Summary of Results for the Project INSUSHELL

Report-No. 237/2009

of the Institute of Structural Concrete (IMB), RWTH Aachen University

Subject: Self-supporting sandwich facade made of Textile Reinforced Concrete and polyurethane rigid foams for the INNOTEX-building of the Institut für Textiltechnik

The project was co-funded by the LIFE06 framework programme of the European Commission.

Reported by: Prof. Dr.-Ing. J. Hegger
Dr.-Ing. N. Will
Dipl.-Ing. M. Horstmann

Project-No.: F-2005-023

Date: Aachen, den 10.07.2009

This report comprehends 25 pages of text.
# Table of Contents

**TABLE OF CONTENTS**

<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>PUBLICATION</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>GENERAL REMARKS</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>DESIGN OF THE SANDWICH CONSTRUCTION</td>
<td>2</td>
</tr>
<tr>
<td>3.1</td>
<td>General</td>
<td>2</td>
</tr>
<tr>
<td>3.2</td>
<td>Geometry of panels</td>
<td>2</td>
</tr>
<tr>
<td>3.3</td>
<td>Composite action and connecting devices</td>
<td>3</td>
</tr>
<tr>
<td>3.4</td>
<td>Fixing system (IGF Zimmermann)</td>
<td>5</td>
</tr>
<tr>
<td>3.5</td>
<td>Structural-physical construction (IGF Zimmermann)</td>
<td>7</td>
</tr>
<tr>
<td>3.5.1</td>
<td>Heat insulation</td>
<td>7</td>
</tr>
<tr>
<td>3.5.2</td>
<td>Moisture proofing</td>
<td>7</td>
</tr>
<tr>
<td>3.5.3</td>
<td>Sound insulation</td>
<td>7</td>
</tr>
<tr>
<td>3.5.4</td>
<td>Fire resistance</td>
<td>8</td>
</tr>
<tr>
<td>4</td>
<td>STRUCTURAL DESIGN OF THE SANDWICH FACADE</td>
<td>8</td>
</tr>
<tr>
<td>4.1</td>
<td>Applied materials</td>
<td>8</td>
</tr>
<tr>
<td>4.1.1</td>
<td>Properties of GFRC</td>
<td>8</td>
</tr>
<tr>
<td>4.1.2</td>
<td>Chopped fibers</td>
<td>9</td>
</tr>
<tr>
<td>4.1.3</td>
<td>Textile reinforcement</td>
<td>9</td>
</tr>
<tr>
<td>4.1.4</td>
<td>Polyurethane rigid foam</td>
<td>12</td>
</tr>
<tr>
<td>4.1.5</td>
<td>Connecting devices (diagonal steel tie and pin connectors)</td>
<td>13</td>
</tr>
<tr>
<td>4.1.6</td>
<td>Fixing devices</td>
<td>13</td>
</tr>
<tr>
<td>4.2</td>
<td>Structural design of the sandwich action</td>
<td>13</td>
</tr>
<tr>
<td>4.2.1</td>
<td>General remarks</td>
<td>13</td>
</tr>
<tr>
<td>4.2.2</td>
<td>Design loads</td>
<td>14</td>
</tr>
<tr>
<td>4.2.3</td>
<td>Required textile reinforcement in the concrete facings</td>
<td>15</td>
</tr>
<tr>
<td>4.2.4</td>
<td>Structural analysis of the connecting devices</td>
<td>15</td>
</tr>
<tr>
<td>4.2.5</td>
<td>Structural analysis of the foam core</td>
<td>17</td>
</tr>
<tr>
<td>5</td>
<td>EXPERIMENTAL VERIFICATION OF THE LOAD-BEARING CAPACITY OF THE SANDWICH PANELS</td>
<td>17</td>
</tr>
</tbody>
</table>
5.1 Pullout capacity of connecting devices.................................17
5.2 Bearing capacity of the fixing devices bolted to the panel (mounting plates).................................................................19
5.3 Bending tests on sandwich panels ........................................21
6 PRODUCTION (DURAPACT) AND MOUNTING OF THE SANDWICH PANELS.................................................................22
7 LITERATURE AND APPLIED DOCUMENTS..........................25
8 CONTACT PERSONS ....................................................................27
1 Publication

The publication of the content of this document – even partwise – by project partners and third parties has to be agreed with the Institute of Structural Concrete before publishing. Moreover, project partners and third parties involved in the project should obey the following guidelines:

The informations provided by this document should only be used for the intended purposes. It is not allowed to publish or commercially use the given information without the written consent of the Institute of Structural Concrete. The information provided in the frame of this document and within the project INSUSHELL shall not be given to other parties and should be dealt as strictly confidentially.

2 General remarks

The “Bau- und Liegenschaftsbetrieb NRW Aachen (BLB)” has erected a new office and laboratory building (INNOTEX) for the Institut für Textiltechnik (ITA) of RWTH Aachen University (Fig. 1). Parts of the façade have been designed as a sandwich construction consisting of Textile Reinforced Concrete (TRC) in combination with short fibers (GFRC = Glass Fiber Reinforced Concrete), a polyurethane rigid foam as well as connecting and fixing devices made of stainless steel. Due to the fact, that neither TRC nor the connecting and fixing devices have a general approval of the Deutsche Institut für Bautechnik (DIBt), an individual approval of the regional building authority was applied which was issued on December, 12th, 2008 [U 7]. The present report is summarizing the main results of the project work performed by the Institute of Structural Concrete and relates them to the work of the other partners.

The sandwich façade covering about 590 m² of the overall façade area was developed in cooperation of the Institute of Structural Concrete (IMB) and the façade engineers IGF Zimmermann, Mülheim, as a self-supporting sandwich construction. The textile reinforcement was provided by the project-leading Institut für Textiltechnik (ita). The sandwich panels were produced by the company Durapact, Düsseldorf, which has a huge experience in the production of GRFC-products. The general planners of the building, the architecture office Carpus und Partner, Aachen, integrated the sandwich panels in the overall façade construction and concept.
3 Design of the sandwich construction

3.1 General

The following chapters briefly describe the design of the developed sandwich construction regarding the geometry, building physics (IGF Zimmermann) and in particular the design of the static and load-bearing system. More detailed surveys of the production technique, durability aspects as well as building physics can be found in annexes [U 1] to [U 6].

3.2 Geometry of panels

The sandwich façade is assembled of 216 single elements with 12 different geometries. The majority of the façade was assembled of two main types. For the northern and southern façade, the maximum element dimensions are 2,975 m x 0,975 m (type P1, [U 1]), for the eastern side 3,425 m x 0,975 m (type P11, Fig. 2). Also in the areas of adjacent building parts and openings, special element types were required (types P2 - P12, [U 1]) whose dimensions range within the aforementioned panel sizes. The relating element plans can be found in [U 1].
The sandwich panels consist in the mean area of two 15 mm thick TRC-facings and a 150 mm thick polyurethane rigid foam (Fig. 2). To strengthen the composite action and the shear transfer, notches have been located at the upper and lower surface of the core with a distance of 50 mm. At the edges of the panel, the thickness of the concrete facings is enlarged to 40 mm to provide sufficient space for the sealings, connecting and fixing devices. The transition of the thickness of the facings from mean to edge areas is homogenized by a bezel to avoid stress concentration as well as concrete pryout of the connecting devices which in part are located close to the edges (diagonal steel tie, Fig. 2).

3.3 Composite action and connecting devices

The load-bearing capacity of the panels is determined by the composite action of the core and concrete facings following the theory of the elastic composite (s. [L 1],[L 4],[L 5]). Due to visco-elastic creep deformations of the core material evoked by dead loads and the fact that a full composite action only can be achieved by a capable joint between core and fac-
ings connecting devices were required to ensure a durable sandwich action and limited time-
dependent deformations. As common systems of e.g. Halfen and Pfeifer require a minimum
thickness of the concrete for a proper anchorage performance, suitable connecting devices
had to be developed meeting the requirements of thin-walled TRC-facings. Pin-connectors
with a hook-like anchorage (hook length 5 x Ø, hook angle 90°) made of stainless steel
(1.4401/1.4404, Ø = 4 mm, S460) are applied (Fig. 3).
The pin connectors supply a high axial stiffness are mainly subjected to normal forces. The
at the same time low bending and shear resistance of the connectors keep constraint forces
in the connectors and TRC-facings due to constricted temperature and shrinkage deforma-
tions of inner and outer concrete facing at a low level.

The dead load of the outer facing of about 160 kg is transfered by two diagonal steel ties
which are placed in the centroid of the front wythe in cooperation with a foam core (truss).
The tensile force in the diagonal steel tie is directly transmitted to the upper horizontal sup-
port (Fig. 3, Fig. 4).
In the thickened edging frames, pin connectors are located in the third points to absorb the
peeling and normal stresses in the jonts evoked by temperature and shrinkage deformations
as well as by wind suction (Fig. 4). The load-bearing capacity of the connecting devices is
dimensioned that way that the dead load of the outer concrete wythe can be taken over and a
collapse respectively drop of the outer facing is prevented.
3.4 Fixing system (IGF Zimmermann)

Each element was fixed separately to a steel post which is connected to the load-bearing steel frame of the building (Fig. 5). The vertical loads are transferred by the inner concrete facing to two consoles (Fig. 4, Fig. 5). For the transfer of horizontal loads the panels are fixed at 4 points to the steel post. The applied fixing system allows for the mounting and removal of single elements without affecting neighbour elements. To solve this issue, the horizontal supports consist of trapezoid-shaped stainless steel mounting plates and a V-shaped shutter plate (Fig. 5) which is mounted to the post and is relocatable in vertical direction due to its slotted hole. At the moment of assembly, the façade panels were placed on the vertical support, the shutter plates were lowered and fixed with screws so that the shutters press the mounting plates against the supporting steel profile. After mounting an element row the sealing was applied which also restricts unauthorized loosening of the screws from the outside of the façade.
Fig. 5: Horizontal section of fixing area (top) as well as pictures of the applied fixing devices (vertical and horizontal supports) for elements near gates-doors (bottom)
3.5 Structural-physical construction (IGF Zimmermann)

3.5.1 Heat insulation

The heat insulation of the panels is designed according to the requirements of the german Energie-Einsparverordnung – EnEV 2007 [L 2] and DIN 4108 [L 3] as well as to avoid condensation of water in the insulation. The difference of surface and air temperature inside the building was designed to be smaller than 5 K.

The transmission loss of the panels was conservatively determined by the calculation of the heat transmission depending on the proportions of areas of the used materials. For the mid-parts of the panels with an area of 2.24 m² (panel type P1) with a polyurethane core type B2 50 and a heat transfer coefficient of 0.025 W/mK, a heat transfer coefficient of $U_W < 0.18 \text{ W/m}^2\text{K}$ was calculated according to EN 12667 [L 4]. If the reduced insulation thickness of the applied mineral fibers with a heat transfer coefficient of 0.035 W/mK at the panel edges as well as the connecting devices are taken into account, a total heat transfer coefficient of $U_W < 0.3 \text{ W/m}^2\text{K}$ was determined which is far below the required values of EnEV and DIN 4108. The airtightness was ensured by applying airtight facings as well as permanent, extrudable silicon sealings at the inside joints of panels.

3.5.2 Moisture proofing

For the moisture proofing the tightness against penetration of rain was achieved by sealing the outer element joints with EPDM- or silicon-sealings. The applied profiles were located in cast grooves overlapping the particular lower horizontal sealing and underlapping the particular upper horizontal sealing. The assembly of panels and the placing of outer sealings was alternately executed.

For the verification of the applicability, the chosen solution for the moisture proofing was successfully tested in an approved test facility.

3.5.3 Sound insulation

The sound reduction index of the panels meets the requirements of exterior building elements in common traffic noise zones according to DIN 4109 [L 5]. In compliance to report 5249 of Bayer MaterialScience AG Leverkusen [L 1], a sound reduction index of
R_w = 43 dB has been determined for a section consisting of facings with 15 mm thickness and a polyurethane core with a density of 30 kg/m³ and a thickness of 130 mm. For the present conditions, a design value of R'_w = 39 dB can be derived under consideration of a derivative measure of 2 dB. Thus, the panels can be applied in noise level areas IV with a noise level of 66 bis 70 dB (A) respectively for residential and office buildings at streets in city and residential areas.

### 3.5.4 Fire resistance

The facings of the sandwich panels are categorized as incombustible materials according to DIN EN 13501 [L 6] as well as in compliance to a performed SBI-test according to DIN EN 13823 [L 7]. The foam core itself has to be classified in A2/B respectively hardly inflammable regarding DIN 4102 B2 [L 8]. A test certificate of the MPA Erwitte [U 3], Germany, is accounting for building material class of B1 according to DIN 4102 for the whole sandwich section. The outer sealing profiles match class B2. In the area of the element joints the polyurethane was replaced by a non-combustible mineral fiber insulation with a melting point of 1000 °C (Fig. 5).

### 4 Structural design of the sandwich façade

#### 4.1 Applied materials

##### 4.1.1 Properties of GFRC

The mechanical properties of the applied GFRC were determined at the Institute for Building Materials Research (ibac) at RWTH Aachen University. The compressive strength as well as the bending strength was determined according to DIN EN 196-1 [L 9]. All properties have been determined at the age of 28 days (Table 1). As expected, the bending strength was significantly raised by the short fibers (1 % per weight) which are added to the concrete in the premix method whereas the influence on the axial tensile strength was low.

<table>
<thead>
<tr>
<th>GFRC</th>
<th>( f_{cm} ) (^{1)})</th>
<th>( f_{ck} ) (^{2)})</th>
<th>( f_{cm} )</th>
<th>( f_{ck} )</th>
<th>( f_{ck,f,r,k} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modified short fiber concrete, Durapact</td>
<td>81,9</td>
<td>72,1</td>
<td>2,1</td>
<td>1,6</td>
<td>10,5</td>
</tr>
</tbody>
</table>

\(^{1)}\) determined with standardized prisms 40x40x160 mm³

\(^{2)}\) \( f_{ck} = f_{cm} (1 - 2,18 \cdot \sigma) \)

Shrinkage \( \varepsilon_S \):

\( \varepsilon_{S,28d} = 1,13 \text{ mm/m}; \varepsilon_{S,\infty} \sim 1,5 \text{ mm/m} \)
4.1.2 Chopped fibers

For the facings, a GFRC was applied for which an integral chopped AR-glass-fiber of the type AR-Force 2 (Durapact) with a length of 12 mm was added (premix method). In addition, a fiber mat with a fiber length of 50 mm and an area weight of 120 g/m² was located near the surfaces of the facings. The short fibers reduce the micro- and macro-cracking of the matrix due to shrinkage or loads as well as the crack spacing in serviceability limit state. A contribution of the short fibers to the load-bearing capacity in ultimate limit state was negelected.

4.1.3 Textile reinforcement

4.1.3.1 Geometrical design and properties

Two different types of fabrics made of AR-glass have been produced at the Institut für Textiltechnik. For the reinforcement of the 15 mm thick parts of the facings an uncoated fabric was chosen (Fig. 6, left, successively called fabric 1), for the thickened frame at the panel edges an epoxy-coated fabric (coated by von Roll, Düren, Germany) with a higher load-bearing capacity was used (Fig. 6, right, successively called fabric 2).

In Table 2 und Fig. 7 the geometrical properties of the two applied fabrics as well as the particular characteristic tensile strengths derived from tensile tests on concrete members are facing each other.
Table 2: Geometrical design of the applied fabrics

<table>
<thead>
<tr>
<th>Direction</th>
<th>Roving</th>
<th>Yarn count</th>
<th>$f_{ck}$</th>
<th>Cross sectional area of reinforcement $a_t$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>[N/mm$^2$]</td>
<td>[mm$^2$/m]</td>
</tr>
<tr>
<td>0°</td>
<td>Vetrotex, AR-glass, 5323, 1200tex</td>
<td>2x1200</td>
<td>537</td>
<td>71,5</td>
</tr>
<tr>
<td>90°</td>
<td>Vetrotex, AR-glass, 5323, 1200tex</td>
<td>1200</td>
<td>511</td>
<td>53,3</td>
</tr>
<tr>
<td>0°</td>
<td>Vetrotex, AR-glass, 5323, 2400tex</td>
<td>2400</td>
<td>1170</td>
<td>108</td>
</tr>
</tbody>
</table>

Fig. 7: Comparison of stress-strain curves of fabric 1 (left) and fabric 2 (right) in 0°-direction obtained from tensile tests on concrete members

4.1.3.2 Durability of the AR-glass fabrics in the alkaline concrete matrix

The durability of AR-glass fibers in the alkaline milieu of the concrete is primarily depending on the environmental conditions and the mixture of the concrete. For uncoated fibers in a concrete with a high cement portion a loss of tensile strength up to 40 % has been determined [L 3],[L 11]. The GFRC-mixture commonly used by Durapact for the production of corrugated panels is distinguished on the one side by only moderate strength losses below 15 % and on the other side by significantly larger shrinkage values compared to normal concrete [L 1]. For the production of the present façade panels, a modified mixture with lower shrinkage values (Table 1) was applied. For this mixture, the long-term loss of strength of AR-glass fibers was determined by the Institute of Building Materials Research in four-point bending tests on accelerated aged, notched bending members (Fig. 8).
The results of the performed tests on accelerated aged specimen proved negligible strength losses of the reinforcement in the applied concrete. Since the pH-value of 11.6 is within the critical area for AR-glass fibers, nevertheless a long-term loss of strength had to be assumed.

The long-term strength loss was conservatively assessed with the model of the SFB 532 [L 3]. At this, for the outer facings the climate of Aachen (temperature, relative humidity, condensation/rainfall) and for the inner facings an indoor climate of 23 °C and 65 % relative humidity was taken as a basis for the calculations with the model. In addition, the time-dependent progress of the carbonation front was determined for naturally weathered specimen and sample stored under laboratory conditions. It was assumed, that the progression of the carbonation front to the position of the reinforcement and the accompanying lowering of the pH-value leads to a stop of the corrosion process. By means of these assumptions and based on parameters for the better known concrete mixture PZ0899-01 of the SFB 532, the curves in Fig. 9 have been calculated with the model.

---

Fig. 9: Modelling of the long-term strength loss for the climate in Aachen (left) and for 23 °C/65% relative humidity (right) (assumptions for model: concrete: PZ-0899-01 (SFB 532); reinforcement: 2400 tex AR-glass uncoated with consideration of carbonation; no consideration of permanent loads) (ibac, [U 6])
For a lifetime of 50 years and the location Aachen an assumed maximal strength loss of 25 % for an uncoated fabric in the outer facings and of 10 % in the inner facings was calculated.

4.1.4 Polyurethane rigid foam

4.1.4.1 Mechanical properties

The applied rigid foam cores were produced by puren GmbH, Überlingen. By cutting the cores out of foam slab stocks, fine particles and dust are covering the surfaces which lead to a low bond strength of glued joints of facings and core. Own investigations in [L 1] proved that a direct bond of foam and concrete which can be obtained by pressing a notched core into the still fresh concrete provides a higher load-bearing capacity of the joint and the sandwich panel.

Polyurethane is an anisotropic material which means that the material properties e.g. Young’s modulus and material strength are depending on the load-direction. Moreover, Hooke’s laws for the coherence of Poisson ratio, Young’s modulus and shear modulus are not valid and the properties scatter according to the variation of density. The mean values of the applied polyurethane core are listed in Table 3.

Table 3: Material properties of the applied polyurethane core (puren GmbH)

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit</th>
<th>RG 50 B2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>[kg/m³]</td>
<td>47 – 52</td>
</tr>
<tr>
<td>Compressive strength</td>
<td>[N/mm²]</td>
<td>0,35 – 0,40 / 0,35¹</td>
</tr>
<tr>
<td>Shear strength</td>
<td>[N/mm²]</td>
<td>0,2 – 0,25</td>
</tr>
<tr>
<td>Transverse tensile strength</td>
<td>[N/mm²]</td>
<td>0,35-0,45 / 0,225²</td>
</tr>
<tr>
<td>Young’s modulus (compression)</td>
<td>[N/mm²]</td>
<td>10 – 13 / 10 - 16¹</td>
</tr>
<tr>
<td>Young’s modulus (transverse tension)</td>
<td>[N/mm²]</td>
<td>15-17,5 / 14 - 16¹</td>
</tr>
<tr>
<td>Shear modulus G</td>
<td>[N/mm²]</td>
<td>3,0-4,0</td>
</tr>
<tr>
<td>Heat conductivity λ</td>
<td>[W/mK]</td>
<td>0,022 – 0,025</td>
</tr>
<tr>
<td>Degree of closed cells</td>
<td>[Vol-%]</td>
<td>90 – 95</td>
</tr>
<tr>
<td>Fire behavior (DIN 4102)</td>
<td>[-]</td>
<td>B2</td>
</tr>
<tr>
<td>Application temperature</td>
<td>[°C]</td>
<td>-80 to +120</td>
</tr>
</tbody>
</table>

¹ derived from tests of IMB² mean value of tests of ibac
4.1.5 Connecting devices (diagonal steel tie and pin connectors)

As connecting devices bended wires with diameter of $\varnothing = 4 \text{ mm}$ and consisting of stainless and high-strength steel (material number 1.4401/1.4404, S460) was used [L 10]. The design values are listed in Table 4.

Table 4: Calculation and design values for stainless steel, strength category S460

<table>
<thead>
<tr>
<th>Young’s modulus [N/mm²]</th>
<th>Shear modulus [N/mm²]</th>
<th>$f_{y,k}$ [N/mm²]</th>
<th>$f_{u,k}$ [N/mm²]</th>
<th>$f_{y,d}$ [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>170000/190000$^{1)}$</td>
<td>65400/76900$^{1)}$</td>
<td>460</td>
<td>600</td>
<td>418.2</td>
</tr>
</tbody>
</table>

$^{1)}$ favorable/unfavorable influence on calculation

4.1.6 Fixing devices

The fixing system of the panels was developed by IGF Zimmermann, Mülheim and structurally analyzed by the engineer’s office Kempen Krause, Aachen [U 4]. The experimental proof of the load-bearing capacity of the connection between mounting plates and concrete facing was performed by the Institute of Structural Concrete.

The anchor and shutter plates were laser-cutted out of 6 mm thick flat rolled stainless steel with the material number 1.4462 and the strength class S460 (Table 4). For the connection with the inner concrete facing, screws of the class A4-70 were used [L 10].

4.2 Structural design of the sandwich action

4.2.1 General remarks

The structural design of the panels was accomplished by comparative calculations with a linear-elastic finite element program and the theory of the elastic composite taking the cracking of the concrete facings into account [L 12],[L 13]. The structural design of the concrete facings for normal, bending and shear forces was executed with the design models of Voss [L 14]. The load-bearing capacity of the connectors was verified considering tensile and buckling failure.
4.2.2 Design loads

In addition to the common direct design loads for facades like dead loads and wind pressure/suction [L 15] also indirect stresses due to hygrothermic influences have to be considered. Hygrothermic effects due to temperature and moisture changes are interacting and depend on production technique and atmospheric conditions causing deformations of the concrete facings. If these deformations are restricted by the foam core or the connectors constraint forces have to be considered in the design of facings and connectors.

For common sandwich panels produced of structural concrete, limit values for temperature and shrinkage stresses are assessed in [L 18],[L 19]. Since the used fine-grained concrete has larger shrinkage values compared to ordinary concrete leading to significant stresses and deformations, the hygrothermic behavior was determined with model tests. The shrinkage behavior of the inner concrete facing was simulated by prism samples stored in a standardized indoor climate of 60 % relative humidity and 20 °C (Fig. 10) whereas the climatic exposure of the outer facing was simulated with prisms stored in alternating conditions. They were stored the first 28 days in the same indoor climate followed by a rain period of 3 days at 95 % relative humidity/20 °C and a subsequent heating period of 4 days at a temperature of 60 °C. This sequence was repeated and a maximum strain difference between the prisms under different storage conditions respectively between inner and outer concrete facing of $\Delta \varepsilon_{\text{max}} \sim 0.7/0.89 \text{ mm/m}$ was determined after 70/200 days of storage covering the hygrothermic behaviour in a simplified and integral approach. This simplified maximum strain difference was converted in an equivalent temperature load for the design calculation.

![Hygrothermic strain behavior of concrete prisms stored for 79 days in different climatic conditions (ibac) [U 5]](image)
4.2.3 Required textile reinforcement in the concrete facings

The required textile reinforcement is determined on the basis of knowledge of the Collaborative Research Center 532 at RWTH Aachen University and in particular with the models of Voss [L 14]. Fig. 11 shows the determined textile reinforcement for panel type P11 with the largest dimensions.

![Reinforcement plan for panel type P11](image)

Fig. 11: Reinforcement plan for panel type P11

4.2.4 Structural analysis of the connecting devices

Due to the high stiffness in axial direction the connectors absorb forces and peeling stresses normal to the joint of core which are evoked by constraint forces. For the structural analysis, the stresses in the connectors due to tension and bending were analyzed according to DIN 18800 [L 16] and the maximum anchorage forces were determined for the experimental proof of the load-bearing capacity. Fig. 12 shows the maximum stress utilization of the connectors in the decisive combination of loads.
The anchorage of the connecting devices is accomplished by hooks bent with an angle of 90° and a leg length of $L_{\text{Hook}} \sim 5 \times \varnothing$. The bending radius was chosen as $2.5 \times \varnothing$ to avoid brittle failure of the steel. In Fig. 13, the anchorage forces according to the decisive combination of DIN 1045-1 [L 17] are displayed. The experimental evidence of the load-bearing capacity of the particular anchorage details can be found in chapter 5.1.
4.2.5 Structural analysis of the foam core

The foam core and the joint quality to the facings determine the degree of composite action. In case of a good bond quality, the foam takes over a large portion of the acting shear forces. The load-bearing capacity of the joint between concrete and foam was ensured by (a) using a notched foam surface perpendicular to the direction of action of the shear forces and (b) by connecting devices. Although the failure of the member is announced by large deformations, typical load-deflection-curves in [L 1] presented themselves as bilinear without a significant load plateau. Thus, a large partial safety factor similar to unreinforced concrete was used for the verification of the load-bearing capacity of the core. Fig. 14 presents the principal tensile stresses in the decisive combination according to DIN 1045-1.

Fig. 14: Principal tensile design stresses $\sigma_{1,Ed}$ according to DIN 1045-1 [L 17], [N/mm²]

The comparison of calculative design stresses to the foam’s design resistance leads to a utilization ratio of 89 % ($\sigma_{1,Ed,max}/\sigma_{Rd} \sim 0.89$).

5 Experimental Verification of the load-bearing capacity of the sandwich panels

5.1 Pullout capacity of connecting devices

Four different anchorage details of the pin connectors and the diagonal steel ties were tested (Fig. 15). The hooks of the connectors were threaded through the upper reinforcement layer (fabric 2, Table 2) of the surrounding concrete frames at the panel edges to ensure a ductile
pullout behaviour. The tests on detail 3a were deliberately stopped at a load of 4 kN to determine the residual carrying capacity of the associated detail 3b.

![Fig. 15: Pullout-tests on anchorage details of pin connector and diagonal steel tie](image)

The specimen were fixed in a manner that a shallow inclined pryout cone with an angle of $\alpha = 15^\circ$ had been possible without being influenced by the fixing clamps.

The arithmetic mean, the variation coefficient (oriented to Gaussian distribution), the 5%-fractile derived from the logarithmic distribution, design values (partial safety factor $\gamma_c = 1,8$) as well as the comparison with the maximum tensile forces of Fig. 13 are listed in Table 5.

<table>
<thead>
<tr>
<th>Detail-No.</th>
<th>Arith. mean $F_{u,m}$ [kN]</th>
<th>Variation coefficient $V$ [-]</th>
<th>5%-fractile $F_{Rk}$ [kN]</th>
<th>Resistance ($\gamma_c = 1,8$)</th>
<th>Design load $F_{Ed}$ [kN]</th>
<th>$F_{Ed}/F_{Rd}$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2,02</td>
<td>0,12</td>
<td>1,35</td>
<td>0,75</td>
<td>0,45 $^{(1)}$</td>
<td>0,60 $\checkmark$</td>
</tr>
<tr>
<td>2</td>
<td>2,09</td>
<td>0,14</td>
<td>1,34</td>
<td>0,74</td>
<td>0,17 $^{(2)}$</td>
<td>0,23 $\checkmark$</td>
</tr>
<tr>
<td>3a</td>
<td>4,04</td>
<td>-</td>
<td>-</td>
<td>$3,9/1,8 = 2,17^{(1)}$</td>
<td>0,76 $^{(3)}$</td>
<td>0,35 $\checkmark$</td>
</tr>
<tr>
<td>3b</td>
<td>1,79</td>
<td>0,06</td>
<td>1,27</td>
<td>0,7</td>
<td>0,17 $^{(2)}$</td>
<td>0,24 $\checkmark$</td>
</tr>
<tr>
<td>4</td>
<td>4,17</td>
<td>0,29</td>
<td>2,63</td>
<td>1,46</td>
<td>0,76 $^{(3)}$</td>
<td>0,52 $\checkmark$</td>
</tr>
</tbody>
</table>

$^{(1)}$ lowest value of 3 prematurely stopped tests, $^{(2)}$ s. Fig. 13, $^{(3)}$ assumption that only diagonal steel tie bears dead load of front facing
For the deliberately stopped tests on diagonal steel ties (detail 3a) no fractile was determined. For the verification a comparative value of the lowest test result and a partial safety factor of $\gamma_c = 1.8$ was applied. In none of the tests a maximum utilization ratio of 60% was exceeded.

5.2 Bearing capacity of the fixing devices bolted to the panel (mounting plates)

The geometry of the specimen as well as the test setup for the determination of the load-bearing capacity of the mounting plates (flat rolled steel: $t = 6\, \text{mm}$, 1.4462, S460; screws M8, A4-70) bolted to the inner concrete facing is presented in Fig. 16.

At each specimen two tests were performed with alternating load direction (wind pressure and suction). In the first load position the mounting plate was positioned beneath the facing
which equals the conditions under wind pressure. The specimen was supported by two line bearings. The load-near bearing also took over torsional forces resulting from the excentric loading of the mounting plate.

The specimen was turned upside down for the second load position and supported as described above (Fig. 16, right). In all performed tests the load was introduced at the vertex of the mounting plate which equals the most awkward lever length.

In load position 1 generally a shear or a punching failure was observed whereas the load-bearing capacity of specimen in position 2 always was determined by a shear failure (Fig. 17).

![Fig. 17: Pictures of typical failure modes for load position 1 (left) and load position 2 (right)](image)

The results of all performed tests are listed and analyzed in Table 6. In the analysis, the influence of the proximity of the load to the support was considered in accordance to DIN 1045-1. Assuming a partial safety factor for concrete failure of $\gamma_c = 1.56$, an adequate structural safety in comparison to the design loads calculated in the finite element simulation (Table 6) was determined.

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Mean $F_{u,m}$ [kN]</th>
<th>Variation coefficient $V$ [-]</th>
<th>5%-fractile $F_{Rk}$ [kN]</th>
<th>Design load $F_{Ed}$ [kN]</th>
<th>$F_{Ed}/F_{Rk}$ ($\gamma_c = 1.56$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load position 1</td>
<td>4</td>
<td>3.74</td>
<td>0.18</td>
<td>2.57</td>
<td>0.87</td>
</tr>
<tr>
<td>Load position 2</td>
<td>4</td>
<td>5.95</td>
<td>0.46</td>
<td>2.96</td>
<td>1.24</td>
</tr>
</tbody>
</table>
5.3 Bending tests on sandwich panels

For a total of 10 sandwich panels the load-bearing behavior was investigated in four-point-bending tests under static and cyclic loading. The load-deflection curves and ultimate loads of three selected tests are presented in Fig. 18 and Table 7.

![Fig. 18: Load-deflection curves of selected four-point bending tests on element type P11 as well as influence of joint quality on shear capacity of the panel](image)

Table 7: Results of selected bending tests on element type P11

<table>
<thead>
<tr>
<th>Name</th>
<th>Age [d]</th>
<th>Failure load $F_U$ [kN/m]</th>
<th>Deformation $w_U$ [mm]</th>
<th>Max. Shear force $Q_u$ (^{1)}) [kN/m]</th>
<th>Failure moment $M_u$ (^{1)}) [kNm/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>SW1</td>
<td>95</td>
<td>31,0</td>
<td>58,2</td>
<td>17,4</td>
<td>17,1</td>
</tr>
<tr>
<td>SW2</td>
<td>109</td>
<td>13,2</td>
<td>26,8</td>
<td>8,7</td>
<td>9,2</td>
</tr>
<tr>
<td>SW3</td>
<td>7</td>
<td>17,0</td>
<td>28,5</td>
<td>10,4</td>
<td>10,1</td>
</tr>
</tbody>
</table>

\(^{1)}\) including dead load of panel

All tested panels failed by shear rupture of the core. The load-bearing behaviour was primarily determined by the quality of the joint between concrete and core. Panel SW1 with perfect joint quality failed at a high load level. After shear rupture of the core the residual load-bearing capacity was still about 50% of the maximum value.
Panels SW2 and SW3 failed at significant lower load levels since in saw cuts large areas with poor joint qualities have been determined. Thus, lower stiffnesses and shear capacities are obtained (Fig. 19). No significant influence of the cyclic loading on the load-bearing capacity became evident.

![Joint of notched and mortared core and outer facing with areas of poor bond quality (SW2)](image)

However, the performed tests also show the activity of secondary bearing capacities resulting from the positive locking of foam core and surrounding concrete frame at the panel edges as well as friction effects which evolve from the action of the connecting devices. In addition to the reduced primary laminar composite action both panels were able to accomplish the $\gamma$-fold design level in ultimate limit state. The cracking of the facings was effectively suppressed by the cooperation of short fibers and textile fabrics for the whole load range. At service load level, no or only few cracks of marginal crack width were observed.

### 6 Production (Durapact) and Mounting of the sandwich panels

The sandwich panels were produced by Durapact in a discontinuous two-step method. Contrary to the common production methods for ordinary sandwich panels made of structural concrete (positive / negative method) the used technique results in two very plane fairfaced concrete surfaces.

First, the 15 mm thick layer of the outer layer was produced by alternately placing GFRC (Fig. 20) and two layers of fabric 1 (Fig. 11). Afterward, the notches of one foam surface were filled with GFRC and the prepared foam core was pressed defined into the fresh cast facing using a gauge. As the surrounding frame at the panel edges was cast and reinforced, the connectors were placed and the first step of the production process was finished.
On the following day, the 15 mm thick layer of the inner facing was analogically cast. The half-finished panel of the day before was turned upside down within its formwork, the notches of the free core side were filled with GFRC again and then lowered into the fresh concrete until a defined overall height was reached. By filling the surrounding frame of the inner facing with concrete and placing the reinforcement, the second step of the production process was finished. After curing for 1-2 days in the formwork, the panels were removed from the form, wrapped in a plastic sheet for further curing and ready for dispatch. The quality of the production process was examined by an external quality control in the scope of the individual approval process. The mounting plates were pre-assembled and the panels were furnished with an Ü-sign. More detailed information on the production of the elements can be found in [U 2].

On the construction site, the panels were placed with a vacuum lifter on the vertical supports and then fixed with the mounting plates/shutter-combination (Fig. 21, lower left). After assembling an element row the outer sealing is inserted whereas the inner silicon-sealing is
applied after completing a whole section/field (Fig. 21, upper right). The erection of the façade was completed at the end of Mai 2009 (Fig. 21, lower right).

Fig. 21: Inside view on vertical und horizontal element junction with vertical support before and after applying the inner sealing (upper left and right); outer sealing and horizontal supports for elements next to a gate as well as completed facade (seen from upper left to lower right)
7 Literature and applied documents

Literature


Summary of results of project INSUSHELL


Documents


[U 3] Prüfzeugnis Nr. 230006496 zum Nachweis der Schwerentflammbarkeit, MPA Erwitte, 23.05.2008.

Summary of results of project INSUSHELL


8 Contact persons

Dr.-Ing. N. Will Dipl.-Ing. M. Horstmann
Tel.: 0241/80-25171 Tel.: 0241/80-25098
Fax: 0241/80-22335 Fax: 0241/80-22616
nwill@imb.rwth-aachen.de mhorstmann@imb.rwth-aachen.de
Mies-van-der-Rohe-Str. 1 Mies-van-der-Rohe-Str. 1
52074 Aachen 52074 Aachen