculation. This is done by pressing ‘ratio curves’ and disabling ‘new calculation’.

### 7.3.7 Discussion of the results

The aim of the calculation is to achieve cross-section independent creep-multipliers. This is only possible for a certain scope of cross-sections. For the posed problem this should be possible due to the limited variations of the cross-section. It is always a construction steel girder with a two parted slab consisting of precast and in-situ concrete. Also the material specifications do not vary widely. Actually the creep multipliers depend on the creep-number, the ageing-factor and the geometry of the cross-section as shown in [9] and [8] they also vary with $t$ and $t_0$.

The creep multipliers given in the EC4-2 also consider the ageing-coefficient. This results in a difference between the creep multipliers for the in-situ concrete and the precast-concrete. For the precast concrete the ageing-coefficient is >1 and for the in-situ concrete the ageing-coefficient is <1. Consequently for the given cross-section 4 multipliers, distinguishing between the concrete and between area and second moment of area, must be given.
To develop verified multipliers it is necessary to make a large parameter study and error-investigation. Results calculated by the program must be compared to results calculated by the use of the determinate multipliers (the multipliers must be used for different cross-sections; using the creep-multipliers calculated from the same cross-sections results in no errors).

The extensive verification of the multipliers and the usually common error-investigation could not be carried out in the frame of this research project, due to the lack of test results investigating the creep and shrinkage effects of two-part concrete slaps in composite girders.

Nevertheless the results of the first tests show the following creep multipliers:

### Table 7-8: Creep multipliers

<table>
<thead>
<tr>
<th></th>
<th>$\Psi_{pc,A}$</th>
<th>$\Psi_{pc,I}$</th>
<th>$\Psi_{is,A}$</th>
<th>$\Psi_{is,I}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Constant load</td>
<td>0.914</td>
<td>1.065</td>
<td>1.026</td>
<td>1.268[1.1]</td>
</tr>
<tr>
<td>shrinkage</td>
<td>0.676</td>
<td>0.658</td>
<td>0.465</td>
<td>0.422[0.55]</td>
</tr>
<tr>
<td>Imposed deformation</td>
<td>0.93</td>
<td>1.102</td>
<td>1.053</td>
<td>1.402[1.5]</td>
</tr>
</tbody>
</table>

The values given in brackets [ ] are the values given in EC4-2.

where

- $\Psi_{pc,A}$ precast concrete and area
- $\Psi_{pc,I}$ precast concrete and second moment of area
- $\Psi_{is,A}$ in-situ concrete and area
- $\Psi_{is,I}$ in-situ concrete and second moment of area

It has to be remarked that the creep multipliers in Table 7-8 are results of only a few calculations. They are under no circumstances final or verified results.

The following pictures show the ratio curves from which the multipliers were calculated. In order to compare the ratio curves, the result from the calculation in 7.3.3.5 is shown here, too. The effect of the separation of the load conditions is very visible. The curves should be similar to the creep functions. The example for shrinkage leads to the idea of separating the shrinkage of the in-situ concrete from the shrinkage of the precast concrete. This could lead to more conform creep multipliers for shrinkage.

In [6] also creep-multipliers are developed. The results still show a variation of the creep-multipliers of more than 9% although a large study was carried out. The creep multipliers for the in-situ concrete, that are calculated by ‘CASE’ do not vary more than 9% from the values given in EC4-2.
Fig. 7-25: Ratio curves for permanent load

Fig. 7-26: Ratio curves for imposed deformation
7.3.8 **Conclusion according to the creep and shrinkage effects**

The long-term behaviour of concrete causes complicate processes in composite girders, especially if the slab consists of two different concrete with different ages.

Two possibilities exist for calculation:

- One possibility is the ‘cross-section method’. It has the advantages of a manual calculation and also its disadvantages. In the case of the described construction method it requires the separation of the construction process (three different static systems) and the separation of the load conditions (at least three). Then the hyperstatic force has to be determined in an iteration process, while cracking is still not considered. Considering cracking requires an assumption for the cracking zone or an additional iteration process. If also tension stiffening should be considered the calculation is not anymore accessible for manual calculation. But also neglecting tension stiffening and cracking, it might be doubted that this possibility is really a solution. In every case the determination of the creep multipliers is still necessary, what can be done using ‘CASE’.

- The other possibility is the use of the program ‘CASE V2.2’ or other software. In the case of ‘CASE’ it can be said that no assumptions are made that are not allowed in the Eurocode. It only executes the calculations explained in EC4-2 and considers the facts given there. It is easy and comfortable to use and allows the control of the calculation.
7.3.9 List of references according to the creep and shrinkage effects

[1] Betonkalender 2000; Verlag Ernst & Sohn
[8] Prof. Dr.-Ing. G. Sedlacek: “Umdruck zur Vorlesung und Übung Verbundbau”; Lehrstuhl für Stahlbau, RWTH Aachen
7.4 Vibration of composite bridges (additional working item)

Today most of the bridges are built of concrete due to general habituation. In future slight composite constructions shall be used with girders made of high-strength steel and prefabricated concrete slabs. In this regard a project of the European Committee for Steel and Coal (ECSC): “composite bridge design for small and medium spans” shall result in a compilation of design rules for such bridges. The use of high strength material leads to a loss of weight and therefore to a higher susceptibility to vibration. This report investigate the influences on human beings crossing the construction simultaneously with vehicle traffic.

Vibrations in massive buildings are intuitively associated with a insufficiency of the construction or even with the danger of collapse. Moreover the perception depends on a multitude of factors as the bearing, the psychological and physiological conditions, the climate or synchronous sonic sources. For that reason the evaluation of human exposure to vibration is very difficult.

In literature /2/ such frequencies are rated as displeasing that are in accordance with the eigenfrequencies of the human body (resonance). Examples of some eigenfrequencies of the human body can be taken from Table 7-9.

<table>
<thead>
<tr>
<th>Place of impact</th>
<th>Eigenfrequency [Hz]</th>
</tr>
</thead>
<tbody>
<tr>
<td>inner organs</td>
<td></td>
</tr>
<tr>
<td>- standing</td>
<td>4 – 12</td>
</tr>
<tr>
<td>- lying</td>
<td>3 – 4</td>
</tr>
<tr>
<td>- sitting</td>
<td>5 – 6</td>
</tr>
<tr>
<td>hand-arm-shoulder-head-system</td>
<td>10 – 20</td>
</tr>
<tr>
<td>lower jaw in reference to the skull</td>
<td>100 – 200</td>
</tr>
<tr>
<td>Skull</td>
<td>300 – 400</td>
</tr>
</tbody>
</table>

Additionally long-term vibration with frequencies lower then 1 Hz may cause kinetosis (e.g. seasickness) which means a congestion of the organ of balance.

7.4.1 Carried out investigations and measurements

In the frame of this research project the following important standards and publications dealing with the evaluation of human exposure to vibration are investigated:

- VDI-Standard 2057, part 1-4 /1/
- Norm ISO 2631, guide for the evaluation of human exposure to whole–body vibration, part 1 and 2 /2/
- Prenorm DIN 4150, part 2 /4/
- British Standard BS 6472 /7/
- Design guide on the vibration of floors /8/
- IABSE report: “Dynamic design of footbridges” /13/
Vibrations due to pedestrian traffic on footbridges, tentative draft (Following the British Standard BS 5400, part 2) /21/

To verify the adaptability of these standards and publications on the evaluation of composite bridges in small and medium spans measurements of 7 composite bridges (Germany) were carried out.

Except bridge number 5008 761 where the measurement lead to insufficient results, the acceleration caused by traffic load was measured on the following constructions:

Table 7-10: Characteristic values of the investigated composite bridges

<table>
<thead>
<tr>
<th>bridge number</th>
<th>year of construction</th>
<th>span length [m]</th>
<th>Number of spans</th>
<th>width between railings [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>4506 903</td>
<td>1985</td>
<td>51.07</td>
<td>3</td>
<td>49.98</td>
</tr>
<tr>
<td>4608 704</td>
<td>1996</td>
<td>124.2</td>
<td>5</td>
<td>12.43</td>
</tr>
<tr>
<td>5005 585</td>
<td>1988</td>
<td>42.8</td>
<td>1</td>
<td>14</td>
</tr>
<tr>
<td>5008 752</td>
<td>1974</td>
<td>50.2</td>
<td>2</td>
<td>7</td>
</tr>
<tr>
<td>5008 761</td>
<td>1974</td>
<td>50.75</td>
<td>2</td>
<td>10.9</td>
</tr>
<tr>
<td>5109 776</td>
<td>1973</td>
<td>57.42</td>
<td>2</td>
<td>13.5</td>
</tr>
<tr>
<td>5109 777</td>
<td>70’s</td>
<td>36.01</td>
<td>1</td>
<td>19.62 *)</td>
</tr>
</tbody>
</table>

*) One single bridge for every roadway

Concerning measurements and calculations of acceleration, velocity and displacement the following standards was used:

- DIN 45669, part 1, Messung von Schwingungsimmissionen /5/
- DIN 45671, part 1, Messungen mechanischer Schwingungen am Arbeitsplatz /9/
- DIN V ENV 28041, Schwingungseinwirkungen auf den Menschen, Messeinrichtungen /10/

The used equipment as well as a detailed description of the investigated bridges can be taken from the annual report 2000.

7.4.2 Evaluation of the existing standards

Fig. 7-29 and Fig. 7-30 show exemplary the evaluation of the measurements according to /1/.

From the acceleration four time periods are established on which the following actions are applied:

- low-pass filtering
- Fast Fourier Transformation (FFT) ? Amplitude peak
- third octave band filtering

As a result the Eigenfrequency can be taken from the peak values in figure 5.1 and the third octave spectra can be compared with curves of equal vibration intensity K according to VDI-Standard 2057 /1/ (Fig. 7-30).
These curves of equal vibration intensity $K$ according to VDI-Standard 2057 correspond to attributes according to the following table:

<table>
<thead>
<tr>
<th>Valued vibration intensity $K_X, K_Y, K_Z, K_B$</th>
<th>Description of perception</th>
</tr>
</thead>
<tbody>
<tr>
<td>$&lt; 0.1$</td>
<td>no perception</td>
</tr>
<tr>
<td>$0.1$</td>
<td>threshold of perception</td>
</tr>
<tr>
<td>$0.4$</td>
<td>just perceptible</td>
</tr>
<tr>
<td>$1.6$</td>
<td>clearly perceptible</td>
</tr>
<tr>
<td>$6.3$</td>
<td>strongly perceptible</td>
</tr>
<tr>
<td>$100$</td>
<td>considerably detectable</td>
</tr>
<tr>
<td>$&gt;100$</td>
<td></td>
</tr>
</tbody>
</table>
Within his thesis S. Drosner /12/ sets an upper limit for vehicle induced perceptible vibrations on bridges at 15 Hz. Therefore the comparison of the personal estimation and the results according to the VDI-Standard is distinguished in frequencies from 1 to 15 Hz and from 15 to 80 Hz. In Fig. 7-30 it can be seen that within a band of 10 to 20 Hz the estimation lies beneath the perception threshold. The evaluation of all measurements using the VDI standard is summarised in the following table.

Table 7-12: Comparison of personal estimations and results according to VDI-Standard 2057

<table>
<thead>
<tr>
<th>Bridge number</th>
<th>acc. to VDI-Standard 2057</th>
<th>personal estimation, standing position</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>&lt; 15 Hz</td>
<td>&gt; 15 Hz</td>
</tr>
<tr>
<td>4506 903</td>
<td>perception threshold</td>
<td>&lt; perception threshold</td>
</tr>
<tr>
<td>4608 704</td>
<td>&lt; perception threshold</td>
<td>&lt; perception threshold</td>
</tr>
<tr>
<td>5005 585</td>
<td>clearly perceptible</td>
<td>Perception threshold</td>
</tr>
<tr>
<td>5008 752</td>
<td>perception threshold</td>
<td>&lt; perception threshold</td>
</tr>
<tr>
<td>5008 761</td>
<td>&lt; perception threshold</td>
<td>&lt; perception threshold</td>
</tr>
<tr>
<td>5109 776</td>
<td>strongly perceptible</td>
<td>Clearly Perceptible</td>
</tr>
<tr>
<td>5109 777</td>
<td>clearly perceptible</td>
<td>Clearly Perceptible</td>
</tr>
</tbody>
</table>

The use of the VDI-Standard leads to very good results concerning the estimation of the measuring persons. Most pedestrians could not give any statement about vibrations which confirms that tendency. An exception is bridge number 5109 777, which is slightly underestimated by the standard.

The full presentation of the evaluation of the above mentioned most common standards and publications can be taken from /22/. The evaluation can be summarised as follows:

- The used criteria are the valued vibration intensity \( K \), the durance of impact, the vibration dose value (VDV) and a maximum tolerable acceleration.
- Concerning footbridges an appraisal by means of the Eigenfrequency underestimates the personal opinion from which the bridges 5109 776 and 5109 777 can be emphasized.
- With regard to the tolerable durance of vibration impact both, norm ISO 2631/1 and the BS 6472 do not come to good results. Those rules are defined for long-term incidents and therefore are not applicable for bridges of small and medium spans.
- The best estimation can be made through a basic approach by the valued vibration intensity \( K \). Considering the existing norms the VDI-Standard 2057 can be rated as the best because the appraisals are very close to the personal opinions.
- Prenorm DIN 4150, part 2 as well works with the \( K \)-value but prescribes a determination by means of the velocity. The evaluation of the bridge reaction taking effect on pedestrians is expedient, too.
Finally the VDV from the BS 6472 shall be mentioned which sets too high limits for an evaluation of vibration impact on composite bridges.

### 7.4.3 Numerical investigations

The numerical investigations were carried out using the program DYNACS of the Institute of Steel Construction, RWTH Aachen. With regard to the dynamic behaviour of composite bridges the output of this program was verified using the measurements on the bridge number 5109 776. The detailed description of the discretization of this bridge as well as specific inputs in the program can be taken from /22/.

#### 7.4.3.1 Verification of the used program DYNACS

In the following exemplary result from the DYNACS-verification is compared to three diagrams from a measurement on bridge number 5109 776. Due to the fact that the calculated time periods are not in accordance with the measured acceleration-time-history the vibration occurrence are deferred and not the same time periods are compared.

**Fig. 7-31:** Comparison of the acceleration amplitudes
**7.4.3.2 Investigation of the influence of various parameters**

After achieving a satisfying approximation of the behaviour of bridge and vehicle the following parameters were examined:
1) variation of the velocity (20, 50, 80, 100 km/h)
2) generation of the lorry in one single or two parallel loads
3) different roadway throatinesses according to /12/
4) influence of roadway joints, prestressing and gradients of the superstructure

These extensive investigation lead to the following results:

1) First of all an increase of the velocity leads to an intensification of the primary impact at the roadway joint. The clear periodical amplitude changes to an irregular one. Moreover up to 80 km/h it increases from about 0.15 to 0.35 m/s², after that it remains constant. This can be ascribed to the strong influence of the vehicle on the bridge vibration while driving slowly. Here the superstructure reacts in the same frequency as is induced by the lorry. Faster crossings finally show a construction frequency of 1.2 Hz which is close to the determined value according to the DYNACS eigenvalue analysis ($\omega_0 = 7.70$ rad/s).

Within the whole range of velocity the average rating of VDI-Standard is “just perceptible”.

2) Due to the transversal roadway joint and a generation of two loads the first clear impact disappears because of a superposition of the two excitements. The further behaviour is the same as the one of a single load.

3) The apply of a medium roadway quality according to /12/ result in a remarkable increase of the amplitude from 0.15 to 0.25 m/s² at 20 km/h. Compared with a good quality the vibrations are rated more intense but do not reach the clearly perceptible limit. The eigenfrequencies do not change.

4) Chapter 6.3.10 according to /22/ describes a roadway joint that was modified to such an extend that the middle width was enlarged from 2 to 10 cm according to /19/. The effect is an increase of the overall acceleration amplitude.

Considering the Eigenfrequency and the threshold of perception no change occurs.

Neither the neglect of the prestressing nor of the gradients of the superstructure show any effect.

7.4.4 Conclusion according to the dynamic behaviour of composite bridges

Summarising the following conclusion can be drawn:

- For composite bridges in small and medium spans usually no adverse comments of pedestrian crossing the bridge simultaneously with traffic occurred.
- Due to some strong perception of vibration occurred in one of the carried out measurements it seems possible that the application of high strength steel and concrete slabs of low weight in future result in vibration problems.
- If it is necessary composite bridges should be examined using VDI-Standard 2057.
- Finally it can be stated that the FE-Program DYNACS (developed from the Institute of Steel Construction, RWTH Aachen) is useful to predict the dynamic behaviour of composite bridges of small an medium spans under vehicle traffic.
7.4.5 List of references according to the dynamic behaviour of composite bridges

/1/ VDI-Richtlinie 2057, VDI-Verlag GmbH Düsseldorf, Augabe Mai 1987
/4/ Vornorm DIN 4150: Erschütterungen im Bauwesen, Ausgabe Dezember 1992
/7/ British Standard BS 6472, Ausgabe 1992
/9/ DIN 45671, Teil 1: Messungen mechanischer Schwingungen am Arbeitsplatz, Ausgabe September 1990
/10/ Vornorm DIN V ENV 28041: Schwingungseinwirkungen auf den Menschen, Ausgabe Juni 1993
/14/ H. Bachmann und W. Ammann: “Schwingungsprobleme bei Bauwerken”; IABSE, Zürich, 1987
/15/ Umdruck zur Vorlesung und Übung Verbundbau, Lehrstuhl für Stahlbau, RWTH Aachen, SS 1999
/16/ Klaus-Jürgen Schneider: “Bautabellen für Ingenieure”; 11. Auflage, Düsseldorf, 1994
/17/ Reto Paduot Cantieni: “Beitrag zur Dynamik von Straßenbrücken unter der Überfahrt schwerer Fahrzeuge”; Diss. ETH Nr. 9505, Zürich, 1991
/19/ Verbesserung der Gebrauchseigenschaften von Brücken hinsichtlich Schwingungen; Heft 615, Forschung Straßenbau und Verkehrstechnik, Bundesministerium für Verkehr, Prof. Dr.-Ing. G. Sedlacek, Dr.-Ing. S. Droser, Bonn, Aachen, 1992
/21/ Vibrations due to pedestrian traffic on footbridges; Tentative Draft, Service d'études techniques des routes et autoroutes (SETRA), Bagneux, April 2000
8 CONTRIBUTION OF PROFILARBED

8.1 Deliverance of steel girders

All the steel profiles needed for the different tests at Luleå and at Liège are HEA 900 in S460 and have been rolled in March 2000. All the specimens have been assembled in April and sent in May to Liège and Luleå, with exception of Test 3b for Liège, which was on hold. This last specimen for the test 3b has been assembled in April 2001, after analysing the results from the tests 3a.

During the years 2000 and 2001 some visits have been undertaken to Liège in order to follow the fatigue tests on the different specimens.

23 test specimens for the University of Wuppertal, which are small pieces of HE 300A in S460 for push-out tests, have been assembled and delivered in March 2000. On request of the University of Wuppertal a second delivery of 10 other beams has been organised in June.

Table 8-1: Delivering dates

<table>
<thead>
<tr>
<th>Partner</th>
<th>Test N°</th>
<th>Delivered</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wuppertal</td>
<td>1a, 1b, 2, 3</td>
<td>30/03/00</td>
</tr>
<tr>
<td>Liège</td>
<td>1.3, 1, 2, 3a</td>
<td>08/05/00</td>
</tr>
<tr>
<td>Luleå</td>
<td>9a, 9b</td>
<td>22/05/00</td>
</tr>
<tr>
<td>Wuppertal</td>
<td>10 x 650mm HEA300</td>
<td>05/06/00</td>
</tr>
<tr>
<td>Liège</td>
<td>3b</td>
<td>25/05/01</td>
</tr>
</tbody>
</table>
8.2 Results of the tensile tests

8.2.1 Tests at the Universities of Liège and Luleå

At least one tensile test has been made at ProfilARBED for each test specimen. The results are given here beneath. In addition small steel pieces (T4 for Liège and T6 for Luleå) have been sent to the laboratories so that they can also make their own tensile tests.

![Fig. 8-1: Cutting of the HEA 900 profiles](image-url)
The tables below show the results of the tensile tests made at ARBED:

Table 8-2: Results of the tensile tests on the HEA 900 beams

<table>
<thead>
<tr>
<th>BEAM</th>
<th>Yield Point [N/mm²]</th>
<th>Tensile Strength [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>H 576</td>
<td>691</td>
</tr>
<tr>
<td></td>
<td>B 571</td>
<td>682</td>
</tr>
<tr>
<td>T2</td>
<td>H 570</td>
<td>694</td>
</tr>
<tr>
<td></td>
<td>B 563</td>
<td>678</td>
</tr>
<tr>
<td>T3</td>
<td>H 572</td>
<td>690</td>
</tr>
<tr>
<td></td>
<td>B 558</td>
<td>677</td>
</tr>
<tr>
<td>T5</td>
<td>H 569</td>
<td>673</td>
</tr>
<tr>
<td></td>
<td>B 572</td>
<td>679</td>
</tr>
<tr>
<td>T7</td>
<td>H 563</td>
<td>680</td>
</tr>
<tr>
<td></td>
<td>B 553</td>
<td>662</td>
</tr>
<tr>
<td>T8</td>
<td>H 568</td>
<td>682</td>
</tr>
<tr>
<td></td>
<td>B 559</td>
<td>668</td>
</tr>
<tr>
<td>Test 3b</td>
<td>H 499</td>
<td>617</td>
</tr>
<tr>
<td></td>
<td>B 501</td>
<td>616</td>
</tr>
</tbody>
</table>

8.2.2 Tests at the University of Wuppertal

The specimens made of the beams 2-C1, 2-C4, 2-C6 and 2-C9 were sent in March to the University of Wuppertal. On request other specimens made of the beams 2-C2 and 2-C3 were sent in May to them.
Table 8-3: Results of the tensile tests on the HEA 300 beams

<table>
<thead>
<tr>
<th>BEAM 2-C 1</th>
<th>Yield Point [N/mm²]</th>
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<tbody>
<tr>
<td>H</td>
<td>525</td>
<td>566</td>
</tr>
<tr>
<td>B</td>
<td>517</td>
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<table>
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<tbody>
<tr>
<td>H</td>
<td>496</td>
<td>544</td>
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<tr>
<td>B</td>
<td>523</td>
<td>572</td>
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</tbody>
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<table>
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<td>496</td>
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<td>576</td>
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<tr>
<td>B</td>
<td>516</td>
<td>588</td>
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8.3 Assembling details for the Tests 3 and 4 for Liège

The problem to solve, was the weld of the lower flange to the cape plate of test N° 3 and 4. The question was to make a weld, which resists under fatigue and which isn’t too expensive to realise.

It has been decided to assemble test 3 with a fillet weld. The problem is, that following EUROCODE 3 a weld with full penetration has a fatigue life of $N = 0.68 \times 10^6$ cycles, but a fillet weld has only a fatigue life of $N = 0.09 \times 10^6$ cycles, which is nearly eight times lesser. But the EUROCODE 3 makes no difference about compression and traction, and as the lower flange is always in the compression zone, there is theoretically no danger of failure because all the efforts of compression are transmitted by contact. The aim of this first test with a fillet weld is, to proof that even if there will be cracks in the weld during the test, they won’t have any influence on the behaviour of the specimen.

Fig. 8-2 shows the solution with fillet welds for the web and the flanges. The end of the beam will not be machined, as not every fabrication hall is presumed to have the possibility to do this, and the aim of this research is to use user-friendly connections. There is no problem to weld in the corner between the web and the flanges. It’s a usual procedure for steel fabricators. The only thing to take care about is to pass from a thickness $a=18$ mm welding to a thickness $a=10$ mm welding.
First test of Test 3a: Filled weld

Test 3a:
Detail 2: Connection of the girder with the cape-plate
Filled weld

Scale: 1/2

Test 3b:
Detail 3a: Connection of the girder with the cape-plate
Full penetration

(Second) test of Test 3b: Full penetration

Fig. 8-2: Connection detail of Test 3

Fig. 8-3: Proposal for the connection detail of Test 4
Due to the fact that, following the Eurocodes, a fillet weld has a very bad fatigue behaviour, some other connection details for the second test 4 have been discussed.

Fig. 8-3 shows a solution with a full penetration. The flange and the web have to be chamfered dis-symmetrically from both sides in an angle of 40°, with a connection point of 2 mm width. As the steel fabricators are able to turn the beam by the different welding steps, it is easier for them to weld on the outside of the beam as on the inside near the web. So the chamfering is foreseen 2/3 of the flange thickness on the outside and 1/3 on the inside. In this case a machining of the end of the beam isn’t needed.

The detail of the chamfering of detail 3 is a proposal of our fabrication hall, and stays for a typical chamfering for this kind of connections (see Fig. 8-4).

![Fig. 8-4: Chamfering of the detail with full penetration](image)

As we got very good results from the test 3, we went a step further for the second identical test 4 by choosing smaller fillet weld thickness as well as by adding an imperfection (see Fig. 8-5).

In addition, the left and the right side from the test specimen have also been assembled differently in order to get two test results from only one specimen (see Fig. 8-6).
Fig. 8-5: Left side of the specimen 4

Fig. 8-6: Right side of the specimen 4
8.4 Design guide

ProfilARBED sent its contribution to the design guide until the end of August 2001 to RWTH. The main topics of the contribution are listed below:

- Fabrication and Erection
  - Fabrication of the steel construction
  - Erection of the steel construction
  - Manufacture of the deck
  - Construction time
8.5 Welding Procedures

8.5.1 Welding procedure of test specimen 3 and 4

The welding procedure of the test 4 is similar with the exception that we’ve less passes due to the fact that we have used thinner fillet welds.
8.5.2 *Welding procedure used in practice*

The following pictures show how many passes you have to apply in order to get a final fillet weld of 18 mm. In general you have to finalise a pass all around before staring the next one. As, with a normal installation, you have to weld only from the top, you have to turn the beam several times. If you are welding huge fillet welds on heavy and long beams, and you don’t want to turn it so often, you can sometimes apply, 2 layers at once, but you have to be very carefully not to heat up to much the basic material.

**First layer**
- Pass 1
- Weld thickness 4 – 6 mm

**Second layer**
- Pass 2 and 3
- Weld thickness 8 mm

**Third layer**
- Pass 4, 5 and 6
- Weld thickness 10 mm

**Fourth layer**
- Pass 7, 8, 9 and 10
- Weld thickness 14 mm

**Sixth pass**
- Pass 11, 12, 13, 14 and 15
- Weld thickness 18 mm
Concerning the specimen for the test 3 the endplate has been welded to the beam by the following way (for test 4 it was similar but with smaller fillet welds):

Step 1: Tack welds in order to fix the beam (red dots)

Step 2: Application of the first layer by using the following order:
Starting by (1) and turning the beam upside down
Continuing by (2) and turning the beam by 90°
Continuing by (3) and turning the beam upside down
After (4) the first layer has been applied

Step 3: The beam has to be put into the initial position. Step 2 has been repeated twice in order to apply the second and the third layer. Now we have a fillet weld of 10 mm thickness everywhere.

Step 4: As for the lower flange, a fillet weld of 18 mm is foreseen, we had to apply 2 other layers on it. After the beam has been brought again to the initial position we started again with (5), turned the beam upside down continued with (6), than turned the beam by 90°, continued with (7), finally turned the beam upside down and finished with (8).

By proceeding like this, we avoided a too high level of internal stresses in the weld. For the same reason, it’s important to start with the “critical” weld.
The 2 pictures below show the effective thickness of the fillet welds of test specimen 3, which have been measured afterwards.
8.5.3 Welding procedure of studs

ARBED didn’t make a quality control of the studs during the assembling of the specimens. The welding procedure described below is representative for studs of diameter 22 welded on a similar profile type (HE 700M in S460 instead of a HE 900A in S460).
### DETAILS DE L'ASSEMBLAGE D'ESSAI
CERTIFICAT DE QUALIFICATION D'UN MODE OPERATOIRE

<table>
<thead>
<tr>
<th>Organisme de contrôle:</th>
<th>SOU 01/17</th>
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<td>No de référence:</td>
<td>LC 5504/01S</td>
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#### Mode opératoire de soudage du constructeur
- **Méthode de préparation et nettoyage:** fraisage + moulage

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#### DETAILS DE LA PREPARATION DE L'ASSEMBLAGE (croquis)

#### PARAMETRES DE SOUDAGE

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<th>Délai minimum d'apport</th>
<th>Intrant</th>
<th>Tension</th>
<th>Type de courant</th>
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<td>2050</td>
<td>43.5</td>
<td>DCC</td>
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- **Métal d'apport marque et type:** Goujone TRW Nelson-a-22 mm (S37-3K)

- **Précaution de séchage ou d'étuvage:** /

#### TRAITEMENT THERMIQUE APRES SOUDAGE

- **Durée, température, méthode:** /

- **Vitesse de chauffage et de refroidissement:** /

- **Le coupé témoin ci-dessous a été souduit en présence de:** /

---

**Organisme de contrôle:** LC Luxcontrol asbl

---

**Nom et signature:** G. Noltermann
### RESULTATS DES ESSAIS

CERTIFICAT DE QUALIFICATION D'UN MODE OPERATOIRE

---

**Organisme de contrôle:**

**SOU 01/17**

**No de référence:**

**LC 3524015**

---

**Mode opératoire de scouage du constructeur:**

**No de référence:** WPS GJ22-Rev.0

---

**Examen visuel:** conforme

**Radiographie:** non effectué

**Batelage ultrasonique:** non effectué

---

**ESSAIS DE TRACTION** suivant EN 865

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**ESSAIS DE PLOIAGE** suivant NF A89-020-1

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**ESSAIS DE RESILIENCE** suivant EN 875

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**ESSAIS DE DURETÉ** suivant EN 1043-1

20 Remarques: /

---

**Les essais ont été effectués en conformité avec les exigences de la norme:** NF A89-020-1

**No de référence du support du laboratoire:** LC 5524/01S

**Les résultats des essais sont acceptables.**

---

**Organisme de contrôle:**

(Organisation Membre de la CEOK)

**LC Luxcontrol asbl**
8.5.4 Control of the welds of specimen 3 by an independent control office

| UT | Rapport / Bericht / Report | avec/mit/with | pages/Seiten/pages |
| X | MT | A 7196-MT | avec/mit/with | 1 | pages/Seiten/pages |
| PT | Rapport / Bericht / Report | avec/mit/with | pages/Seiten/pages |
| VT | Rapport / Bericht / Report | avec/mit/with | pages/Seiten/pages |

**Contractor:** PROFILARBED Differdange.

**Command No:** 5.880.924198

**Project:** 2 poutres HE 900 A, repérées 3P ET 3L avec platines de 70 x 480 x 1000 mm, repérées A113.

**Reference Intern.:** LC 4508/00C

**Date of report:** 12.04.2000

**Operator responsible:** Mathieu P.

**Annexes:**

**Distribution of the report:** PROFILARBED Differdange

**Report distributor:** SERVICE PARACHÈVEMENT

A l'attention de M. Tani C.

L - 4503 Differdange

LC Luxcontrol asbl

Braun J. chef de service
## Composite Bridge Design for Small and Medium Spans

---

### Rapport d'examen

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Remarques: Les soudures contrôlées sont conformes aux critères d'acceptation du code ASME, section VIII, division 1, appendix 6.

---
8.5.5 Control of the welds of specimen 4 by an independent control office

Rapport de visite
No: A 7957

Date de visite: le 27 avril 2001
Date de rapport: le 30 avril 2001
Inspecteur responsable: Papa
Tél. ext. - 215

Concerné: 2 poutres HEA 900, repère EG74 A et B, pour ARBED Recherches.

DESCRIPTION DE L'OBJET

Contrôles non destructifs des soudures de rabotage des poutres A et B/plaque de tête.
(voir plan de situation en annexe 1)

VT CONTRÔLE VISUEL/DIMENSIONNEL

VT 01 Objet et étendue du contrôle

a) Contrôle visuel à 100% des soudures de rabotage des poutres A et B/plaque de tête.
b) Contrôle dimensionnel de chacun des cordons d'angle (ailles et âme) des soudures de rabotage des poutres A et B/plaque de tête.

VT 02 Contrôle effectué suivant

Procédure de contrôle: LC CMS-VT 001.
Critères d'acceptation: EN 28817, classes B et C pour les soudures.

VT 03 Résultats du contrôle

a) Aucune anomalie particulière n'a été observée au niveau des soudures contrôlées.
   Les soudures contrôlées sont conformes aux critères d'acceptation définis au paragraphe VT02.
b) Les gorges des cordons d'angle sont conformes aux cotations des plans en annexes 2 et 3.

MT CONTRÔLE PAR MAGNETOSCOPIE

MT 01 Généralités

Appareillage: Electro-aliment Karl Deutsch 35 V, repère MAG 012.
Réalisateur: Deutroflux noir sur fond blanc.
Magnetisation: Suivant 2 directions perpendiculaires.

MT 02 Contrôle effectué suivant

Procédure de contrôle: LC-CMS-MT 001.
Critères d'acceptation: Code ASME, section VIII, division 1, appendix G.

MT 03 Objet et étendue du contrôle

Contrôle par magnétoscopie à 100% des soudures de rabotage des poutres A et B/plaque de tête.

Les résultats d'essai se rapportent uniquement aux positions (cièces) examinées.
Ce rapport ne peut pas être partiellement reproduit sans l'accord écrit et préalable de LC Luxcontrol AG.

Av des Termes Rouges BP 35C L-4001 Esch-sur-Alzette +352 54 70 5 X +352
8.5.6 Control of the welds of specimen 4 by an independent control office

8.5.6 Contrôle des soudures de l’échantillon 4 par un contrôle indépendant
8.6 References

[1] NFA89-020-1
[2] EN 25817
9 SOFTWARE FOR THE PREDESIGN OF THE BRIDGE

In frame of the project a computer program named CBD (Composite Bridge Design) has been developed to pre-design composite bridges for small and medium spans. The aim of the software is to find optimised solutions for hot-rolled and plate girders. The design is based on the Eurocodes.

The software is programmed in VISUAL FORTRAN 90. Special regard was laid on user friendly interfaces to allow an easy creation of an input file without any further knowledge.

The program works with the method to divide multi-beam bridges into strips of one beam with a corresponding concrete chord. If the design checks of each strip are satisfied, the whole structure will be considered to satisfy all required design checks.

The calculable constructions are rectangular, simple or multi-span composite bridges consisting of a prefabricated steel beam and a concrete slab. The slab can be solid or constructed using partially or fully prefabricated elements. The longitudinal and transverse reinforcement is considered not to be pre-stressed. All construction situation of the structure are considered and checked. With the implementation of high-strength materials and hybrid girders, new research developments of the modern structural engineering have been implemented in the software (e.g. shear connectors in high strength concrete).

The program is designed in three independent layers.

9.1 The Main Menu

The first layer of the program is the main menu. All main actions of the program are initialised out of the main menu using pull-down menus.

One action is to call user-friendly dialog boxes where input data of a bridge is entered. The user is able to apply data for a complete new superstructure or edit a former specified bridge data input file. Additionally it is possible to print input or output files out of the main menu. The calculation is also started out of the main menu. Online-Help to the program is available.
9.2 The dialog boxes

The second layer of the program is represented by the dialog boxes, the user interface to apply data for the superstructure.

![Static system](image1)

*Fig. 9-2: Static system*

The user can choose between the pre-design of a simple, two-span or multispans bridge with different support situations. Each span length of the bridge could be entered separately and the bay width of the bridge, which is used to estimate the self-weight, is individually modifiable.

![Concrete Plate](image2)

*Fig. 9-3: Concrete Plate*

The slab can be constructed as a solid slab or with the help of partially or fully prefabricated elements. The cross section in the field is differentiated from the cross section over the support.
Each cross section, the concrete class (from C 20/25 up to C 100/115) and the reinforcement strength can be varied.

Rolled or plate girder sections are implemented. Choosing a rolled section means that the program determines the section, increasing standard profile sections out of tables, until the checks for the ultimate limit state and serviceability limit state of EC3 and EC 4 are satisfied in all construction and final situations. For welded sections, the height of the profile is increased from 500 [mm] up to a maximum of 2500 [mm] in 10 [mm]-increments up to the point where the former specified limitations are satisfied. To finally receive an optimised plate section, the flange and the web thickness has to be modified for each calculation run. A special regard should be taken on the possibility to apply hybrid girders for the structure. The common steel is included in the software and additionally high-tensile steel (S460) can be chosen.
Studs with a variable height, diameter (up to 25 mm) and steel strength can be selected as shear connectors. The possibility to use an alternative shear connector with a user specified bearing capacity is added. The distribution of the shear connectors is up to the user and has to be checked additionally.

The specification of the loads are differentiated between their type, kind, place and temporary occurrence. The specification has an influence on safety and combination factors in the checks. The self-weight of the steel girder, the concrete slab and the actions due to creep and shrinkage at $t = 8$ are estimated by the program itself.
The following loads have to be added by the user:

- Construction loads (for checks in the construction situation)
- Permanent loads
- Traffic loads
- Temperature loads
- Breaking loads

Generally loads have to be determined considering their transversal distribution over the cross section of the bridge and entered with regard to their place and magnitude.

Fig. 9-7: Settings

All checks are done for variable construction situation and final situations. The place of the joint between the steel girders in the construction phase can be modified. Safety factors of different nations can be chosen for the design checks. It is up to the user, if creep, shrinkage, moment distribution and a user specified deflection limit is taken into account.
9.3 The Calculation Module

The third layer of the software is the calculation module. After opening an input file the pre-design calculation referring to EC 3 and EC 4 is initialised.

The main steel beams are coupled with each other through the concrete plate. Transversal plates are only considered at the supports, which are not part of the composite construction. The supports are designed to be directly under the web. The materials implemented in the program are introduced in the Design Guide of this project.

During the calculation design points are defined at the quarter points and end points of each bridge span. All checks of the ultimate limit state and serviceability limit state of EC3 and EC4 are carried out at these design points.

The cross section is determined with respect to time. Therefore the area and moment of inertia is calculated for $t = 28$ days and $t = 8$.

The inner forces due to the loads are elastically calculated. A moment distribution related to the cross sectional class of the steel beam can be carried out.

Starting with the determination of the steel and composite cross sections for all construction and final situations, differentiated between field sections and sections over the support and time, the loads are applied and the inner forces are estimated.

Table 9-1: Construction Sequences for a solid- and fully prefabricated slab

<table>
<thead>
<tr>
<th>Construction Situation</th>
<th>Description</th>
<th>Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>System 1</td>
<td>Multispan bridges: Steel beam with a hinge</td>
<td>Dead load of the steel beam</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>System 2</td>
<td>Steel beam with welded hinge</td>
<td>Dead load of the solid slab or the fresh concrete and construction loads</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>System 4</td>
<td>Full composite action considering cracks in concrete in the hogging moment area</td>
<td>Infrequent and frequent load</td>
</tr>
</tbody>
</table>
Table 9-2: Construction Sequences for a partially prefabricated slab

<table>
<thead>
<tr>
<th>Construction Situation</th>
<th>Description</th>
<th>Load</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>System 1</strong></td>
<td>Multispan bridges: Steel beam with a hinge</td>
<td>Dead load of the steel beam</td>
</tr>
<tr>
<td><strong>System 2</strong></td>
<td>Steel beam with welded hinge</td>
<td>Dead load of the partially prefabricated slab</td>
</tr>
<tr>
<td><strong>System 3</strong></td>
<td>Partial composite action</td>
<td>Dead load of the fresh concrete and construction loads</td>
</tr>
<tr>
<td><strong>System 4</strong></td>
<td>Full composite action considering cracks in concrete in the hogging moment area</td>
<td>Infrequent and frequent load</td>
</tr>
</tbody>
</table>

The relevant forces for each construction phase are multiplied by safety and combination factors and summed up. With the load combinations and the estimated cross sections the program is able to perform the following checks of the ultimate limit state and serviceability limit state.

**Ultimate limit state of the steel girder referring to EC 3:**
- ultimate resistance against positive bending
- ultimate resistance against negative bending
- ultimate resistance against positive bending moments taking interaction of shear into account
- ultimate resistance against negative bending moments taking interaction of shear into account
- ultimate resistance against shear
- ultimate resistance against torsional buckling

**Ultimate limit state of the composite section referring to EC 4:**
- ultimate resistance against positive bending
- ultimate resistance against shear
- ultimate resistance against negative bending moments taking interaction of shear into account
- bonding strength of the shear connectors, number of shear connectors
- ultimate resistance against torsional buckling
Serviceability limit state referring to EC 4:
- stress analysis
- crack width limitation and check of the minimal reinforcement
- deflection check

To complete the calculation, the shear bearing resistance of the shear connectors is derived. If studs are applied, the number of shear connectors is estimated. The distribution of the studs has to be done by the user, considering the geometrical parameters given by the program.

9.4 The results

Fig. 9-9: Finish calculation

After the run of the calculation model, which is monitored by an output on the screen, the final dialog pops up on the screen (Fig. 9-9). The estimated height of the plate section, respectively the rolled profile out of the producer list is given in the dialog. If the check against lateral torsional buckling is not satisfied, it will be remarked, that further stiffeners have to be added to the structure.

The results of each iteration step of the calculation are written into a similar named output file with the extension (.OUT). The last and relevant iteration step is summarised into a similar named data file with the extension (.RTF). This output file can be directly opened in the MS Notepad, added to the program.

If the user obviously made any input mistakes, an error message will be presented and an error log is written in the output file.

The output is arranged such as the user is able to follow the result information easily. The output to each defined design point is structured successively:
- the calculated cross section
- the proper design loads
- classification of the steel beam
- loads after moment distribution
- design checks of the steel beam in the ultimate limit state
- design checks of the composite beam in the ultimate limit state at $t = 0$
- design checks of the composite beam in the ultimate limit state at $t = \infty$
- stress analysis

If the user excluded a construction situation, the output related to the situation will be dropped in the output file. If a design check is irrelevant for a certain design point, the check will be skipped and no information will be found in the output file.
Additionally the following information are added at the end of the output file:
- minimal reinforcement design
- crack width design
- deflection check
- information about the aimed super-elevation of the structure during construction
- shear bearing capacity of the shear connectors
- number of shear connectors and geometrical parameters for their distribution between the critical cuttings
- distance of the lateral stiffeners in the construction phases
- distance of the inner transversal stiffeners in service

To complete the pre-design of a structure, the following items have to be carried out retrospectively:
- distribution of the shear connectors
- check of the contour area of the shear connector
- vibration behaviour of the structure
- fatigue design

The check formats of the EC are included in the output file
INSTALL.bat installs the program in a former specified directory on the computer.
10 DESIGN GUIDE

The Design Guide including the tender drawings has been prepared as an supplement document including several variants of composite bridges and the software with detailed examples.