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EDITORIAL

This issue contains a large spectrum of topics, again. There are four papers on non-destructive testing of materials of various kinds. One paper deals with the determination of concrete properties at early age, another uses ultrasound techniques to measure the state of large timber compound elements, and a third deals with the measuring of the thickness of a polymer concrete layer on a mature concrete substrate. Scanning impact-echo techniques are used for crack depth determination.

Shear fracture of concrete is an old but still an attractive topic for research. A survey is given on shear in concrete on the basis of fracture mechanics. Durability of concrete is treated in another paper with the emphasis on cracking. Curing of concrete is important for durability, that is why it is tried to use super-absorbent polymers as an internal water reservoir. Self-compacting concrete (SCC) receives attention in two papers one of which deals with the rheology of fresh SCC and another tackles the problem of thermal curing. A case-study on a prestressed slab with unbonded tendon shows the possibilities of partly destructive measurements. Three more papers deal with timber which is an old construction material with many innovative aspects. One paper deals with statistical evaluation of the shear strength of laminated wood, another one gives a comparison of various methods for the design of lateral joints. There is a contribution by co-operative partners on the seismic behaviour of wall panels.

The MPA has many specialists on metal. One paper gives a survey on stainless steel as reinforcement in concrete structures which are exposed to seawater or deicing salts. Another one presents an innovative technology for joining aluminium components by friction stir welding.

Environmental issues get more and more attention. One speciality of the MPA is the determination of organic pollutants in indoor air by sophisticated measuring techniques. Another aspect of concern is the fire resistance of construction products. New testing methods have been developed in Europe and, therefore, Germany takes account of them in new building regulations.

There are numerous organizations which have supported the research projects. A few will be mentioned: the Federal Ministry of Culture, Research and Technology (BMBF), the Federal Ministry of Transportation, Construction and Housing (BMVBM), the State’s Ministries of Science and Research
(MWK), Social affairs (SM) and Environment (UM) of Baden-Württemberg, the German Science Community (DFG), the German Institute for Building Technology (DIBt), the German Association of Structural Concrete (DAfStb), the German Association for Timber Research (DGfH), the German Society of Concrete and Construction Technology (DBV), the Cooperative Industrial Research Community (AiF) and the Gips-Schule Foundation. The support and cooperation by these organizations and those companies and organizations not mentioned are gratefully acknowledged.

More details of the organisation of the MPA, over the departments and working fields can be found in the internet:

http://www.mpa.uni-stuttgart.de/

Also, the list of contents of previous issues and full papers of the years 1998-2004 are available under the same internet address.
CONCRETE WITH ENHANCED DURABILITY

BETON MIT ERHÖHTER DAUERHAFTIGKEIT

PLUS DURABLE BÉTON

Hans W. Reinhardt

SUMMARY

Durability is mainly a question of transport properties of concrete. Bad curing leads to larger porosity and hence larger diffusivity. By the use of hybrid aggregate concrete bad curing can be avoided even in dry climate. Cracking is the second cause of large permeability. Cracks should be minimized by fibre reinforcement. Since corrosion of steel is one main problem of durability non-ferrous materials would be a remedy.

ZUSAMMENFASSUNG


RESUME

La durabilité du béton est une question des propriétés du transport. Un traitement mal cause une grande porosité et diffusivité. Si on utilise des aggrégats hybrides on peut améliorer le béton même dans un climat sec. La fissuration est la deuxième cause d’une grande permeabilité. On doit minimiser les fissures avec fibres. Parce que la corrosion d’acier est une problème principal de la durabilité on peut utiliser des armatures non ferreuses.

KEYWORDS: Concrete, durability, cracking, curing, permeability
1. BACKGROUND

Durability of concrete is mainly a question of transport properties and chemical composition. This contribution does not deal with chemical composition and attack due to sulfates or carbonic acid, alkali-silica reaction, and other matters as such, it will not deal with frost action although frost action is also related to transport properties. Most degradation processes depend on the transport properties, one can think about corrosion of reinforcement due to carbonation and chloride ingress but also about the transport of ions which react with the matrix or with the aggregate. Transport properties are a function of porosity and cracking. If one reduces porosity and if one eliminates cracking or keeps the crack width very small durability will increase.

2. CURING

With modern concrete technology one can produce a concrete with low porosity. The means are a low water-cement ratio and use of additions and admixtures together with sound aggregates. But even if the composition is optimal and the compaction is well done something can go wrong by bad curing. Bad curing increases the porosity and hence the permeability. Fig. 1 shows the effect of the degree of hydration on the porosity. Fig. 2 shows the direct effect of curing on the gas permeability of concrete. The difference of blast furnace slag cement vs. portland cement is also visible.

![Graph showing the effect of curing on porosity](image)

*Fig. 1: Porosity of hydrated cement paste as function of water-cement ratio and degree of hydration, acc. to formula in [1]*)
Concrete with enhanced durability

Fig. 2: Effect of curing on gas permeability [2]. Curing: D = 28 d sealed, C = 3 d sealed, B = 1 d sealed, then in air 65% RH, 20°C, age about 50 d, PZ = Portland cement, HOZ = slag cement

3. HYBRID AGGREGATE CONCRETE

Hybrid aggregate concrete (also known as modified density concrete) is a concrete with normal weight aggregates and lightweight aggregates. Lightweight aggregates are either dry or presaturated. They contain water which is used for an internal curing. Experiments have been carried out with various compositions of concrete shown in Tables 1 and 2. Table 1 shows the composition of the matrix. Table 2 contains the various concrete mixes in terms of aggregates used. HB0 is the designation of a normal weight concrete whereas designations HB15 to HB30 belong to the hybrid concrete where the fraction 4/8 mm has been replaced by lightweight aggregate.

Table 1: Paste composition of 1 m$^3$ fresh concrete [3]

<table>
<thead>
<tr>
<th>Component</th>
<th>Amount</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland cement CEM I 42.5 R</td>
<td>450</td>
<td>kg</td>
</tr>
<tr>
<td>Dry mass of silica fume</td>
<td>45</td>
<td>kg</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>13.6</td>
<td>liter</td>
</tr>
<tr>
<td>Retarder</td>
<td>1.75</td>
<td>liter</td>
</tr>
<tr>
<td>Total water</td>
<td>148.5</td>
<td>liter</td>
</tr>
<tr>
<td>Water-binder ratio</td>
<td>0.30</td>
<td></td>
</tr>
</tbody>
</table>
Table 2: Aggregates of 1 m$^3$ fresh concrete, unit kg [3]

<table>
<thead>
<tr>
<th>Aggregates</th>
<th>Type of concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>HB 0</td>
</tr>
<tr>
<td>Fraction 0/2 mm</td>
<td>534</td>
</tr>
<tr>
<td>Fraction 2/4 mm</td>
<td>281</td>
</tr>
<tr>
<td>Fraction 4/8 mm</td>
<td>375</td>
</tr>
<tr>
<td>Fraction 8/16 mm</td>
<td>558</td>
</tr>
</tbody>
</table>

1) 118 kg lightweight, 262 kg normal weight aggregate (in dry state)
2) only lightweight aggregate

This type of concrete is mainly interesting for high-performance concrete, i.e. one with a low water-cement ratio. In this case water cannot be transported from outside to the interior of the concrete because of the low permeability of the concrete. Tests have been carried out at very bad curing. As can be seen from Table 3 there has been good curing, normal curing, and very bad curing.

Table 3: Curing regimes after demoulding

<table>
<thead>
<tr>
<th>Code</th>
<th>Curing regime</th>
<th>Curing efficiency</th>
</tr>
</thead>
<tbody>
<tr>
<td>FK</td>
<td>6 days in fogroom, 20°C, then in air 20°C, 65% RH</td>
<td>good</td>
</tr>
<tr>
<td>KK</td>
<td>in air, 15°C &lt; T &lt; 25°C, 40% &lt; RH &lt; 45%</td>
<td>very poor</td>
</tr>
<tr>
<td>KL</td>
<td>sealed in aluminium and polyamid foil</td>
<td>good</td>
</tr>
<tr>
<td>KR</td>
<td>in air, 20°C, 65% RH</td>
<td>poor</td>
</tr>
<tr>
<td>NK</td>
<td>6 days submersed in water, then in air 20°C, 65% RH</td>
<td>standard, acc. to DIN 1048</td>
</tr>
</tbody>
</table>
The bad curing consisted of no treatment at all at a low humidity of 40%. The next figures show the weight loss distribution in a fictitious column and the strength development at very low relative humidity.

![Fig. 3: Distribution of weight loss in the 3 x 3 cubes array after 360 days, a) Mixture 1 (HB0), b) Mixture 2 (HB25) [3]](image)

![Fig. 4: Distribution of strength in the 3 x 3 cubes array after 360 days, a) Mixture 1 (HB0), b) Mixture 2 (HB25) [3]](image)

It can be seen that the hybrid concrete lost more water than the normal weight concrete. However, due to the storage of extra water in the lightweight aggregates this does not hamper the strength development. The strength of the hybrid concrete is higher than that of the normal-weight concrete.

Shrinkage is substantially influenced by the mix composition as can be seen from Fig. 5. Fig. 5 shows that with larger amount of lightweight concrete
shrinkage decreases. Although the final shrinkage has not been reached after a 140 days the shrinkage of concrete with lightweight aggregates has the advantage that the tensile strength develops at the same time and that shrinkage cracks are not likely to occur.

![Graph of shrinkage of drying specimens](image)

*Fig. 5: Shrinkage of drying specimens [4]*

The positive influence of lightweight aggregates has also been established by the use of the hydration model of Chaube and Maekawa [5] as is shown by Fig. 6.
The more lightweight aggregate the higher is the degree of hydration at 14 days. The water-cement ratio was 0.33 for all mixes, i.e. the mixing water would not be enough for complete hydration of the cement.

The next generation of hybrid concrete is the one which uses a superabsorbent polymer for the storage of water in the interior which is used for hydration later [7, 8]. Fig. 7 shows two samples of superabsorbent polymer. The left hand picture shows the original dry powder whereas the right hand picture shows the powder mixed with water.

*Fig. 6: Degree of hydration at the age of 14 days [6]*

*Fig. 7: Superabsorbent polymer, left: dry, right: mixed with water [9]*
Fig. 8 gives an impression about water absorption by superabsorbent polymers.

![Graph of water absorption by superabsorbent polymers](image)

**Fig. 8: Water absorption by superabsorbent polymers [7]**

It can be seen that various types of superabsorbent polymers exist which are very different in their absorbing capacity. Type B absorbs water very quick within about 2 min. and has an absorption degree of more than 35 g water with respect to 1 g of dry gel. Type A absorbs water less and slower than type B. The maximum water absorption after 30 min. is only 17 g fluid/g dry gel. It should also be mentioned that many superabsorbent polymers cannot be used in concrete because they collapse in an ionic environment.

These were the means how to produce a robust concrete which does not need curing.

### 4. CRACKS

Another inherent feature of concrete is cracking. Cracking may be due to loading or may be strain induced such as by shrinkage or temperature. The permeability is a function of crack width. Theoretically, the permeability of a crack depends on the third power of the width. Fig. 9 gives an example for water permeability. Another investigation with organic chemicals has shown that a crack with about 0.04 mm width behaves as uncracked concrete. Fig. 10 shows a result.
Concrete with enhanced durability

Fig. 9: Crack permeability as function of crack width and temperature [10]

Fig. 10: Penetration of acetone in a crack as function of time [11]

From these observations it follows that a robust, i.e. curing insensitive concrete and a concrete with tiny cracks leads to enhanced durability.
5. ENGINEERED CEMENTITIOUS COMPOSITE (ECC)

In order to make cracks very fine ECC have been developed. ECC consists of a cementitious matrix with a rather high percentage of fibers. About 2% polymer fibers are added which make the matrix rather ductile. The maximum aggregate size of this concrete is 1 to 2 mm. This means that this concrete cannot be used for large structural applications but it can be used for overlays in bridges or for layers in repair work. An example of a composition is given in Table 4.

<table>
<thead>
<tr>
<th>Cement</th>
<th>Water</th>
<th>Sand</th>
<th>Fly ash</th>
<th>SP</th>
<th>V_f (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>0.53</td>
<td>0.8</td>
<td>1.2</td>
<td>0.03</td>
<td>2.0</td>
</tr>
</tbody>
</table>

The ductility is very large as can be seen in Fig. 11. The material exhibits a strain-hardening behaviour. The tensile strength is about 3.5 MPa but the ultimate strain is more than 5% [12].

![Figure 11: Tensile stress-strain and crack width curves of ECC [12]](image)

The picture on the right shows that the crack width is in the range of 40 µm.

6. STANDARDIZATION

The European standardization organisation (CEN) has published a new standard for concrete. In this standard, the various types of attacks are listed in a very systematic way. There are two main types of attack, the one is corrosion of
reinforcement and the other is degradation of concrete. The list contains XC for carbonation attack, XS for seawater attack and XD for chloride attack by other sources than seawater. The second category contains XF for frost and frost and deicing salts and XA for chemical attack. In Germany, there is an extra category for mechanical abrasion. All these categories have examples of exposure. In an extra table the composition of a concrete is given which fulfills, at a minimum level, the requirement for such an attack.

7. NON-FERROUS REINFORCEMENT

Due to the immense consumption of the Chinese economy the steel prices went up. Therefore non-ferrous reinforcement is now attractive. Furthermore, it has the advantage that it does not corrode. So, the cover to reinforcement can be reduced and the elements get lighter.

8. CONCLUSION

Durability of concrete is mainly a question of transport properties in the bulk material and through-cracks. It has been shown that fluid transport in cracks is a function of the third power of crack width. Therefore concrete structures should be designed and built in such a way that only tiny cracks can occur. A remedy would be the use of engineered cementitious composites which are however only applicable for thin-walled products. To design a robust concrete which does not need curing is hybrid aggregate concrete with incorporation of lightweight aggregates. This concrete shows a smaller shrinkage and thus it is less likely to cracking. A standardisation with clear distinction of exposure classes as developed by the European standardisation organisation can also enhance durability. Finally, the use of non-ferrous reinforcement prevents corrosion and may be advantageous in some applications, mainly with salt exposure.

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SHEAR FRACTURE ON THE BASIS OF FRACTURE MECHANICS

SCHUBVERSAGEN AUF DER BASIS DER BRUCHMECHANIK

LA RESISTANCE AU CISAILLEMENT A LA BASE DE LA MECANIQUE DE RUPTURE

Shilang Xu, Hans W. Reinhardt

SUMMARY

The shear resistance of reinforced concrete beams is explained on the base of fracture mechanics. If one uses the stress intensive factor $K_{II}$ one can predict the shear resistance of concrete. The formulas which are necessary to calculate the ultimate loads are given.

ZUSAMMENFASSUNG

Der Schubwiderstand von Trägern aus Stahlbeton wird auf der Basis der Bruchmechanik erklärt. Wenn man den Spannungsintensitätsfaktor $K_{II}$ verwendet, kann man das Schubversagen vorhersagen. Die notwendigen Formeln zur Berechnung der Schubtragfähigkeit werden angegeben.

RESUME

La résistance au cissaillement des poutres en béton armé est expliqué à la base de la mécanique de rupture. Si on utilise le facteur $K_{II}$ de la mécanique de rupture on peut prédire le resistance au cissaillement. Les formules pour calculer les forces ultimes sont données.

KEYWORDS: Shear, fracture mechanics, concrete, $K_{II}$, cracking

1. INTRODUCTION

In practical engineering, the shear failure is a fundamental problem. In a reinforced concrete beam, the shear force acted at a section is commonly taken by the concrete compress zone, the stirrups, the dowel action of the longitudinal reinforcement and the aggregate interlock in the crack. All sections of a beam are checked and reinforced according to the shear action. Nowadays, various empirical formulae to evaluate the shear loading capacity are given in different
design codes respectively that were developed using a regression analysis of experimental data. Even for the flexural-shear failure problem in the slender beams without stirrups, the empirical regression formulae are used in the design codes, no satisfactory physical model exists yet. In fact, the diagonal failure occurred in slender beams without stirrups is a typical brittle fracture, of which, the experimental results observably size effect. Therefore, several researchers attempted to apply fracture mechanics to the shear failure for gaining a satisfactory physical model of the shear failure. But, it is not clear that the shear stress component plays somehow a role in the shear failure due to the complication of the stresses distribution near the tip of the diagonal crack. In order that the shear crack propagation and shear fracture properties can be well understood, mode II testing needs to be performed for measuring mode II fracture toughness $K_{IIc}$ and mode II fracture energy $G_{IIc}$ of concrete materials. In practical engineering structures, there are some cases, for instances, the joints between dissimilar media under shear forces and normal forces parallel to the existing crack and door-case in buildings under shear forces where mode II prevails right from the crack propagation initiation. In such cases, the mode II fracture parameters can be directly applied to analysis.

In order to understand mode II fracture properties of concrete, it is necessary to perform mode II fracture tests on suitable specimen geometry. Many researchers have paid their attention on seeking mode II fracture tests without mode I component supplement. Some testing methods have been proposed and applied to various materials. Figure 1 shows eight specimen geometry and loading configurations which have been also applied to concrete. Figure 1a indicates the situation of pure shear stresses along a crack which is envisaged by testing but cannot be realized.

Figure 1b goes back to Iosipescu [1] who proposed this geometry for testing metals and welded joints. It looks very attractive and has been used by several researchers on concrete either with a single notch specimen or a double notch specimen [2-7]. The results and interpretations were rather controversial. After Barr and Derradj [8], Schlangen [9] came to the conclusion that mode I is the governing mode of this test. The push-off specimen of Figure 1c was proposed by Mattock and Hawkins [10] to investigate interfaces in reinforced concrete. Finite element analyses have shown that a tensile stress exists at the crack tip, which is of the same magnitude as the shear stress, that is, a mixed state of stress exists. A variation of the same idea has been realized by Nooru-Mohamed
The axisymmetric punch-through specimen (Figure 1d) has been analysed by Tada [12]. It has been used on mortar and concrete [13] due to its easy handling. However, numerical studies have shown that large tensile stresses occur at the crack tip. A recent study has shown that the tensile stresses can be reduced considerably by choosing four notches and by varying the depth of the notches [14].

Figure 1. Different mode II testing configurations: (a) Schematic state of shear stresses along a crack; (b) Iosipescu specimen; (c) Push-off specimen; (d) Punch-through specimen; (e) Four-notch cylinder; (f) Mixed-mode device according to Richard; (g) Mixed-mode device according to Arcan; (h) Mixed-mode disk loading; and (i) Off-centre notched beam.
For rock core testing, the cylindrical specimen with four notches (Figure 1e) has been proposed by Luong [15] which yields mixed mode results. Figures 1f and 1g show elaborate testing devices which allow various mixed combinations by rotating the holder of the specimen. Both devices (Figure 1f, according to Richard [16], Figure 1g, according to Arcan [17]) are used in photo-elastic studies but not on concrete. Izumi et al. [18] have converted the device for compressive loading and applied to concrete in mixed mode loading. Due to compressive loading there is a negative $K_I$ at the crack tip. Figure 1h has been proposed by Irobe and Pen [19] and also used by Jia et al. [20]. Finally, the off-centre notched beam specimen by Jenq and Shah [21] has been applied in a study on mixed mode fracture (Figure 1i). More investigations on mode II fracture testing methods had been made [22-31].

Although there are several methods proposed for mode II testing, none of them produces a pure mode II situation. Either by eccentric loading or by deformation during testing, a mode I contribution cannot be avoided which make these testing arrangements mixed mode devices. It was the reason to carry out this research project to look for an improved testing method to study the pure shear fracture of concrete.

Recent years, many researchers have carried out mode II fracture tests on new specimen geometry, called double-edge notched specimen. The new specimen geometry and loading arrangement has firstly been applied to wood. For these highly orthotropic materials, the double-edge notched specimen has been subjected to tensile loading which enables to measure $K_{IIc}$ and $G_{IF}$ in the direction normal to the grain (Xu, Reinhardt and Gappoev, 1996) [32].

Reinhardt, Ozbolt, Xu and Abebe (1997) [33] did not only measure $K_{IIc}$ of high strength concrete using the same approach as the above mentioned, but they also carried out numerical studies on the double-edge notched specimen using the MASA Finite Element Program based on the microplane model. In 1998, the numerical studies on the mode II geometry were continuously enhanced (Ozbolt, Reinhardt and Xu)[34].

Cedolin, Bisi and Nardallo (1997) [35] performed experiments and a numerical study on the double-edge notched specimens too. Their numerical study confirmed that the formulae (1) and (2) apply to different half width of the tested specimen. They observed the crack propagation at the notch tip through moiré interferometry. The fact was confirmed that the initiation of crack propagation at
the notch tip corresponds with the discontinuity point of the load vs. displacement recorded. This discontinuity point is the critical mode II fracture point. According to the critical mode II fracture load they determined the mode II fracture toughness $K_{IIc}$. Then using the relation of $G = K^2/E$, they calculated the fracture energy $G_{IIF}$. In fact, their experimental arrangement could have been improved. The positions of two extensometers are too far from the notches because fracture is very localized. This could easily be affected by random noise (Cedolin et al. 1998)[36]. As the result, to determine the critical mode II fracture load on the recorded plots of load vs. displacement could become difficult. Later, Prisco and Ferrara (1998) [37] numerically studied the double-edge notched mode II specimen made of high strength concrete. They aimed to evaluate fracture energy. Their numerical results are in agreement with those achieved by Ozbolt et al. (1998)[34]. When Prisco and Ferrara [37] compared their numerical results with Cedolin et al.’s experimental ones, differences of both the concrete strength and the geometry sizes used in their numerical study and the experimental observation of Cedolin et al. (1997) [35] were ignored. This could be a reason that they questioned the experimental results of Cedolin et al. (1997) [35] in their conclusions. However, Prisco and Ferrara[37] provided numerical results in detail for this new mode II geometry in their work. Then, mode II fracture toughness $K_{IIc}$ of normal strength concrete was realistically measured using tests on the double edge notched specimens by Reinhardt and Xu in 1998 [38].

Later, Reinhardt and Xu [39] used the double edge notched specimens and unnotched specimens to carry out mode II fracture tests and proposed a practical approach to determine mode II fracture energy $G_{IIF}$ from the experiments for concrete materials.

In this chapter, we mainly introduce the mode II fracture testing on the double-edge notched specimens, the measurements of mode II fracture toughness $K_{IIc}$ and mode II fracture energy $G_{IIF}$ of concrete materials, mode II fracture properties and mode II crack propagation observed in experiments and in numerical simulation. In the final section, the fracture mechanics approach to predict the shear capacity of slender beams without stirrups proposed by other researchers is introduced too.
2. THE ANALYTICAL THEORIES FOR THE MODE II TESTING METHOD

2.1 The Double-Edge Notched Infinite Plate

The theoretical analysis was carried out to propose an improved testing method. The double-edge notched plate seems appropriate. Figure 2 shows the geometry of a double-edge notched infinite plate under in-plane tensile loading. For such a case in Figure 2 a plate with unit thickness is infinite both in x-direction and in y-direction. The length of ligament, 2a, in the plate is finite, but, the lengths of double-edge notches are infinite. Under the loading condition shown in Figure 2, Tada [12] solved this problem and gave the formulae of stress intensity factors as follows:

\[ K_I = 0 \]
\[ K_{II} = \frac{\sigma}{4} \sqrt{\pi a} \]

(1)

Figure 2. Double-edge notched infinite plate.

It is important to notice that \( K_I \) vanishes and \( K_{II} \) remains as the only stress component. This theoretical solution is valid for an infinite plate and the question is whether it is also applicable to a specimen of finite size.

2.2 The Double-Edge Notched Plate of Finite Size

If the width of the plate in y-direction shown in Figure 2 is finite and is denoted with w, then, the problem becomes a double-edge notched infinite strip (see Figure 3). The stress intensity factor for such case shown in Figure 3 can be solved by methods of \( J \)-integral proposed by Rice [40].
If we take out a half of the strip, the stress distribution along the symmetry axis of the strip could be assumed to be linear distributions (see Figure 4). The $J$-integral had been stated by Radaj and Zhang [41] as in the following form:

$$J = \int_I \left( \tau_{xy} du + \sigma_y dv - W dy \right)$$  \hspace{1cm} (2)

where

$$W = \frac{1}{2E} \left[ \sigma_x^2 + \sigma_y^2 - 2ny\sigma_x\sigma_y + 2(1+ny)\tau_{xy}^2 \right]$$

According to the relation between of $J$-integral and stress intensity factor we have

$$K_{II}^2 = JE \text{ (for this case } K_I = 0) \hspace{1cm} (3)$$

---

**Figure 3. Double-edge notched infinite strip.**

**Figure 4. Integration path.**
The $J$-integral is path independent. Hence, we can choose such an integral path:

$$
J = \int_{AB} + \int_{BC} + \int_{CD} + \int_{DE} + \int_{EF} + \int_{FG} + \int_{GA} \ldots
$$

along the contour lines $GA, AB, EF, CD$:

$$
\int_{GA} \ldots = \int_{AB} \ldots = \int_{EF} \ldots = \int_{CD} \ldots = o 
(\sigma_y = o, \ \tau_{xy} = o, \ dy = o)
$$

along the contour line $FG$:

$$
\int_{FG} \ldots = o \ (\tau_{xy} = o, \ \sigma_x = o, \ \sigma_y = o)
$$

Therefore

$$
J = -(\int_{bc} W \, dy + \int_{de} W \, dy)
\quad W = \frac{\sigma_x^2}{2E} \ (\sigma_y = \tau_{xy} = o)
$$

(4)

where along the $BC$: $\sigma_x = -\sigma$, and along the $DE$:

$$
\sigma_x = \frac{\sigma}{2} - \frac{3\sigma}{4w} y
$$

By submitting these relations to (4):

$$
J = -[\int_{w} \frac{\sigma^2}{2E} \, dy + \int_{w} \frac{\sigma^2}{2E} \, dy] = \frac{\sigma w}{16E}
$$

(5)

Note with (3) we can get:

$$
K_{II} = \frac{\sigma}{4} \sqrt{w}
$$

(6)

The same numerical result has been achieved by various authors [42-45] who used other mathematical tools when they analyzed the contact problem of two perfectly bonded infinite strips of dissimilar materials. The special case of
the two equal materials and same width leads to (6) and $K_I = 0$. From [43] it is concluded that a finite length of the strip $h = 2a$ can be assumed to be infinite. Using this knowledge, new specimen geometry for pure shear testing can be designed. The length of the specimen $h$ should be $= 2a$ in order to apply equation (6). Furthermore, for $h = 2a$ and $w = \pi a$ equation (1) applies.

### 2.3 Predicted Size Effect

Equations (1) and (6) are suited to predict a size effect. According to eq. (1), $K_{II}$ increases with $a^{1/2}$, that is, if the ligament $(2a)$ of one structure is twice the one of another structure, the stress needed to reach $K_{IIc}$ is only $(1/2)^{1/2}$ as large. Figure 5 shows the relation between ligament and critical stress. This relation applies to an infinite plate.

On the other hand, if the infinite plate is reduced to an infinite strip with $2w$, eq. (6) does not show a dependence of the critical stress on the ligament length but only on the strip width. This means that the critical stress is constant for a constant $w$ and varying ligament length. This is true until $w = \pi a$. Figure 6 shows the critical stress versus the ligament length for different strip widths.

The transition point shifts to the left with smaller width of the strip. Figure 6 shows that a size effect of the ligament length does only exist if $a = w/\pi$ (stable crack growth). The same have been shown by Ozbolt [46] for mode-I and mixed failure modes. Therefore, when judging a certain structure, the starting point in Figure 6 can lie on the curves line or on the horizontal line depending upon the combination of $a$ and $w$.

There is a second feature of the double-edge notched plate which should receive attention. Equation (1) and (6) predict a critical stress for a certain $K_{IIc}$. If $K_{II} = K_{IIc}$ the crack (notch) will extend and, thus, the ligament length will decrease. As a consequence, the critical stress increases what means that $\sigma < \sigma_c$ and the situation is stable again. Figure 6 and eq. (6) predict that a crack will propagate until $a = w/\pi$ and that the crack arrests if the remote stress is kept constant. The crack propagates again only if the stress increases. This may lead to the situation that final shear failure will not occur but that other mechanisms govern the collapse for instance, compressive failure.
3. NUMERICAL STUDY ON THE DOUBLE-EDGE NOTCHED SPECIMEN OF NORMAL STRENGTH CONCRETE FOR THE MODE II TESTING

The results gained using MASA FE-Program based on the microplane model proposed by Ozbolt et al. [47] in the numerical study are shown in this section for having a good understanding of the stress field distribution, strain field distribution as well as the failure pattern during the loading process. The loading arrangement and the specimen geometry used in the numerical study are shown in Figure 7. Herein, the specimen is 200 mm high, 200 mm wide and 50 mm thick and the notch is 50 mm long and 2 mm wide. The half width to subject compressive load is 99 mm. The compressive strength of concrete is 40 MPa, the tensile strength is 2.8 MPa, the modulus of elasticity is 32 GPa and the fracture energy in mode I is assumed 80 N/m. The relation of stress - strain in uniaxial tension is given in Figure 8 (a) and the one in uniaxial compression in Figure 8 (b).

In the calculation, the gained plot of load versus displacement for the specimen is shown in Figure 9. Through investigation of the stress field distribution, strain field distribution as well as the failure pattern during the loading process, we can observe whether shear fracture occurs along the ligament prior to the compressive failure in the loaded part, or opposite. Four typical loading stages are chosen and marked on Figure 9. The corresponding four typical loading stages are 83% of $P_{\text{max}}$, 92% of $P_{\text{max}}$, $P_{\text{max}}$ and 92% of $P_{\text{max}}$ post peak load. The lateral stresses distributions for the four loading stages are shown in Figure 10. The shear stresses distributions are given in Figure 11.
Then, the shear strain distributions and the principal strain distributions are presented in Figure 12 and 13 respectively. From Figure 12, it can be seen that high shear strains occur along the ligament. Prior to exceeding the peak load, the principal strains show the same distribution as the shear strains. It implied that the shear strains are mainly dominant in the principal strain distributions.

Importantly, the results gained in the numerical study show that the shear fracture along the ligament happened prior to the compressive failure in the loaded part. Figure 15 shows somehow softening of shear stress in the front of mode II crack. Combining Figures 10 and 11, one can find mode II crack propagation prior to exceeding the peak load. It provided a good proof for the testing determination of fracture energy $G_{II}$ in mode II that will be presented later.

![Figure 7. Loading arrangement and specimen configuration used in the numerical study.](image_url)

**Figure 8.** The stress – strain relations assumed in MASA: (a) uniaxial tension and (b) uniaxial compression.
Figure 9. The plot of load vs. displacement gained in numerical study.

Figure 10. The lateral stresses distributions of four typical loading stages: (a) 83% of $P_{\text{max}}$; (b) 92% of $P_{\text{max}}$; (c) $P_{\text{max}}$ and (d) 92% of $P_{\text{max}}$ post peak load.
Shear fracture on the basis of fracture mechanics

Figure 11. The shear stresses distributions of four typical loading stages: (a) 83% of $P_{\text{max}}$; (b) 92% of $P_{\text{max}}$; (c) $P_{\text{max}}$ and (d) 92% of $P_{\text{max}}$ post peak load.
Figure 12. The shear strain distributions of four typical loading stages: (a) 83% of $P_{\text{max}}$; (b) 92% of $P_{\text{max}}$; (c) $P_{\text{max}}$ and (d) 92% of $P_{\text{max}}$ post peak load.
Shear fracture on the basis of fracture mechanics

Figure 13. The principle strain distributions of four typical loading stages: (a) 83% of $P_{\text{max}}$; (b) 92% of $P_{\text{max}}$; (c) $P_{\text{max}}$ and (d) 92% of $P_{\text{max}}$ post peak load.

Figure 14. The lateral stress distributions along the ligament at the different loading stages.
Especially, the stress distributions along the ligament were studied in detail. The lateral stresses along the ligament for several loading stages are illustrated in Figure 14 and the shear stresses in Figure 15. It can be seen that the tensile or compressive stresses distributed along the ligament that are far lower than the corresponding strength values. It shows that tensile failure cannot happen along the ligament. The shear stress distributions indicate that even though the loading level is lower, the shear stress concentration has already appeared in the front of ligament. With increase of loads, the high shear stresses go forward to centre of the ligament. The highest shear stress level appeared along the ligament when load arrives at 83% of \( P_{\text{max}} \), which corresponds about to the critical...
shear fracture. It is confirmed once more that the shear fracture along the ligament occurred prior to the compressive failure in the loaded part.

Figure 16 presents the crack slide displacement (CSD) distributed on the ligament. At the tip of the preformed crack, the mode II CSD$_c$ is 0.05 mm corresponding to the critical shear fracture.

4. MODE II TESTS FOR DETERMINING MODE II FRACTURE TOUGHNESS $K_{IIc}$

In this section, we will introduce the testing method to perform mode II fracture tests for measuring mode II fracture toughness of normal strength concrete (see [38] in details). It includes the specimen preparation, specimen sizes, loading arrangement, testing procedure, data recording, distinguishing of the critical shear fracture state and the calculation of mode II fracture toughness $K_{IIc}$.

4.1 Specimen preparation

According to the requirements shown in equations (1) and (6), the height of the specimen should meet condition of $h \geq 2a$ for a uniform stress distribution on the two end of the loaded specimen. The width of specimen mainly depends on the strength of the materials tested for having a satisfied ratio of $\sigma_c/\sigma_{max}$. The lower the value the better because it means that the shear crack develops far before the loaded part of the specimen fails under compression. To reach this, the initial notch should not be too shallow. From the experience of this investigation the notch depth should be 20 to 50 mm if the depth of the specimen is 200 mm. It turned out that the width of the specimen is important and it should not be too small. To secure good handling, the thickness of the specimen should be 50 mm to 100 mm. Table 1 contains some proposed and expected values for different strength classes of concrete. For convenience to prepare specimens according to different strength classes of concrete, one could referee the specimen sizes shown in Table 1. The notch should be cut out or be cast with a width of 1 mm to 4 mm.
Table 1. Proposed and expected critical stress for $h = 100$ mm and $d = 100$ mm

<table>
<thead>
<tr>
<th>Cube strength, MPa</th>
<th>25</th>
<th>35</th>
<th>45</th>
<th>65</th>
<th>85</th>
<th>105</th>
</tr>
</thead>
<tbody>
<tr>
<td>Assumed $K_{IIc}$, MPa m$^{1/2}$</td>
<td>1.7</td>
<td>1.9</td>
<td>2.2</td>
<td>3.3</td>
<td>4.4</td>
<td>5.5</td>
</tr>
<tr>
<td>Proposed half width $w$, mm</td>
<td>200</td>
<td>200</td>
<td>200</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Expected critical stress, MPa</td>
<td>15</td>
<td>17</td>
<td>20</td>
<td>42</td>
<td>56</td>
<td>70</td>
</tr>
</tbody>
</table>

It follows from Table 1 that a 200 mm cube could be cut into two halves if high strength concrete is to be tested. Lower grade concrete needs larger specimens because the material is less brittle. The weight of such a specimen is the same as a 200 mm cube.

4.2 Testing procedure

The tests of the double-edge notched plate specimens (DENP) should be carried out in a compressive testing machine being of enough stiffness with closed-loop servo control. The specimen is grounded to make the two loading surfaces smooth and parallel to each other. Steel plates with smooth surface were put under and on top of one half of the specimen. For eliminating friction between steel plate and concrete, a sheet of PTFE has to be added at either side. Finally, the whole arrangement consisting of steel plates and specimen was positioned very carefully between the loading platens of the testing machine in order to avoid eccentricity. The testing arrangement configuration is illustrated in Figure 17. The load should be applied with constant cross-head displacement rate of 0.002 mm/s to 0.006 mm/s at the beginning and of 0.001 mm/s to of 0.003 mm/s when about half of the expected maximum load was reached.

![Figure 17. Testing arrangement for normal concrete](image-url)
During testing, the following displacement should be continuously recorded using LVDTs: the total displacement between the loading platens, the shortening of the specimen on both surfaces of the loaded part on a distance of the height of the specimen and the notch tip opening (here called crack tip opening displacement, $CTOD$). The measuring signals should be digitized, amplified and stored using a computer data collection system.

Eq. (1) predicts for a constant $K_{IIc}$ that the critical stress is

$$\sigma_c = \frac{4 K_{IIc}}{(\pi a)^{1/2}}$$  \hspace{1cm} (7)

This means that $\sigma_c$ is smallest at the beginning of the test and $\sigma_c$ increases when a crack is initiated at the notch tip and is propagating. Therefore, it is expected that the load vs. displacement shows a discontinuity when a crack is generated.

### 4.3 Observation on crack pattern in mode II fracture tests

The results gained in numerical study showed that using the geometry configuration and the loading arrangement the shear stress concentration occurs at the crack tip. It is an expectation that mode II crack will propagate along the ligament within a certain range in front of the notch tip. We must also note such a fact that due to the compressive loading acted on the half part of the specimen, the compressive failure (lateral tensile failure) must happened in the loaded part of the specimen after the compressive strength is exceeded. When the height of the tested specimen is too large, it could lead to a tensile failure happened in the unloaded part of the specimens. This means that after the final failure happened in the whole specimen, one can find different types of failure in different parts in the specimens. Regard to that the ligament own is a part of the outline of the loaded part, the shear crack could be perhaps misinterpreted as one among the distributed cracks at the loaded part. Therefore, it is important that one must carefully chose the correct specimen size according to the strength of the tested material and exactly perform the mode II fracture tests to ensure that the shear fracture firstly occurs prior to the compressive failure happened in the loaded part caused by the maximum load. Herewith, several typical shear crack patterns observed in mode II fracture tests are described for having basically an understanding the shear crack.
Figure 18. The shear crack initiates at the tip of bottom notch that was observed in test on specimen D30-2 (after test).

Figure 19. Crack pattern of a specimen from series A with 20 mm notch depth

Figure 20. Crack pattern of two specimens from series D with 30 and 40 mm deep notches
During tests, the surface of the specimens especially the area around the notch should be carefully observed. At a certain loading a shear crack initiated at the tip of the notch. Sometimes, the crack propagated along the ligament, sometimes these were inclined cracks in the loaded part of the specimen. There was not a great difference between the load which caused shear cracking and the one causing compression failure which will be discussed later. Figure 18 shows the shear crack initiated from the tip of bottom notch. Differently, on the upper part, the shear crack begun from the top notch tip is merged into the cracks in the loaded part. Figure 19 shows another example where the shear crack started at the notch tip and merged into distributed cracks at the loaded part. Figure 20 shows a different example where the crack started from both notch tips and propagated towards the middle of the ligament to coalesce. These last examples concern tests which were stable during crack initiation and further rapid crack propagation.

By a way, we would like to describe the observation in the test on specimen numbered D30-1. In the test on specimen D30-1, after shear fracture happened along the ligament, no cracking occurs in the loading part of the specimen. Then, the loading stops. The specimen was taken out to observe carefully whether there is some cracking in the loaded part of the specimen. The observing result shows that no any cracking is found in the loading part of the specimen. After observation, the specimen D30-1 was replaced and reloaded until
failure. Now, the cracks in the loaded part of the specimen as seen in the photograph were generated during the second reloading.

Figures 21 shows the microphotographs of shear crack taken at the top and the bottom notches of specimen D20-2. After the test, no body finds any shear crack on the surfaces of the specimens. But, during the test, the discontinuity appeared on the recorded plot of load versus displacement implied that the shear crack started already at the tip of the notches. Therefore, the microphotographs were taken and the shear crack occurred at the both notches were observed. It provides a good proof to the predication using Equation 7.

4.4 Distinction of critical shear fracture load

Now, we would like to introduce how to distinguish the critical shear fracture state using the discontinuity shown on the plots of load vs. displacement. Here, four plots of load vs. displacement are selected which are typical as examples. Figure 22 shows a plot of load vs. displacement between loading platens. Due to the strain of the PTFE layer there is a nonlinear relation mainly in the beginning.

A pronounced discontinuity appears at about 176 kN due to crack initiation. After some load release the load increases again up to the maximum load which causes final compression fracture in the loaded part. When the PTFE deformation is subtracted from total displacement the load vs. displacement relation becomes almost linear up the point of discontinuity. A similar corrected plot is given for specimen B30-1, C30-1 and D30-1 in Figures 23, 24 and 25.

All figures contain the point of discontinuity which will be used later for the computation of the critical stress and hence $K_{IIc}$. It should be mentioned that the line for D30-1 is not complete and does not show the maximum load. Because the plots for load vs. displacement measured on the surface of the specimen is almost the same as the figures above they are not shown.

The plots of load vs. displacement are typically almost linear up to the point of discontinuity that is identified as the critical point of mode II fracture. As concrete is not really a brittle material, prior to the critical point, a small region of the line is slightly nonlinear. Therefore, the two formulae which are given in the equations (1) and (6) and which are developed from linear elastic fracture mechanics can be approximately applied to the critical fracture load.
For concrete materials, the compressive loading arrangement is illustrated in Figure 17. After the PTFE deformation is subtracted from the total displacement that was measured between the loading platens, the typical plots of the load vs. corrected displacement are shown in Figure 22 to Figure 25 for four different concrete mixtures. The line for specimen D30-1 is not complete and the maximum load is not shown in Figure 25. All plots show significant discontinuity mark. The relation up the point of discontinuity is almost linear.

According to the specimen sizes tested in the experiments, the critical mode II stress intensity factors for several concrete materials were determined submitting the critical fracture stress $\sigma_c$ into formulae (1) and (6). It was shown that the ratio of the critical fracture stress $\sigma_c$ to the maximum stress $\sigma_{\text{max}}$ which corresponds with compressive failure of the loaded part of a tested specimen, depends on both the half width $w$ of the specimen and the concrete strength.

![Graphs showing load vs. displacement](image1)

*Figure 22. Load vs. displacement between loading platens of specimen A30-1; a) direct measurement; b) corrected.*

![Graphs showing load vs. corrected displacement](image2)

*Figure 23. Load vs. corrected displacement for specimen B30-1.*

*Figure 24. Load vs. corrected displacement for specimen C30-1.*
Careful observation for all measured plots was made. It had been found that for a same specimen the points of discontinuity on the plots of load vs. total displacement, load vs. deformation and load vs. CTOD almost corresponds the same load value. According to the investigation to the tests on the specimens of series D, it is known that the point of discontinuity on the measured plots is caused by critical fracture of the preformed cracks in the tested specimens.

In order to determine whether the points of discontinuity on the plots measured from other testing series A, B and C are caused by the critical cracking of the preformed cracks too, further comparison should be made. As a comparing test, a curve of load versus total displacement between two platens of the testing machine and a curve of load versus deformation were measured on a cube specimen with side length of 200 mm, without preformed notches. The test for the cube was under a same testing condition. The layer of Teflon was put too for keeping the boundary condition as the same as the shear fracture tests, reducing friction between the surfaces of steel block and concrete. The measured plots of load versus total displacement of the cube numbered DC-1 are given in the Figure 26.

It can be seen that there is no any observable discontinuity on the ascending branches of the plots of the load versus displacement measured from the test on the cube specimen with side length of 200 mm without preformed notches. It is well known that in the compressive tests for concrete when the applied load exceeds about 50% of its maximum load, the observable cracking has already appeared, then, will continually develop until final collapse. The testing result of the cube specimen without preformed notches, as evidence on another aspect, confirms that the point of discontinuity on the plots of the load vs. displacement measured from the shear fracture tests on the double-edge notched plate specimens can solely be caused by the critical shear fracture of the preformed cracks in the specimens.

According to the characters appeared in the all curves from Figure 22 to Figure 26 and the investigation to the tests on specimens of series D, it can be confirmed that the point of discontinuity on the plots of load versus displacement is a critical shear fracture point. The corresponding load must the critical shear fracture load $P_c$. Therefore, the critical stress intensity factor of mode II, $K_{IIc}$, can be determined using the critical shear fracture load $P_c$. 
4.5 Experimental determination of $K_{IIc}$ values

After the appearance of the cracks and the typical load-displacement behaviour were understood it seemed acceptable to calculate the fracture toughness $K_{IIc}$ from the stress at the discontinuity point. Table 2 summarizes the results of all specimens tested.

Using the testing data gained in our mode II fracture tests shown in Table 2, the procedure for determining mode II fracture toughness $K_{IIc}$ is introduced as follows.

From the column of dimensions, it appears that the real values differ a little from the design value due to the width of the saw blade. Series A and B contain also two specimens with grooves (indicated by k). Critical stress $\sigma_c$ and maximum stress $\sigma_{max}$ are supposed to be uniform over the cross-section. Eq. (6) is used for calculating $K_{IIc}$ although, strictly, the condition of $h \geq 2a$ is only met in four cases. However, the more important condition $w \leq \pi a$ is met in all cases.

Mean values, standard deviation (S.D.), and coefficient of variation (C.V.) is given for the four series. The rounded mean values of $K_{IIc}$ are 1.68, 1.86, 2.00 and 2.04 MPa m$^{1/2}$, the respective coefficients of variations are 7.2, 10.7, 8.8, and 6.9%. These values are not larger than usual variations of mechanical properties of concrete. The $K_{IIc}$ values are higher than about twice $K_{Ic}$ values obtained by other researchers.
S. XU, H. W. REINHARDT

Table 2. Testing results of all specimens
Specimens

Dimensions
wxhxd(dn)
(mm)

A10-1
A10-2
A20-1
A30-1
A30-2
A40-1
A40-2
A40k-1*
A40k-2*
A50-1
A50-2
Mean
S.D.
C.V.
B10-1
B10-2
B20-1
B20-2
B30-1
B30-2
B40-1
B40-2
B40k-1*
B40k-2*
B50-1
B50-2
Mean
S.D.
C.V.
C20-1
C20-2
C20-3
C20-4
C30-1
C30-2
Mean
S.D.
C.V.
D20-1
D30-1
D30-2
D40-1
Mean
S.D.
C.V.

Half
Length of
ligament
a (mm)

Pc

Pmax

(kN)

(kN)

97.8x100x 93.7
98.0x100x99.0
97.8x100x93.9
98.5x100x93.1
97.8x100x99.5
97.7x100x92.4
97.5x100x99.4
98.6x100x96.7(39.4)
96.6x100x95.7(44.1)
98.9x100x98.8
96.9x100x93.5

90
90
80
70
70
60
60
58.3
58.4
50
50

211.5
207.0
204.7
176.0
178.2
185.5
222.2
158.0
144.3
215.9
205.0

211.5
220.0
223.0
206.3
235.0
197.8
226.0
213.0
202.6
215.9
205.0

98.3x100x97.9
96.4x100x94.1
97.9x100x93.5
98.5x100x97.7
99.1x100x93.4
98.3x100x98.6
99.9x100x93.2
98.5x100x98.2
98.4x100x97.0(38.6)
98.8x100x95.7(37.1)
99.3x100x95.2
98.5x100x96.4

90
90
80
80
70
70
60
60
59.5
59.1
50
50

170.0
no
229.3
212.8
207.8
210.3
237.0
238.0
180.4
175.0
237.8
242.4

258.5
278.6
240.0
276.2
238.0
263.2
243.0
259.1
282.6
264.4
237.8
253.1

97.1x100x99.2
98.1x100x96.3
98.0x100x99.6
97.1x100x98.3
98.4x100x98.4
97.2x100x99.3

80
80
80
80
70
70

225.0
280.0
255.0
232.8
236.0
246.0

331.5
417.0
355.6
351.0
374.0
382.0

200.4x100x101.0
197.4x100x102.4
199.5x100x100.9
199.0x100x102.9

80
70
70
61

397.8
380.0
359.4
351.0

559.0
490.0
530.0
495.0

σc

(MPa)
23.08
21.34
22.29
19.21
18.31
20.56
22.93
16.59*
15.58*
22.09
22.63
21.38
1.695
0.079
17.67
no
25.05
22.11
22.45
21.7
25.45
24.6
18.90*
18.51*
25.16
25.53
23.30
2.603
0.112
23.40
29.64
26.12
24.39
24.37
25.50
25.57
2.210
0.086
19.65
18.80
17.85
17.10
18.35
1.111
0.061

σmax

KIIc

(MPa)
23.08
22.68
24.28
22.51
24.15
21.92
23.32
22.34
21.91
22.09
22.63
22.81
0.823
0.036
26.89
30.72
26.22
28.71
25.72
27.16
26.10
26.78
29.60
27.97
25.16
26.66
27.31
1.651
0.060
34.4
44.14
36.43
36.78
38.63
39.60
38.33
3.374
0.088
27.62
24.24
26.33
24.17
25.59
1.684
0.066

(MPa m1/2)

* The values are not included in the calculation for the values of mean, S.D. and CV.
46

1.804
1.670
1.743
1.507
1.432
1.606
1.790
1.757
1.670
1.737
1.767
1.682
0.122
0.072
1.385
no
1.959
1.735
1.767
1.701
2.010
1.931
2.042
1.994
1.982
2.003
1.864
0.200
0.107
1.821
2.321
2.045
1.900
1.911
1.987
1.998
0.176
0.088
2.199
2.090
1.993
1.871
2.038
0.140
0.069

σc/σmax

1.000
0.941
0.918
0.853
0.758
0.938
0.983
0.743*
0.711*
1.000
1.000
0.933
0.082
0.087
0.657
no
0.955
0.770
0.873
0.799
0.975
0.919
0.639*
0.662*
1.000
0.958
0.878
0.115
0.130
0.680
0.671
0.716
0.663
0.631
0.644
0.668
0.030
0.045
0.711
0.776
0.678
0.707
0.718
0.041
0.058


For the four specimens numbered A40k-1, A40k-2, B40k-1 and B40k-2 in the Table 2, two kerfs with the width of 4 mm were sawn on the both lateral sides along the ligament for each one. In the calculation for $K_{IIc}$, the conventional assumption of section equivalence was used as the same as in [32]. The thin thickness $b_n$ of the ligament are given in the corresponding parentheses in the Table 2. However, due to inhomogeneity of concrete, complete shear fracture exactly along the ligament cannot always happen. Besides the drawback that the cracking process along the ligament cannot be directly observed, the critical shear fracture point on the recorded plots of load versus displacement is less observable than in the tests on wood and its distinction is not so easy. A great care must be taken for distinguishing it. So, it is unsuitable for concrete mode II tests to make two kerfs on both lateral sides along the ligament.

In the test on the specimen numbered B10-2, no any shear cracking was found. On the recorded plots of load versus displacement and other plots, no any observable discontinuity can be found too. So, the corresponding result for $K_{IIc}$ was lack in the Table 2.

Figures 22 to 25 indicate that there is some stable crack development prior to the critical load and also between critical and maximum load. This means that fracture energy is dissipated and a $G_{IIF}$ value would apply. A practical approach to determine mode II fracture energy $G_{IIF}$ will be introduced in next section.

![Graph](image)

*Figure 27. $K_{IIc}$ vs. $f_{c,cube}$*

A plot of $K_{IIc}$ vs. $f_{c,cube}$ of series A, B, D shows a relation which increases with strength (see Figure 27). Series C is omitted because the composition of the concrete is not known. Despite the little amount of data this relation is not in
contradiction to other fracture mechanics parameter dependencies on compressive strength.

5. DETERMINATION OF MODE II FRACTURE ENERGY $G_{IIF}$ OF CONCRETE USING A PRACTICAL APPROACH

As well known, in the mode I fracture tests, for example, when an extra load is applied to a three-point bending notched beam, the beam can be completely separated two parts and the extra work could be assumed to be wholly dissipated by the fracture process zone along the whole ligament. Differently from the mode I test on beam, in mode II fracture test on double-edge notched specimen, the extra work is not only dissipated by the shear crack propagation along the ligament, but also by compressive failure happened in the loaded part and tensile failure in the unloaded part. In order to evaluate the mode II fracture energy, our main attention will focus on the energy to drive the crack propagation near the notch tip. However, it is very difficult to determine directly the mode II fracture energy. Reinhardt and Xu (2000)[39] proposed a practical approach to evaluate the mode II fracture energy of concrete. In this section, we introduce this approach as follows in details.

5.1 Basic consideration and specimen preparation

The idea to measure $G_{IIF}$ is to compare the load vs. displacement plots of double-edge notched specimens with different ligament lengths. The difference between such lines is due to the extra work which is dissipated by mode II displacement. The longer ligament the larger the extra work. This extra work is considered to be mode II fracture energy when the specimen is completely separated along the ligament.

It is well known that concrete is inhomogeneous. The scatter of its mechanical properties is quite considerable. The influence of a small difference of ligament length on a change of the initial compliance of load vs. displacement may be of the same amount as caused by the inhomogeneity. In our previous experiments on the double-edge notched specimens it was found that the difference of the initial compliance of load vs. displacement is so small that it could not be detected when the difference of the ligament lengths used in the experiments was equal to or less than 10 mm. So, the difference of the ligament lengths for different double-edge notched specimens should be large enough that the influence of the inhomogeneity of concrete can be ignored.
Table 3. Specimen dimensions used in mode II fracture tests for measure fracture energy

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Width, 2w (mm)</th>
<th>Depth, 2h (mm)</th>
<th>Thickness (mm)</th>
<th>Notch depth (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DC30-1</td>
<td>200</td>
<td>200</td>
<td>95.7</td>
<td>29.5</td>
</tr>
<tr>
<td>DC30-2</td>
<td>200</td>
<td>200</td>
<td>99.8</td>
<td>31.1</td>
</tr>
<tr>
<td>DC0-1</td>
<td>200</td>
<td>200</td>
<td>95.7</td>
<td>0</td>
</tr>
<tr>
<td>DC0-2</td>
<td>200</td>
<td>200</td>
<td>96.0</td>
<td>0</td>
</tr>
</tbody>
</table>

In order to enable to perform the tests for obtain the extra work difference between the two specimens with different notch length, from cubes with the same dimensions of 200 x 200 x 200 mm³, two kinds of specimens will be prepared, of which, one kind of them has a constant notch length being 30 mm sawn with 4 mm wide, and the other one has no notch. The dimensions of the specimens used in the test are given in Table 3. The cube compressive strength of 28 days is 39.5 MPa.

5.2 Loading arrangement and testing procedure

The loading arrangement of the mode II tests for evaluating mode II fracture energy \( G_{II} \) is the same as that used for determining mode II fracture toughness \( K_{IIc} \) mentioned above. According to our experience, it is not so easy to perform such tests. This could be one of the reasons why some researchers reported that their tests on double-edge notched specimens carried out were mixed mode tests, not mode II tests. In order to enable readers understanding more details of the mode II tests, herewith, we prefer to repeat some procedure for performing the mode II tests.

The loading arrangement of the tests on the double-edge notched specimens is shown in Figure 28 (a), and the one without notches in Figure 28 (b). The tests were carried out in the MFL 3 MN compressive testing machine with closed-loop servocontrol. Steel plates with smooth surfaces were put under and on top of one half of the specimen. To eliminate friction between the steel plate and concrete, a sheet of PTFE was added at either side. The whole arrangement consisting of steel plates and specimen was positioned very carefully between the loading platens of the testing machine in order to avoid eccentricity. The load was applied with a constant crosshead displacement rate of 0.003 mm/s at
the beginning and of 0.0015 mm/s when about half of the expected maximum load was reached.

The following displacements were continuously recorded by LVDTs through the Diadem computation system: the total displacement between the loading plates, the shortening of the ligament on both sides of the notch tips and on both surfaces with a distance of about 140 mm. The LVDT that is attached at the front of the specimen and at the loaded part is noted with LF, and at the unloaded part with UF (see Figure 28). On the back surface, they are denoted with LB and UB respectively. For the specimens that have no notches, shown in Figure 28 (b), the LVDTs were attached at both sides of the centre line.

![Testing machine platen](image)

**Figure 28.** The testing arrangement, (a) the notched specimen; (b) the unnotched specimen.

### 5.3 Description of observed crack propagation

During the tests, the crack propagation on both kinds of specimens was carefully observed. For the notched specimen, a shear crack started at the tip of the notch when the loading reached a certain value; then, the crack propagated along the ligament; finally, the crack entered the loaded part of the specimen. In the loaded part, vertical cracks could be observed basically parallel each other after the maximum load was reached. The specimens without notches were tested in the same loading arrangement as the one for the notched specimens. Due to the shear stress concentration at the middle points on the bottom and the top of the unnotched specimen, at the edge of the loading steel block, a very shape crack was formed. The two cracks propagated from the bottom and the top
of the specimen and then developed almost exactly along the ligament, e.g., the middle line. Finally, complete fracture occurred along the ligament.

Comparing the observation for the two kinds of specimens, it could be thought that unnotched specimens under the loading arrangement shown in Figure 28 (b) could be more suitable for pure mode II fracture tests. The fact that a very sharp tip of the crack was formed by the shear stress concentration at the edges before crack propagation is very important for mode II fracture. Contrarily, the notches sawn in the notched specimens are 4 mm wide.

How to determine the length of an initial crack formed by the stress concentration needs further study. When the dimension conditions of specimens should satisfy formula (1), one needs to know it for determining $K_{IIc}$. In fact, the process of an unnotched specimen includes the formation of an initial crack due to the stress concentration and the further propagation of the crack formed.

As a limit case, the length of the initial crack could be assumed to be zero for the unnotched specimen. So the maximum length of the ligament $a$ will be equal to the half depth $h$ of the specimen. Under this condition the stress intensity factor $K_{II}$ will arrive at its maximum value at a certain loading when the formula (1) can be applied. Therefore, the unnotched specimen is most beneficial for initiating fracture.

Although this new problem without notches has to be theoretically solved yet, the new specimen type and the loading arrangement is valuable for carrying out pure mode II tests. Due to the shear stress concentration at the middle point at the bottom and the top of the unnotched specimen, the formation of an initial crack is possible. Then, the sharp tip of a crack is a suitable starter for crack propagation. Finally, the important testing phenomenon is that complete fracture happened along the ligament.

### 5.4 Distinction of critical shear fracture using the discontinuity shown on the recorded plots of load vs. displacement

The measured displacement between the loading plates, the total displacement includes the deformation of the PTFE layer. Due to the PTFE, there is a nonlinear relation mainly in the beginning (see Figure 22(a)). When the PTFE deformation is subtracted from total displacement, the load vs. corrected displacement for the four tested specimens are given in Figure 29 to Figure 30 for the notched specimens and the unnotched specimens respectively.
Some feature of the load vs. displacement curves shown in Figure 29 and 30 should be mentioned. Average loading values $P_c$ at the points of discontinuity and the average maximum load $P_{\text{max}}$ are 314 kN and 424 kN for the notched specimens and 321 kN and 421 kN for the unnotched specimens respectively. The plots of load vs. displacement are almost linear up to the points of discontinuity. It is the same observation as the description in section 4 in this Chapter. For each group of specimens, the plots of load vs. displacement including the maximum load and the loads at the points of discontinuity coincided very well.

The load vs. displacement plots measured from the two groups of specimens that have different ligament lengths are drawn in one graph as in Figure 31. A partly enlarged graph is given in Figure 32.
Shear fracture on the basis of fracture mechanics

Figure 33. Typical plots of load vs. deformation measured from the back surface and the front at the loaded part and unloaded part (specimen DC30-2).

Figure 34. Average plots of load vs. deformation for specimen DC30-1.

Figure 35. Average plots of load vs. deformation for specimen DC30-2.
Due to enough large difference of ligament length chosen in the tests, the initial compliance difference due to the ligament length are manifested in both the whole relation of load vs. displacement as Figure 31 and the locally enlarged Figure 32. It can be used for the determination of mode II fracture energy. The approach will be introduced in the next section.

In the tests, four LVDTs were used to measure the deformation of the ligament at both sides of the middle line, of which one side is at the loaded part and another one is at the unloaded part, and both the front and the back surfaces of the specimen. The distance to measure deformation is about 140 mm for all specimens. The typical plots of load vs. deformation measured from the frond and the back surfaces are shown in Figure 33. The average plots measured from the two surfaces are given in Figure 34 to Figure 37.

Points of discontinuity can be seen on Figure 34 to 37. They are in agreement with those manifested in plots of load vs. displacement shown in Figure 29 and Figure 30 very well, even though more sensitive than the discontinuous character of Figures 29 and 30. Reinhardt and Xu (1998) [38] attributed the points of discontinuity to the critical point of shear fracture. And Cedolin et al.’s observation (1997) [35] certified that through the moiré interferometry. However, to detect the critical point on the curves of load vs. deformation measured by Cedolin et al. is not easy because the measuring extensometers were positioned far from the local fracture zone [35].

The messages that a plot of load vs. deformation measured from the unloaded part provides are most important for manifesting a shear fracture proc-
ess in the tests. Figure 34 to Figure 37, besides Figure 36, show that the deforma-
tion of the unloaded part almost linearly increased until the critical load was
achieved. Between the critical load and the maximum load, the deformation
slowly decreased due to the shear crack initiation and the propagation. When the
loading arrived at the maximum load value, the deformation suddenly went
down in the opposite direction. The phenomena imply that after the maximum
load was exceeded, the unloaded part can be approximately thought to be free of
the action of the loaded part. This means that a part of the total deformation en-
ergy was dissipated by the shear fracture along the ligament during the stable
crack propagation prior to the critical load and between the critical load and the
maximum load. Therefore, it is possible to determine fracture energy in mode II
through experiments.

5.5 Determination of mode II fracture energy $G_{II}$ of concrete

When experiments are carried out on the two different specimens according
to Figure 38 (a) and (b), the initial compliance of load vs. displacement relations
was found to be different due to the difference of ligament length. The testing
results could be seen in Figure 31 and 32.

![Figure 38. The same loading arrangement on two different specimens: (a) a notched specimen configuration; and (b) an unnotched specimen.](attachment:image.png)

When the ligament lengths are different, a difference of the deformation
energy could be experimentally measured. Using the difference, the fracture en-
ergy needed by driving the crack with a unit area can be determined. Generally,
there are two ideal loading approaches to perform the tests. One is that the
boundary condition of deformation is given. The other is that the applied load to the two specimens shown is the same.

If the boundary condition of deformation has been given in the tests, two load vs. displacement curves corresponding to the two different specimen configurations are illustrated in Figure 39.

The total work of load transforms mainly into three parts, besides thermal energy dissipation that can be neglected until the maximum load is reached. The first part transforms into the compressive strain energy. The plastic deformation and the slight compressive failure in the loaded part dissipate the second part. The third part is dissipated due to shear crack propagation along the ligament. According to the load vs. deformation plots measured at the unloaded part, the energy dissipation due to shear crack propagation along the ligament proceeds until the loading arrives at its maximum. Once the maximum load is exceeded, the dissipation of the work of load is predominantly due to the compressive failure in the loaded part.

Now we discuss two cases shown in Figure 39 and Figure 40 separately. In Figure 39 the boundary condition of deformation is given. The thickness of two kinds of specimens is the same and is denoted with b. The notch depth of the notched specimen is c. The above plot of load vs. displacement measured from the unnotched specimen is marked with curve 1 and the lower one that is measured from the notched specimen is curve 2. According to the definition of fracture energy by Hillerborg (1976) [48], the mode II fracture energy \( G_{\text{IF}} \) can be evaluated as follows:

\[
G_{\text{IF}} = \frac{\Delta W}{2cb} = \frac{1}{2cb} \int_0^{\delta_c} \left[ F_1(\delta) - F_2(\delta) \right] d\delta
\]

(8)
In our tests, the specimens that have the same outline dimension, besides the ligament length, and are composed of the same concrete were tested under the same loading condition. The same loading arrangement, the same specimen dimensions and the same concrete led the approximately same maximum load. The difference of the ligament length led the different deformation. So, the measured plots of load vs. displacement presented in Figure 32 can be abstracted into the pattern shown in Figure 40.

The mode II fracture energy $G_{II}$ for the tests illustrated in Figure 40 can be calculated as follows:

$$G_{II} = \frac{\Delta W}{2cb} = \frac{1}{2cb} \int_{0}^{F_p} \left[ \delta_i(F) - \delta_c(F) \right] dF$$  \hspace{1cm} (9)$$

According to integration (9), if one knows the area between the curves 1 and 2 shown in Figure 40, the mode II fracture energy $G_{II}$ can be evaluated using the measured load vs. displacement plots. In fact, the difference $\Delta W$ between load work $W_p$ is equal to that between residual work $W^R_p$ corresponding to the peak load $F_p$. Considering such a fact that there is some deviation of practically measured load vs displacement plots from the ideal cases, a better way calculating the difference $\Delta W$ of load work between two specimens with different liga-
ment lengths is to evaluate the difference between residual work measured from them.

As shown in Figure 40, curve 1 represents the load vs. displacement plot measured from the unnotched specimen and curve 2 does the notched specimen. The residual work \( W_{RP} \) for the curves 1 and 2 can be evaluated as follows respectively:

\[
W_{RP} = F_{p1} \delta_{p1} - W_{p1} = F_{p1} \delta_{p1} - \int_{0}^{\delta_{p1}} F_{1}(\delta) \, d\delta \quad \text{(for unnotched specimen)} \tag{10}
\]

It is known that for load work the relation \( W_{p1} > W_{p2} \) generally satisfies. Therefore, there is a relation of \( W_{RP}^{p2} > W_{RP}^{p1} \) too. So, integration (8) can be simply expressed as follows:

\[
W_{RP} = F_{p2} \delta_{p2} - W_{p2} = F_{p2} \delta_{p2} - \int_{0}^{\delta_{p2}} F_{2}(\delta) \, d\delta \quad \text{(for notched specimen)} \tag{11}
\]

\[
G_{II} = \frac{\Delta W}{2cb} = W_{RP}^{p2} - W_{RP}^{p1}
\]

The mode II fracture energy \( G_{II} \) gained according this approach in the experiments is presented in Table 4. For normal strength concrete, mode I fracture energy is about 80 N/m to 100 N/m. From the testing results shown in Table 4, mode II fracture energy is about 20 times to 25 times mode I fracture energy.

An average value of \( K_{IIc} \) from the two specimens DC30-1 and DC30-2 is 2.56 MPa m\(^{1/2}\) that was calculated using formula (1). According to the relation of \( G = K^2/E \), the critical energy release rate \( G_{IIc} \) in mode II is 240 N/m where the modulus of elasticity of the concrete was evaluated according to the compressive strength. To compare \( G_{IIc} \) with \( G_{II} \) in Table 4, the mode II fracture energy \( G_{II} \) is 8.6 times mode II critical energy release rate \( G_{IIc} \). The ratio is the same amount as that in mode I.

Figure 29 and 30 and Table 4 show that the testing results gained in the experiments almost have no disparity due to good controlling of concrete quality and carefulness during testing procedure. However, more specimens need to be studied in the future.
Table 4. Fracture energy $G_{IIc}$ and critical energy release rate $G_{IIc}$ from mode II concrete fracture tests.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$F_c$ (kN)</th>
<th>$\delta_c$ (mm)</th>
<th>$F_p$ (kN)</th>
<th>$\delta_p$ (mm)</th>
<th>Residual work $W^R_p$ (N.m)</th>
<th>$\Delta W$ (N.m)</th>
<th>$G_{IIc}$ (N/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DC30-1</td>
<td>319</td>
<td>0.460</td>
<td>422</td>
<td>0.649</td>
<td>129.78</td>
<td>12.35</td>
<td>2058</td>
</tr>
<tr>
<td>DC30-2</td>
<td>309</td>
<td>0.445</td>
<td>426</td>
<td>0.652</td>
<td>132.77</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>314</td>
<td>0.453</td>
<td>424</td>
<td>0.650</td>
<td>131.27</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DC0-1</td>
<td>320</td>
<td>0.420</td>
<td>420</td>
<td>0.599</td>
<td>118.62</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DC0-2</td>
<td>322</td>
<td>0.416</td>
<td>422</td>
<td>0.683</td>
<td>119.30</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>321</td>
<td>0.418</td>
<td>421</td>
<td>0.641</td>
<td>118.92</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

6. SHEAR FRACTURE OF REINFORCED CONCRETE BEAMS WITHOUT STIRRUPS

In last two decades the experimental investigations focused on shear failure of reinforced concrete beams have been intensively performed by many researchers for capturing the failure mechanisms [49-66]. As the consequence, various shear failure models were proposed to attempt acquiring adequate formulae to predict the shear strength of the reinforced concrete beams with good accuracy for designing engineers. However, the formulae that are used in various design codes are empirically yet [67-72] inasmuch as no physically sound analytical mode that enables to yield satisfactory results for all cases of the shear failure of the reinforced concrete beams exists.

Recent years, some researchers have put their attentions on application of fracture mechanics to the shear failure of the reinforced concrete beams without stirrups for developing an analytical model with soundly physical fundament [63-66]. This is based on such a fact revealed in the intensively experimental investigations that the shear failure of a beam is triggered off by a series of fracture processes occurred in the beam. It is investigated that these fracture phenomena includes diagonal shear fracture of concrete in the web region, shear-compression fracture of concrete in the region above the tip of the critical crack, interface bond fracture between steel bar and concrete due to the shear stress concentration and splitting fracture of concrete cover. The later mentioned two fractures somehow lead to lost of the dowel action so called. In this section, we
will briefly introduce the some empirical methods used in several current designing codes; then, in details, introduce the fracture mechanics approaches to predict the shear failure of reinforced concrete beams without stirrups proposed by other researchers; finally, a physically sound analytical formula termed in mode II fracture toughness $K_{IIc}$ of concrete materials that we recently achieved using fracture mechanics approach is presented.

6.1 Empirical approaches

As the mentioned above, inasmuch as the mechanism of shear failure of reinforced concrete beams without stirrups is very complex, no analytical model was developed for the use of current design codes for practical engineers. However, intensive experiments on reinforced concrete beams without stirrups have been carried out by many researchers and a lot of experimental data has been accumulated for appearances of various empirical formulae using phenomenological study approach, dimensional analysis and regression analysis.

However, according to intensively experimental observations and the conventional opinion of view, the shear transfer capacity of the reinforced concrete beams without stirrups is regarded as to be influenced by the five facts which are shear stresses distributed on the compression zone of concrete, arching action formed by the inclined compression force in the compression zone, aggregate interlock that enables to transfer the shear stress along the diagonal shear crack, the softening cohesive stress vertically distributed across the diagonal shear crack and the dowel action due to the longitudinal reinforcement. In spite of several models with somehow physical meaning, like the modified compression field theory (MCFT)[55] and truss models with concrete ties[57], were proposed; the corresponding calculation formulae are empirically formed using regression analysis of experimental data. Due to lack of the physically analytical model, various calculation formulae that are coded in the current design codes in the world wide are empirically using regression analysis of the experimental data. Currently, ACI Subcommittee 445-F is making a new proposal [73] to present an empirical formula to predict the shear capacity of the reinforced concrete beams without stirrups using a regression analysis of the experimental data based on the Evaluation Shear Database (ESDB) developed by the subcommittee. If one sees the empirical formulae used in the design codes, it can be found that the strength of concrete where it is in term of tensile strength appeared in a form of being approximately proportional to $\sqrt{f_c}$ , the longitudinal reinforcement
ratio and the shear span-depth ratio were considered as the main influence facts on the shear capacity of the reinforced concrete beams without stirrups.

As an instance, we can see a basic form of earlier empirical formula proposed by Zsuty in 1968 [50] using the data of 86 reinforced concrete slender beams without stirrups which was constructed through a dimensional analysis and a regression analysis as follows:

\[
V_c = 2210 \left( f_c \rho \frac{d}{a_s} \right)^{1/3} bd
\]  

Where the unit of the critical shear bearing capacity \( V_c \) is kN, \( \rho = A_s / bd \) longitudinal reinforcement ratio, \( a_s / d \) is the shear span-depth ratio, \( f_c \) is compression strength of concrete in MPa, the width \( b \) and the depth of beam \( d \) are in unit of m.

Furthermore, many experimental results show that there is size effect on shear capacity of the reinforced concrete beams. Therefore, the size effect on shear capacity has been considered in the British design code [68], the CEB-FIP Model Code 1990 [69], the Canadian Standards Association Model Code (CSA Code) [70] and the concrete design code of Japan (JSCE Code) [72].

Inasmuch as the formula used in CSA code [70] is based on the Modified Compression Field Theory (MCFT)[55], in addition to the difficulty of the method in practical use, when the depth \( d \) is quite large, the formula shows an over strong asymptotic size effect \( V_c \sim d^{1} \), which is contrary to the well-known knowledge that exponent \(-1/2\) is the strongest size effect possible for the fracture mechanics size effect (Bazant and Yu 2003)[63]. The formulae that is gained using the MCFT approach are shown as follows:

\[
V_c = \frac{245}{1275 + s_x} \sqrt{f_c} bd \quad \text{(mm and MPa units)}
\]  

\[
s_x = \frac{35s_x}{(a + 16)} \quad \text{(mm unit)}
\]

Where, \( s_x \) is proportional to the depth \( d \). For example, \( s_x \) is taken as 0.9d for the beams with concentrated reinforced near the tension face. This reveals the size effect with an exponent \(-1\).
The corresponding formula used in JSCE Code [72] was proposed by Okamura and Higai (1980)[51] using the weakest link assumption according to Weibull’s statistical theory (1939) [74].

The formulae proposed by the British code [68] and the CEB-FIP Model Code 1990 [69] were developed using much more experimental data from the basic form of the empirical formula of Zsuty[50].

Differently, the British design code did not consider the shear span-depth ratio $a_s/d$ shown in following equation [68].

\[
V_c = \frac{790}{\gamma_m} \left(100\rho\right)^{1/3} \left(\frac{0.4}{d}\right)^{1/4} \left(\frac{f_c}{25}\right)^{1/3} bd
\]  

(16)

Where, $\gamma_m$ is a safety factor being 1.25 and $100 \rho < 3$, $V_c$ is kN, $f_c$ is MPa, $b$ and $d$ are in m.

As the most sophisticated formula in the current design codes in the world wide, the formula proposed by the CEB-FIP Model Code [69] is as follows:

\[
V_c = 150 \left[1 + \sqrt{\frac{0.2}{d}} \left(\frac{3d}{a_s}\right)^{1/4} (100\rho)^{1/3} f_c^{1/3} bd\right]
\]  

(17)

Where, $V_c$ is kN, $f_c$ is MPa, $b$, $d$ and $a_s$ are in m. It can be seen that the size effect is presented by an expected exponent 1/2 in formula (17).

Different from the aforementioned codes, ACI 318-89 code [67] proposed a simplified formula that the shear capacity is assumed to be entirely contributed by concrete.

\[
V_c = \frac{1}{6} \sqrt{f_c bd}
\]  

(18)

Where, the unit of $V_c$ is N, $f_c$ is MPa, $b$ and $d$ are mm. However, it was found that formula (18) cannot predict the experimental results that the shear strength of the reinforced concrete beams without stirrups decreases as the depth of the beam decreases and the longitudinal reinforcement ratio decreases. Especially, when the size of the members is larger and is lightly reinforced, formula (18) will lead to an overestimated prediction result. Therefore, the ACI sub-committee 445F [73] is working to propose a new proposal for adding new terms to attempt to reflect the influence of the size effect, the longitudinal rein-
forcement and the shear span-depth ratio on the shear capacity. The subcommittee has provided three empirical formulae for three different considered facts for choice. In the three empirical formulae, an exponent -1/3 is used for size effect that is not ideal according to fracture mechanics. An exponent -1/2 should be expected to be adopted for predicting size effect once the foregoing formulae would be improved using the concept of fracture mechanics. As the results, the corresponding coefficients used in the aforementioned formulae should be refitted according to the experimental data.

6.2 Fracture mechanics approaches

It was recognized that the shear failure of reinforced concrete beams is a very complex brittle fracture process and behaves significant size effect. Besides the various empirical formulae, the existing models with somehow physical meaning based on such intensive experiments on the shear failure of the reinforced concrete beams without stirrups carried out by many researchers only capture a little of interpretation for the shear mechanisms both in physics and mathematics. It motivated many researchers putting their attentions on application of fracture mechanics into shear failure of reinforced concrete beams without stirrups to attempt a physically sound analytical model.

Among the research works appeared in the last two decades, there mainly are two research objects that can be distinguished. One put an emphasis on predication of size effect of shear capacity of the reinforced concrete beams without stirrups. Another did on good interpretation of failure mechanisms.

Bazant developed a size effect formula from fracture mechanics for predicting the shear strength in 1984 [53]. In addition, Bazant and Kim (1984) [54] studied the size effect in shear failure of reinforced concrete beams using the size effect approach. In the work, the shear strength is assumed to be mainly due to a common contribution of the arching action and the composite beam action. Latterly, the size effect formula was extended by Bazant in 1987[56] to such a case that the cohesive stresses across the diagonal shear crack remain a limited residual value, do not decrease to zero, based on the concept of the cohesive crack model (i.e. fictitious crack model) that was proposed by Hillerborg in 1976[48].

Little later, Gustafsson and Hillerborg (1988) [59] numerically studied the diagonal shear cracking processes with different crack patterns in RC members without stirrups using the fictitious crack model, of which, the main aim is to
investigate how predict the size effect of shear strength of the members. In the study, not only the size effect, but also the longitudinal reinforcement ratio and the shear span-depth ratio were investigated in details. As the result, they adopted the size effect formula of shear strength of RC beams without stirrups in a form of $d^{-1/4}$.

In this year (2003), Bazant and Yu [63] presented a new formula to predict the size effect of shear capacity of RC members without stirrups. According to the small- and large-size second-order asymptotic properties of the cohesive crack model gained by using dimensional analysis, a general form of the size effect with the same form as the previous work of Bazant was revealed once again that is presented as follows:

$$V_c = \beta \frac{f_c}{\sqrt{1 + d/d_0}}$$

(19)

Where, $\beta$ and $d_0$ are two constants. Using a statistical regression based on the meticulously chosen experimental data with sound size effect from the database collected by ACI 445, the two constant parameters $\beta$ and $d_0$ were determined for two different cases considering the influence of shear span-depth ratio $a_s/d$ and without consideration of $a_s/d$ respectively. They are shown as follows.

When the shear span-depth ratio is not considered, the two constants are given as follows:

$$d_0 = 30930 \left( \frac{\rho}{f_c} \right)^{2/3} \quad \beta = 0.415 \text{ (mean)}, \ \beta = 0.315 \text{ (design)} \quad (20)$$

And, if the shear span-depth ratio is included, they are in formula (21).

$$d_0 = 37280 \left( \frac{\rho}{f_c} \right)^{2/3} \left( \frac{d}{d_s} \right)^{1/3} \quad \beta = 0.457 \text{ (mean)}, \ \beta = 0.35 \text{ (design)} \quad (21)$$

In the foregoing formulae, $V_c$ and $f_c$ are MPa, in SI units and $d$, $a_s$ are in mm, $\rho$ is a number, not a percentage.

In order to better understand the shear failure mechanisms of the reinforced concrete members without stirrups, Jenq and Shah (1989) [64] employed the two-parameter fracture model to analyses the diagonal shear crack. In their study, the shear capacity is assumed to be a combination contribution of the
concrete and the longitudinal reinforcement. In this way, the steel action associates with the bond stress which is assumed to be function of the embedded length. Later, Karihaloo (1993) [65] made a modification for Jenq and Shah’s model [64] by taking into account the bond-slip relationship, the dowel action and the aggregate interlock.

Recently, Gastebled and May (2001) [66] proposed an analytical model for the shear failure of the RC beams without stirrups using a fracture energy approach. As the consequence, they archived an analytical formula that agrees very well with the empirical formula of the CEB-FIP Model Code 1990. Using this formula, one can predict the shear bearing capacity of the reinforced concrete beams without stirrups by taking into account the size effect, shear span-depth ratio, the longitudinal reinforcement ratio, the elastic modulus of steel and the concrete strength. Herein, we will introduce their shear fracture model in details to show how use fracture mechanics approach to predict the shear failure of the reinforced concrete beams without stirrups.

The main principle used in Gastebled and May’s approach [66] is that the unit extra work produced by the extra moment to the unit rotation at the tip of diagonal shear crack is equal to the fracture energy that is necessary to extend the unit unbonded length of longitudinal reinforcement. In fact, the unbounded failure could be dominated by a mode II fracture caused by high concentration of the bond stress on the interface between the steel and the concrete.
The free body diagram used in the paper of Gasteble and May (2001) [66] is shown in Figure 41. The detail of the steel deformation after unbonded is illustrated in Figure 42. If the angle of the diagonal shear crack is assumed to be 45°, the horizontal component $F_s$ and the vertical $F_d$ of the internal force in the longitudinal reinforcement crossing the diagonal shear crack are assumed to be associated with the angle of rotation $\theta$ at the tip of the diagonal shear crack according to the elastic properties of the steel and the deformation coordination conditions. The corresponding equations are given as follows:

$$F_s = \frac{E_s A_s}{\eta} \Delta u_s = \frac{E_s A_s}{\eta} y \theta$$  \hspace{1cm} (22)

$$F_d = \frac{G_s B_s}{\eta} \Delta v_s = \frac{9}{26} \frac{E_s A_s}{\eta} y \theta$$  \hspace{1cm} (23)

Where, $E_s$ and $G_s$ are the elastic modulus and the shear modulus of the steel respectively and they meet a relation of $G_s = \frac{E_s}{2(1+\nu_s)} = \frac{9 E_s}{26}$ for engineering application. $A_s$ represents cross section area of the longitudinal steel bar.

Under shear condition, the calculated area of the cross section should be reduced, assuming $B_s = 0.9 A_s$. $\eta$ is the unbonded length of the steel.

According to the equilibrium conditions of the free body shown in Figure 41, one can get the following set of equations:

$$F_s = F_c$$ \hspace{1cm} (24)

$$V = V_c + V_d$$ \hspace{1cm} (25)

$$V a_c = y V_d + jd F_s$$ \hspace{1cm} (26)

If the horizontal projection length $y$ of the diagonal crack and the internal moment arm $jd$ are assumed to be proportional to the height of the beam $H$, there are $y = qH$ and $jd = rH$. Then, submitting equations (22) and (23) into equation (26), we have equation (27).

$$V a_c = q(\frac{9}{26} q + r) \frac{E_s A_s}{\eta} H^2 \theta$$ \hspace{1cm} (27)
If \( \theta \) is considered as a function of variable \( \eta \) and a differentiation for function \( \theta \) is made referring to the variable \( \eta \), one can set up a differentiation equation as follows:

\[
\delta \theta = \frac{a_s}{A_sE_s q(9q/26 + r)H^2} \delta \eta
\]  
(28)

Where, \( \delta \eta \) is the variation of the unbonded length.

According to the linear elastic fracture mechanics, the variation rate of the extra work is 2 times the energy variation at the critical fracture state, it is presented as following fundamental equation:

\[
\partial W = 2\delta G
\]  
(29)

For considering the energy equilibrium at the critical fracture state in the unbonded failure shown in Figure 41, we have the following equation.

\[
a_v \delta \theta = 2G \delta \eta
\]  
(30)

Where, \( G \) is the fracture energy necessary to drive the crack propagation of unit length which is equal right unit unbonded length of steel bar.

Formula (30) applies to the unit thickness case. If we want to know the shear bearing capacity of a beam with thickness \( b \), the thickness \( b \) must be considered in next analytical derivation. Additionally, the foregoing bond fracture is mode II fracture domination case, not mode I fracture. According to the conventional definition, the fracture energy is necessary to create a unit area of crack propagation. Therefore, now we should use the term \( G_{IIF} \) that represents mode II fracture energy to create unit area of crack, instead of the symbol \( G \) that is the fracture energy to create unit length of crack for the unit thickness case shown in formula (30). As the consequence, the corresponding expression of critical shear capacity of a reinforced concrete beam without stirrups can be obtained by submitting equation (28) into equation (30) as follows:

\[
V_c^2 = \left( \frac{qH}{a_s} \right)^2 \left( \frac{9}{13} + \frac{2r}{q} \right) A_s E_s G_{IIF} b
\]  
(31)
In the work, Gastebled and May (2001) [66] assumed \( q=0.8 \) and \( r=0.9 \) according to the shear capacity properties of reinforced concrete beams without stirrups. Then, we have following expression:

\[
V_c = 1.372 \frac{H}{a_c} \sqrt{b G_{IIr} A_s E_s} \tag{32}
\]

In order to predict the position of diagonal crack a semi-empirical formula that had been proposed by Kim and White (1991) [61] is given as follows:

\[
a_c = 3.3 a_s \left( \frac{\rho (d/a_s)^2}{1 - \sqrt{\rho}} \right)^{1/3} \tag{33}
\]

When equation (33) is submitted into equation (32) and let \( d=0.9H \), the equation for predicting the shear capacity of the reinforced concrete beams without stirrups can be expressed as follows:

\[
V_c = 0.446 \left( \frac{H}{a_s} \right)^{1/3} \rho^{1/6} (1 - \sqrt{\rho})^{2/3} \sqrt{E_s G_{IIr}} bH \tag{34}
\]

Due to lack of mode II fracture energy measured form tests, Gastebled and May [66] directly used mode I fracture energy \( G_f \) submitting into equation (34), instead of \( G_{IIr} \). According to CEB-FIP Model Code 1990 [69], the mode I fracture energy is estimated using the following equation:

\[
G_f = (0.0469 d_{agg} - 0.5 d_{agg} + 26) \left( \frac{f_c}{10} \right)^{0.7} \text{N.m/m}^2 \tag{35}
\]

Let \( d_{agg} = 0.02 \text{ m} \), the above equation can be expressed a simple form.

\[
G_f = 5.2 f_c^{0.7} \text{N.m/m}^2 \tag{36}
\]

Submitting (36) into (34), we can once again get the analytical formula to predict the shear capacity of reinforced concrete members without stirrups achieved by Gastebled and May (2001) [66]. However, the coefficient is little different.
After comparing with the empirical formula proposed by CEB-FIP Model Code 1990 [69], it was found that the formula (37) has good agreement with that of CEB-FIP Model Code 1990. It means that fracture mechanics can be employed as a useful tool to analyse the shear fracture of reinforced concrete members without stirrups.

Using a basic fundament of linear elastic fracture mechanics, there is a relationship of $K_{IIc} = (G_{III} E_c)^{1/2}$. Then, considering a relationship between the elastic modulus of the steel and the concrete, the equation (34) could be expressed as follows.

$$V_c = \frac{1.016}{\sqrt{H}} \left( \frac{H}{a_s} \right)^{1/3} \rho^{1/6} (1 - \sqrt{\rho})^{2/3} f_{c}^{0.35} \sqrt{E_s} bH \quad (N, \text{mm, MPa, GPa units}) \quad (37)$$

By the way, the equation that associates with mode II fracture toughness $K_{IIc}$ of concrete can be deduced by submitting equation (33) into equation (38) and let $d= 0.9H$. The equation for predicting the shear capacity of the reinforced concrete beams without stirrups in terms of mode II fracture toughness $K_{IIc}$ can be expressed as follows:

$$V_c = 1.372 \sqrt{\frac{E_s}{E_c}} \frac{1}{a_c} \sqrt{\rho H K_{IIc}} bH$$  \quad (38)

Theoretically specking, we would prefer to use formula (39) for predicting the shear strength of RC members without stirrups. It needs to perform intensive mode II fracture tests to know the mode II fracture toughness $K_{IIc}$ or mode II fracture energy $G_{III}$ of concrete materials with different strength scales and the design values of $K_{IIc}$ or $G_{III}$ which correspond to 5% probability cut-off.

When $E_s/E_c = 6$ is used, equation (39) can be simplified as follows. In assessment of shear capacity of RC beams without stirrups, a larger safety factor $\gamma_m = 1.35$ is assumed considering mode II fracture toughness as a new material property.
\[ V_c = \frac{1.09}{\gamma_m \sqrt{H}} \left( \frac{H}{a_s} \right)^{1/3} \rho^{1/6} (1 - \sqrt{\rho})^{2/3} K_{IIc} bH \] (N, mm, N/mm^{3/2} units) (40)

Now we could use some data as an example to calculate the shear strength using formulae (40), (37) to compare with the corresponding results using the formulae of the CEB-FIP Model Code and the ACI 318-89. In the calculations, \( f_c = 30 \text{ MPa}, \rho = 0.02, d = 500 \text{ mm}, H = 555 \text{ mm}, a_s/d = 2, K_{IIc} = 1.2 \text{ MPam}^{1/2} = 38 \text{ N/mm}^{3/2} \) are assumed. The shear stresses calculated using the aforementioned formulae are given in Table 5.

It could be concluded that fracture mechanics applies to the prediction of shear bearing capacity of reinforced concrete members without stirrups. It provides a new tool to get analytical formula for shear fracture problems in reinforced concrete members.

Table 5. The shear stresses calculated using several formulae.

<table>
<thead>
<tr>
<th>Formulae used</th>
<th>ACI 318-89</th>
<th>CEB-FIP</th>
<th>Formula (37)</th>
<th>Formula (40)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear stresses (MPa)</td>
<td>0.913</td>
<td>0.650</td>
<td>0.776</td>
<td>0.745</td>
</tr>
</tbody>
</table>

7. CONCLUSIONS AND DISCUSSIONS

As well known, there are no valid analytical formulae in current various design codes as the shear fracture in concrete and reinforced concrete members are very complex. Many experimental observations show that shear fracture happened in concrete and reinforced concrete members without stirrups are brittle, which prompted that many researchers have attempted to use fracture mechanics as analytical tool for predicting the shear bearing capacities of both the concrete members and the reinforced concrete members without stirrups. Their investigation results show that fracture mechanics applies shear fracture of both concrete members and the reinforced concrete members. The several analytical models based on fracture mechanics proposed by some researchers reveal that these models not only can analytically predict the shear strength of reinforced concrete beams without stirrups, but also physically interpret the shear fracture mechanisms. The formulae (37) and (40) in section 6 deduced from the fundamental of fracture mechanical enable to accurately estimate the size effect in shear fracture, the contributions of the shear span-depth ratio, the reinforcement ratio and the concrete quality to shear strength. Due to mode II fracture domina-
tion in shear fracture both in a few cases of realist pure shear fracture in practical structures and in the shear fracture of reinforced concrete members, it needs to understand the shear fracture mechanics and shear fracture properties of concrete materials. Therefore, it is important to perform more mode II fracture tests for various concrete materials to know the mode II fracture toughness $K_{IIIc}$ and mode II fracture energy $G_{IIF}$ of concrete materials with different strength scales.

In last two decades, many researchers have focused their attentions on seeking mode II fracture testing approaches to carry out mode II fracture tests without mode I component supplement. Their experimental investigation showed that it is very difficult to perform pure mode II fracture tests without mode I component supplement. In this chapter, we mainly introduced our developments on mode II fracture tests on double-edge notched specimens including theoretical analyses, numerical studies and experimental investigations. Using this new developed specimen geometry and loading arrangements, critical mode II stress intensity factor, i.e. mode II fracture toughness $K_{IIIc}$ of concrete materials were measured. The mode II fracture energy $G_{IIF}$ is determined using a practical approach too. These results are beneficial for better understanding mode II fracture properties of concrete. Practically measured mode II fracture parameters introduced in sections 4 and 5 provide sufficient supports for the efforts on applying fracture mechanics to shear fracture of reinforced concrete members without stirrups introduced in section 6. Comparing with intensive fracture tests, mass accumulation of experimental data and good understanding for mode I softening properties of the fracture process zone and the crack propagation investigations for mode I crack in concrete materials, both on understanding for fracture mechanisms and practical experiments on mode II fracture tests on concrete materials are not enough. It is expected that more mode II fracture tests for concrete materials and more researches on development of fracture mechanics model to predict shear fracture of reinforced concrete members will be performed in more laboratories in the future.

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A MODEL APPROACH TO DESCRIBE THE FRESH PROPERTIES OF SELF-COMPACTING CONCRETE (SCC)

EIN MODELLANSATZ ZUR BESCHREIBUNG DER FRISCHBETON-EIGENSCHAFTEN VON SELBSTVERDICHTENDEM BETON (SVB)

UN MODELE DECRIVANT LES PROPRIETES A L'ETAT FRAIS DES BETONS AUTOPLAÇANTS

Timo Wüstholz

SUMMARY

Within the scope of a parameter study the influence of the mixture composition on the fresh concrete properties of self-compacting concrete was investigated. The concrete was modelled as a two-phase system, consisting of the fluid phase “paste” and the solid phase “aggregates”. Thus the consistency control parameters paste volume, mortar volume and the coarse aggregate volume could be transferred into the model parameter excess paste thickness. By means of this model parameter the characteristic values of the standard tests – such as slump flow test and V-funnel test – and the fundamental rheological parameters yield stress and the plastic viscosity could be described.

ZUSAMMENFASSUNG


RESUME

Dans le cadre d'une étude paramétrique, l'influence de la composition sur les propriétés à l'état frais des bétons autoplaçants a été analysée. Le béton a été modélisé par un système bi-phase composé de la phase fluide "pâte pure" et de
T. WUSTHOLZ

la phase solide "granulats". Ainsi les paramètres contrôlant la consistance (volume de la pâte pure de ciment, du mortier et des gros granulats) ont pu être convertis en un paramètre épaisseur de pâte. Ce paramètre permet de décrire les valeurs caractéristiques des tests standards – comme par exemple l'essai d'étallement et l'essai d'écoulement – et les paramètres rhéologiques fondamentaux que sont la limite d'écoulement et la viscosité plastique.

KEYWORDS: Self-compacting concrete, SCC, rheological behaviour, model for SCC, excess paste thickness

1. INTRODUCTION

The flow behaviour (rheological behaviour) of self-compacting concrete is mostly characterized by test methods like the slump flow test and the V-funnel test etc. The equipment is inexpensive and the tests are easy in handling and therefore suitable for use on site. But they have one disadvantage: they provide no fundamental physical flow parameters. Fundamental parameters can only be derived from so-called flow curves which are obtained by means of viscometers or rheometers. A flow curve of a fluid describes the relationship between the shear stress \( \tau \) and the shear rate \( \dot{\gamma} \). Many fluids like water or oil behave like a Newtonian fluid where the shear stress is directly proportional to the shear rate. This relationship is expressed by Eq. (1). The characteristic value of such a Newtonian fluid is the viscosity \( \eta \).

\[
\tau = \eta \cdot \dot{\gamma}
\]

(1)

In contrary to a Newtonian fluid fresh ordinary concrete and also self-compacting concrete starts to flow only if the shear stress exceeds the yield stress \( \tau_0 \). In the simplest way fresh concrete can be modelled by a Bingham fluid (Eq. (2)). In such a case two characteristic parameters are necessary to describe the flow curve. These are the yield stress \( \tau_0 \) and the plastic viscosity \( \eta_{pl} \).

\[
\tau = \tau_0 + \eta_{pl} \cdot \dot{\gamma}
\]

(2)

In the literature papers dealing mostly with the modelling of the rheological parameters \( \tau_0 \) and \( \eta_{pl} \). In this article a simple model approach is presented which is also applied to the standard consistency parameters of the slump flow test and the V-funnel test. In this approach the self-compacting concrete is considered as
a two phase system which consists of the fluid phase paste and the solid phase aggregates.

2. SCOPE OF INVESTIGATION

2.1 Varied parameters

Three self-compacting concrete mixtures (Table 1) were the backbone of a test programme in which the influence of the concrete composition on the rheological behaviour was investigated. The main differences between these three reference concretes were the type of filler (limestone powder LS respective fly ash FA), the equivalent water-cement ratio \((w/c)_{eq}\) and the particle-size distribution of the aggregates (Fig. 1a).

Table 1: Mixture proportions of the reference concretes; filler types: limestone powder (LS) and fly ash (FA)

<table>
<thead>
<tr>
<th>Component</th>
<th>LS (A) Powder Type</th>
<th>FA (A) Combination Type</th>
<th>FA (B) Combination Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength class</td>
<td>C30/37</td>
<td>C45/55</td>
<td>C45/55</td>
</tr>
<tr>
<td>Cement content (CEM II/A-LL 32.5R)</td>
<td>(m_c) [kg/m³]</td>
<td>239</td>
<td>345</td>
</tr>
<tr>
<td>Equivalent water-cement ratio ((w/c)_{eq})</td>
<td>[-]</td>
<td>0.70</td>
<td>0.43</td>
</tr>
<tr>
<td>Superplasticizer (Woermann FM/BV 375, type: PCE)</td>
<td>(m_{SP}/m_c) [% by mass of cement]</td>
<td>1.00</td>
<td>1.05</td>
</tr>
<tr>
<td>Total water content (inclusive water from superplasticizer)</td>
<td>(m_w) [kg/m³]</td>
<td>166.1</td>
<td>169.6</td>
</tr>
<tr>
<td>Viscosity agent (Woermann UW Compound)</td>
<td>(m_{V}/m_c) [% by mass of cement]</td>
<td>0</td>
<td>0.10</td>
</tr>
<tr>
<td>Mass of filler (limestone LS or fly ash FA)</td>
<td>(m_f) [kg/m³]</td>
<td>337</td>
<td>194</td>
</tr>
<tr>
<td>Aggregates (round, river sand and gravel)</td>
<td>(m_A) [kg/m³]</td>
<td>1600</td>
<td>1604</td>
</tr>
<tr>
<td>Powder content (cement and filler)</td>
<td>(m_p) [kg/m³]</td>
<td>576</td>
<td>540</td>
</tr>
<tr>
<td>Water-powder ratio (by volume)</td>
<td>(V_w/V_p) [-]</td>
<td>0.82</td>
<td>0.86</td>
</tr>
<tr>
<td>Paste volume (inclusive 15 litres of air)</td>
<td>(V_{paste}) [litres/m³]</td>
<td>385</td>
<td>383</td>
</tr>
<tr>
<td>Mortar volume (paste and aggregates &lt; 4 mm)</td>
<td>(V_M) [litres/m³]</td>
<td>670</td>
<td>709</td>
</tr>
</tbody>
</table>

\[
(w/c)_{eq} = \frac{m_f}{m_c + 0.4 \cdot m_{FA}}
\]

These reference concretes are identified by the filler type followed by “A” or “B”. Table 2 gives a survey of the parameter variations. Based on the reference concretes LS (A) and FA (A) the paste volume \(V_{paste}\), mortar volume \(V_M\) and also the coarse aggregate volume \(V_{A>8}\) was changed. The change of the coarse aggregate volume \(V_{A>8}\) was controlled by the mass ratio \(m_{A,4/8}\): \(m_{A,8/16}\) whereas the total volume of aggregates, paste and mortar were held constant. To investigate how the particle-size distribution of the aggregates influences the flow behaviour of SCC a set of different particle-size distributions was chosen.
(Fig. 1b) which are lying within the range of the standardized particle-size distributions A16 and C16 according to Appendix L of DIN 1045-2 [1]. For this variation the reference concrete FA (B) has built the base (Table 1). All types of variations are given in Table 2, whereas the italic numbers indicate the reference concrete mixtures. The paste volume $V_{\text{paste}}$ consists of the volume parts of water, cement, filler and admixtures and includes an air content of 15 litres per m³ concrete. All concretes were produced with rounded river sand and gravel. The maximum aggregate diameter was set to 16 mm. The particle-size distributions of the aggregates were characterized by the fineness modulus $k$ (Eq. (3), Table 2). Herein stands $p_A$ for the passed aggregates per sieve, whereas the nine sieves from 0.25 mm up to 63 mm are incorporated. The fineness modulus $k$ increases with an increasing maximum aggregate diameter and an increasing coarse aggregate content from C16 to A16 (Table 2).

$$k = \sum_{j=1}^{9} 1 - \frac{p_{A,j}}{100}$$  \hspace{1cm} (3)

**Table 2: Parameter variations; reference concretes are indicated by italic numbers**

<table>
<thead>
<tr>
<th>Variation</th>
<th>Type of filler</th>
<th>Nr. 1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Paste volume</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$V_{\text{paste}}$ [litres/m³]</td>
<td>LS (A)</td>
<td>335</td>
<td>366</td>
<td>385</td>
<td>396</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>FA (A)</td>
<td>340</td>
<td>370</td>
<td>383</td>
<td>392</td>
<td>-</td>
</tr>
<tr>
<td>Mortar volume</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$V_M$ [litres/m³]</td>
<td>LS (A)</td>
<td>604</td>
<td>653</td>
<td>670</td>
<td>704</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>FA (A)</td>
<td>638</td>
<td>691</td>
<td>709</td>
<td>730</td>
<td>-</td>
</tr>
<tr>
<td>Coarse aggregate volume</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$(m_{A,4/8} : m_{A,8/16})$</td>
<td>LS (A)</td>
<td>294</td>
<td>185</td>
<td>155</td>
<td>17</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>FA (A)</td>
<td>210</td>
<td>160</td>
<td>137</td>
<td>15</td>
<td>-</td>
</tr>
<tr>
<td>Fineness modulus of the aggregates</td>
<td>FA (B)</td>
<td>A 16</td>
<td>AAB16</td>
<td>AB16</td>
<td>B16</td>
<td>C16</td>
</tr>
<tr>
<td>$k$ [-]</td>
<td></td>
<td>4.54</td>
<td>4.33</td>
<td>4.12</td>
<td>3.65</td>
<td>2.81</td>
</tr>
</tbody>
</table>
A model approach to describe the fresh properties of self-compacting concrete (SCC)

Fig. 1: Particle-size distributions of the aggregates; a) reference concretes; A16 and B16 are standing for the standard particle-size distributions according to Appendix L of DIN 1045-2 [1]; b) variations for the concretes including FA (B)

2.2 Rheological investigations

Additionally to the standardized tests like slump flow and V-funnel (dimensions of devices see Fig. 2a) rheological measurements were performed with a concrete rheometer “BTRHEOM” which was developed at LCPC (Laboratoire Central des Ponts et Chaussées Paris, Fig. 2b).

Fig. 2: a) Dimensions of the slump cone and V-funnel; b) photograph of the “BTRHEOM”, inner diameter of the container $D = 240$ mm, height of the sheared concrete sample $h = 100$ mm
The BTRHEOM measures the torque which is necessary to shear a concrete sample at a defined number of revolutions. From the measured torque $\Gamma$ and the number of revolutions $n$ it is possible to derive a $\Gamma$-n-curve by a regression analysis (Fig. 3). Such a $\Gamma$-n-curve can then be transferred into a $\tau$-$\dot{\gamma}$-curve (e.g. Eq.(2)). In the simplest case (Bingham) the measured data can be fitted by a straight line (Eq. (4)). From the torque $\Gamma_{0,B}$ respective the slope of the straight line $A_B$ the yield stress $\tau_0$ respective the plastic viscosity $\eta_{pl}$ can be calculated. If this linear approach is applied, often negative yield stresses – especially for self-compacting concrete – are calculated. But negative yield stresses are physically not possible. These negative yield stresses have also been described by de Larrard et al. [2] and were confirmed by measurements at the Department of Construction Materials in Stuttgart.

\[ \Gamma = \Gamma_{0,B} + A_B \cdot n \]  

(4)

By means of the application of the so-called Herschel-Bulkley approach (Eq. (5) and (6)) negative yield stresses can be avoided.

\[ \Gamma = \Gamma_{0,HB} + A_{HB} \cdot n^b \]  

(5)
\[
\tau = \tau_{0,HB} + a \cdot \dot{\gamma}^b
\] (6)

However, Eq. (6) implies the disadvantage of three curve parameters, whereas only the yield stress \(\tau_{0,HB}\) can physically be interpreted. Hence, a combined method was used in that the Herschel-Bulkley approach was combined with the Bingham approach (modified Bingham method), Eq. (7). The corresponding \(\Gamma\)-n- relationship is given with Eq. (8), see Fig. 3.

\[
\tau = \tau_{0,HB} + \eta_{pl,HB} \cdot \dot{\gamma}
\] (7)

\[
\Gamma = \Gamma_{0,HB} + A_B \cdot n
\] (8)

3 RESULTS

3.1 Modelling of the rheological behaviour of SCC

Self-compacting concrete can be considered as a concentrated suspension in that the solids are dispersed in the fluid phase water, de Larrard [3] and Ferraris et al. [4]. Others, Nielsen [5] and Geiker et al. [6], consider self-compacting concrete as a two phase system which consists of the Bingham-phase mortar in that the coarse aggregates are dispersed. The models of these authors are based on the so-called relative solid concentration \(\phi/\phi_{max}\).

Generally, the relative viscosity of a suspension, i.e. the ratio between the viscosity of a suspension and the viscosity of the fluid phase, can be traced back to the so-called relative solid concentration of the particles \(\phi/\phi_{max}\). Whereas \(\phi\) stands for the volume concentration of the solids in a suspension, \(\phi_{max}\) represents the maximum volume concentration of the particles in the state of the maximum packing and is called maximum packing fraction [7]. The maximum packing fraction \(\phi_{max}\), being controlled by the type of packing, is very sensitive to particle-size distribution and particle shape [7]. In this context the models for concentrated suspension of Mooney [8] and Krieger and Dougherty [9] have to be mentioned which are both based on the well-known model of Einstein [10] that is only valid for dilute dispersed suspensions [7].
The model presented in this article assumes that SCC consists of the fluid phase paste and the solid phase aggregates, this stands in contrast to models of [3], [4], [5], [6]. The solid-fluid interactions respective the solid-solid interactions are controlled by the so-called excess paste thickness $t_{Paste,ex}$ which is equivalent to the half distance of two neighbouring aggregates if the distance is assumed to be independent of the aggregate size (Fig. 4). The excess paste thickness $t_{Paste,ex}$ can be determined if the excess paste volume $V_{Paste,ex}$, that remains after the voids between the aggregates $V_{A,void}$ are filled with paste, is uniformly layered on the surface of the aggregates. Another approach with a particle-size dependent paste thickness is described by Oh et al. [12]. Krell [11] showed that the consistency of ordinary concrete can be expressed as a function of the thickness of excess paste.

![Fig. 4: Model of SCC](image)

The excess paste volume $V_{Paste,ex}$ can be calculated with Eq. (9) where $m_A$ respective $\rho_A$ stands for the mass respective the density of the aggregates. Further is the loose bulk density $\rho_{A,bulk}$ of the aggregates required.

$$V_{Paste,ex} = V_{Paste} - V_{A,void} = V_{Paste} - \frac{m_A}{\rho_A} \left( \frac{\rho_A}{\rho_{A,bulk}} - 1 \right) \quad (9)$$

In this study the loose bulk density was calculated using a 10 litre container according to DIN EN 1097-3 [13]. For this purpose a mass of about 20 kg of dry aggregates 0/16 was first homogenized in the concrete mixer and then filled into the container without any additional compaction as described in [13]. To calculate the excess paste thickness the surface area of the aggregates is required. The aggregates were thereby idealized as spheres. The particle-size distribution of the aggregates is divided into 9 classes according to the sieves of DIN 1045-2.
Appendix L [1], see also Table 3. Each class i is represented by the mean particle diameter $d_i$ between two subsequent sieves. Aggregate particles with a diameter < 0.125 mm are ignored. However, they are not part of the paste volume $V_{\text{Paste}}$ in Eq. (9), since the powder content of the aggregates is indirectly incorporated in the volume of voids $V_{A,\text{void}}$ that is derived from the loose bulk density test.

The results in Table 3 show that the particle fraction 0.25/0.5 mm is dominating with about 48% of the total surface area. This is an evidence for the importance of the fine sand fraction with regard to the workability of concrete respective the flow behaviour of SCC.

Table 3: An example of the calculation of the aggregate surface area per m³ concrete, aggregate particles are considered to be spheres

<table>
<thead>
<tr>
<th>Particle-size distribution</th>
<th>Calculation of the surface area $S_A$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter (sieve opening)</td>
<td>Passed aggregates</td>
</tr>
<tr>
<td>[mm]</td>
<td>[mm] [ % by mass]</td>
</tr>
<tr>
<td>0.125</td>
<td>0.09/0.125 0.0975 0.075 - -</td>
</tr>
<tr>
<td>0.25</td>
<td>0.125/0.25 0.1875 2.86 5.096E+09 5659618.7 18.7</td>
</tr>
<tr>
<td>0.5</td>
<td>0.25/0.5 0.375 14.70 3.274E+09 14544824.3 48.2</td>
</tr>
<tr>
<td>1</td>
<td>0.5/1 0.75 9.21 2.564E+08 4556388.9 15.1</td>
</tr>
<tr>
<td>2</td>
<td>1/2 1.5 5.62 1.956E+07 1390168.6 4.6</td>
</tr>
<tr>
<td>4</td>
<td>2/4 3.0 14.31 6.225E+06 1769876.7 5.9</td>
</tr>
<tr>
<td>8</td>
<td>4/8 6.0 22.39 1.218E+06 1384603.0 4.6</td>
</tr>
<tr>
<td>16</td>
<td>8/16 12.0 27.61 1.877E+05 853704.5 2.8</td>
</tr>
<tr>
<td>31.5</td>
<td>16/31.5 23.75 2.44 2.139E+03 38119.6 0.1</td>
</tr>
<tr>
<td></td>
<td>100.00 23.75 2.44 2.139E+03 38119.6 0.1</td>
</tr>
<tr>
<td></td>
<td>100.00 23.75 2.44 2.139E+03 38119.6 0.1</td>
</tr>
</tbody>
</table>

For a comparison of different particle-size distributions of aggregates the specific surface area $S_{A,\text{spec}}$ can be calculated as the ratio between the surface area $S_A$ related to the mass of the aggregates $m_A$ (Eq. (10)):

$$S_{A,\text{spec}} = \frac{S_A}{m_A} \quad (10)$$

If a spherical paste layer on the aggregates is assumed (Fig. 4), the excess paste thickness $t_{\text{Paste,ex}}$ can be calculated using Eq. (11), whereas $r_i$ is the radius of a particle of class i, $n_i$ is the number of particles per class and $t_{\text{Paste,ex}}$ is the thickness of the paste layer.
\[ V_{\text{Paste,ex}} = \frac{4}{3} \pi \sum_i n_i \left( r_i + t_{\text{paste,ex}} \right)^3 - r_i^3 \]  \quad (11)

In a simplified way, the excess paste thickness can directly be calculated with Eq. (12). This assumes that the thickness of the paste layer is small compared to particle size, because Eq. (12) is only valid for a planar paste layer.

\[ t_{\text{paste,ex,plan}} = \frac{V_{\text{paste,ex}}}{S_A} \]  \quad (12)

For small paste thicknesses both methods coincide. The deviation between both methods is increasing with an increasing paste thickness (Fig. 5).

---

**Fig. 5: Comparison of both methods for calculation of the excess paste thickness after Eq. (11) and Eq. (12)**

It is possible that the calculated excess paste thickness lies below the median particle diameter\(^1\) of the powder which was determined about 10 µm for all investigated SCC mixtures. Two reasons are responsible for this: 1) The void volume was determined in an uncompacted state by filling the aggregates in a container according to DIN EN 1097-3 [13]. 2) The excess paste thickness was assumed to be independent of the aggregate particle size (Fig. 4). Based on a two-dimensional imagination the thickness of the excess paste should be greater than half of the maximum diameter of the particles in the paste. But in reality a three-dimensional aggregate skeleton exists with sufficiently sized voids between the aggregates. Furthermore, also the particles of the paste as well as the

---

\(^1\) 50 % (by volume) of the paste particles are below this diameter
voids between the aggregates are continuously distributed. The calculated excess paste thickness is – despite of a partially small value – a suitable parameter for the evaluation of different aggregate particle-size distributions, this will be shown later. The excess paste thickness, which is given in the following figures, was calculated after the spherical method of Eq. (11).

It can be seen from Eq. (9) and (12) that (for a constant paste volume) both, the void volume $V_{A, \text{void}}$ of the poured aggregates (respective the porosity $\varepsilon = \frac{V_{A, \text{void}}}{(V_A + V_{A, \text{void}})}$) and the specific surface area $S_{A, \text{spec}}$ are controlling the thickness of the paste layer $t_{\text{Paste,ex}}$. Hence, particle-size distributions of aggregates have to be preferred which possess a small specific surface area and a small porosity. Both criteria are fulfilled by particle-size distributions which are lying approximately in the range between AB16 and B16 (Fig. 6a). These particle-size distributions have of a reduced coarse aggregate content, which reduces also the risk of blocking. Fig. 6b) shows the influence of the aggregate particle-size distribution (expressed by the fineness modulus $k$, Table 2) on the slump flow and on the excess paste thickness for a constant paste volume of 352 litres/m³. The maximum paste thickness was calculated for the mixture containing the particle-size distribution AB16. This concrete reached also the maximum slump flow value. It has to be mentioned that the origin of the sand fraction of the aggregates of the SCC mixtures produced with limestone powder LS (A) and fly ash FA (A) differed from those aggregates which were used for the concretes including fly ash FA (B). This is probably one reason for the differences in the porosity shown in Fig. 6a).

Fig. 6: a) Porosity versus specific surface area of poured aggregates with different particle-size distributions; b) slump flow and excess paste thickness versus the fineness modulus $k$
Fig. 7 shows the fresh concrete parameters yield stress $\tau_{0,HB}$, slump flow $sf$, plastic viscosity $\eta_{pl,HB}$ and the V-funnel flow time $t_V$ as function of the excess paste thickness $t_{Paste,ex}$. The different symbols stand for the variation of the paste volume, mortar volume and the coarse aggregate volume (see Table 2). It can be seen from Fig. 7 that the excess paste thickness is a suitable parameter to describe the consistency of SCC.

The differences regarding the filler type and the paste compositions (see Table 1) are leading to a different flow behaviour. The given functions were derived from a regression analysis. The deviations of the yield stresses are greater than those of the plastic viscosities, which can be traced back to the measurement procedure and the method of calculating the yield stresses by an extrapolation. The higher yield stresses of the concretes made of limestone powder compared to the fly ash concretes are also indicated by the reduced slump flow values of the concretes containing limestone powder (Fig. 7a). If greater deviations between model and measured data are accepted it is sufficient to use
A model approach to describe the fresh properties of self-compacting concrete (SCC)

only one function to describe the relationship between the plastic viscosity and the model parameter excess paste thickness (Fig. 7b).

In the slump flow test the flow time $t_{500}$ is normally recorded to get an indication of the apparent viscosity of a SCC mixture. The $t_{500}$-value is equivalent to the time that is measured from the start of the upward movement of the cone until the concrete spread has reached a diameter of 500 mm. The relationship between the flow time $t_{500}$ and the model parameter excess paste thickness (Fig. 8) can be described – independently of the filler type – by one function.

$$y = 0.143x^{-0.986}$$

$R^2 = 0.85$

![Graph](image_url)

*Fig. 8: Flow time $t_{500}$ (slump flow test) as a function of the excess paste thickness $t_{Paste,ex}$*

The similar relationships between the model parameter excess paste thickness $t_{Paste,ex}$ and the plastic viscosity $\eta_{pl,HB}$ (Fig. 7b) respective the flow time $t_{500}$ (Fig. 8) denote that a relationship exists between the plastic viscosity $\eta_{pl,HB}$ and the flow time $t_{500}$. In fact it was found in [14] that the plastic viscosity can be estimated based on the flow time $t_{500}$, independently of the concrete composition. However, the V-funnel flow time $t_V$ could not be described (independently of the concrete composition) as a function of the plastic viscosity $\eta_{pl,HB}$. Probably blocking effects in the vicinity of the V-funnel orifice played a role, see also [16]. Hence, it is better to use the flow time $t_{500}$ to get an indication of the plastic viscosity.
4. CONCLUSION

The flow properties of different self-compacting concretes including fly ash respective limestone powder were investigated by means of standard tests and a fresh concrete rheometer. It was shown that the influence of the content and composition of the aggregates on the flow properties can be described by means of the model parameter excess paste thickness.

The flow parameters yield stress, slump flow and V-funnel flow time depended – beside the model parameter excess paste thickness – also on the paste composition which was different for the concretes including fly ash and limestone powder. The plastic viscosity and the flow time $t_{500}$ from the slump flow test could be described independently of the filler type only as function of the excess paste thickness.

ACKNOWLEDGEMENT

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REFERENCES


ORGANIC POLLUTANTS IN INDOOR AIR – BASICS AND PROBLEMS

ORGANISCHE VERUNREINIGUNGEN DER INNENRAUMLUFT – GRUNDLAGEN UND FEHLER

POLLUANTS ORGANIQUES DANS L'AIR INTERIEUR – BASES ET PROBLEMES

Gerhard Volland, Günter Krause, Dagmar Hansen, Dieter Zöltzer

SUMMARY

The concentration of volatile and semivolatile organic compounds (VOC and SVOC) in indoor air is the result of a complex interaction of sources and the specific situations within a contaminated building. It is influenced by a set of factors which should be taken into account to obtain comparable results. Even the same sources lead to different indoor air concentrations depending on structural situations, room climate, ventilation habits and the season. Indoor air is a dynamic system of which the composition is in principle varying. The parameters, which determine the concentration in indoor air are the source strength and the emission characteristics of the source. The source strength is a function of the diffusion potential and the vapour pressure of the pollutant/emitter and therefore a function of temperature and relative humidity. Besides this, the dilution factor with ambient air depending on varying air exchange rates and ventilation habits influences the concentration of organic pollutants in indoor air. Different results in the same building can normally be ascribed to these problems if all relevant parameters are documented in the test reports. In common the analytical problems determining organic compounds in indoor air after collecting on different absorbers are less important, compared to these influences. This leads to a situation that reported data, without precise information about the boundary conditions during the sampling period, can not be compared or evaluated. The standard deviation for the total analytical procedure (sampling and determination) differs for common pollutants (volatile aliphatic and aromatic organic compounds, persistent semivolatile organochloro-compounds) between 10 and 30 % even if the sampling is carried out under the same conditions with stan-
Standardised methods. Higher differences between two measurements can normally be ascribed to differences of the room climate conditions existing while sampling. These differences lead to the fact that deviations of more than 100% arise in practice. This specific situation complicates the estimation of an average annual concentration of pollutants in indoor air.

ZUSAMMENFASSUNG

Substanz sind für die üblichen flüchtigen und schwer flüchtigen organischen Verbindungen (Aliphate, Aromate, Ester, schwer flüchtige, persistente chlororganische Verbindungen) Schwankungen zwischen 10 und 30 % mit den gegebenen Normverfahren möglich.

**RESUME**

La concentration en composés organiques volatiles et semi-volatiles dans l'air intérieur est le résultat d'interactions complexes entre les sources d'émission et les conditions spécifiques du bâtiment contaminé. Elle est influencée par une série de facteurs dont on doit tenir compte afin d'obtenir des données comparables et donc exploitables. Des sources d'émission similaires peuvent mener à des concentrations différentes, en fonction de la configuration du bâtiment, du climat intérieur, de l'aération et de la saison. L'air intérieur constitue un système dynamique dont la composition n'est en général pas constante. La concentration à l'intérieur dépend essentiellement de l'intensité et des caractéristiques de la source. L'intensité de la source est une fonction du potentiel de diffusion et de la pression de vapeur du polluant et par conséquent fonction de la température intérieure, de la température de la source et de l'humidité relative de l'air. En plus, la dilution variable par l'air extérieur en fonction du taux de renouvellement de l'air et des habitudes d'aération joue un rôle important, et est largement influencée par le climat extérieur (vent). Les divergences de résultats pour un même bâtiment peuvent en général être attribuées à ces facteurs, si tous les paramètres utiles ont été documentés lors de l'échantillonnage. Les fluctuations dues au procédé (échantillonnage et analyse) sont en général nettement inférieures à celles dues à la dynamique de l'air intérieur. En pratique, des divergences de 100 % entre deux mesurages sont courantes. Sans la documentation des conditions marginales lors de l'échantillonnage, les résultats des analyses ne sont ni exploitables ni comparables. La base pour la comparabilité est l'écart-type des mesurages comparatifs réalisés sur des échantillons prélevés dans les mêmes conditions. Selon la substance et sa concentration, des fluctuations entre 10 et 30 % sont possibles avec les méthodes standard pour les composés organiques courants (composés aliphatiques, hydrocarbures aromatiques, esters, composés organiques persistants).

**KEYWORDS:** Volatile and semivolatile organic compounds (VOC, SVOC), indoor air, basics, sampling and determination, limits, errors
1. BASICS

Changed life and working habits lead to the fact that the majority of humans in industrialised countries stay inside of buildings up to approx. 80 % per day [1]. Due to a lot of sources inside (paints, furniture, flooring materials, consumer products and other building materials) in combination with reduced or inadequate ventilation, the average concentrations of organic pollutants in indoor air are mostly 2 – 5 and occasionally more than 100 times higher than in ambient air [1]. This leads a lot of reports dealing with health and comfort problems, which have been associated with the presence of organic compounds in indoor air. Therefore, it should be of special interest to receive reliable data for the concentrations of organic pollutants in indoor air. This can be achieved by choosing suitable measuring and sampling procedures which are adapted to the different problems. Comparable and thus evaluated data can be obtained using the meanwhile existing technical guidelines in combination with a basic knowledge of physical and physicochemical behaviour of sources and emitters. This includes a documentation of the problem to be solved, as well as a complete documentation of structural and climatic boundary conditions during the sampling period in the reports.

In principle, the measurements of organic pollutants in indoor air demands to define the objectives of the measurement before beginning any measurement action. In practice different questions arise. Usually measurements in indoor air are carried out because of vague complaints of users in respect to a "bad" air quality. In this case normally no specific species and source of the indoor air pollution is known. This situation requires a screening for possible reasons of these troubles assumed. In this case no general strategy is possible and almost any individual case is different. Number and effectiveness of possible sources in indoor air are different in almost each individual case. In indoor air generally several hundred single organic compounds in a concentration range between few pg/m³ up to mg/m³ can be detected. These are emitted from different building materials, furniture, cleaning agents and consumer products [2-5]. To clarify the question whether there exists an unusual situation the knowledge of the average concentrations in indoor air is helpful [4]. Besides this, it is necessary to determine the room temperature, the relative humidity and the concentration of CO2 in indoor air of rooms with reported “bad” air quality. In many cases the reported troubles can be traced back to those indoor air parameters. In indoor air of class rooms for example CO2-concentrations of 1500 up to 3000 ppm in
combination with temperatures above 25 °C during the lessons are not unusual [6,7].

Besides these orientating or screening measurements or as a consequence of a first screening the question whether a defined limit or guideline value in indoor air is exceeded. To answer these questions in general, a increased effort is required, especially when a concentration in the range of the limit or guideline value is expected. The guideline value for formaldehyde in indoor air in Germany is given by 120 µg/m³. The confidence interval for these measurements based on two parallel samplings is given by 45 µg/m³ (VDI 4300 Bl. 3). This means that all results obtained between 100 and 140 µg/m³ may describe the same indoor air concentration. This means, that it is not possible to decide whether the given guideline value is exceeded or not based on only two obtained results. This principle problem has to be considered always, when the concentration in indoor air is close to a given limit or a guideline value. Currently, for example, a guideline value for dioxin-like PCBs of 4,7 pg WHO-TEQ/m³ is discussed in Germany. The measurement uncertainty for those dioxin-like PCBs (sum of dioxin-like PCBs calculated as WHO-TEQ) in indoor air is about 0,6 pg WHO-TEQ/m³. This means, that for results within the range of 4,1 and 5,3 pg WHO-TEQ/m³ it is in principle not to decide whether this guideline value discussed is exceeded or not (see table 1) [6]. This means, that any exceeding of a given limit or guide line value based on only one result can not be assessed in principle if the concentration in indoor air is in the range of a limit or guide line value. In practice frequently exceedings of a given limit or guide value are assessed based on just one result. Based on these considerations three different questions and thus three measurement strategies have to be differentiated.
Table 1: Standard deviation of the determination of dioxin-like PCBs in indoor air (sampling and determination) [6]

<table>
<thead>
<tr>
<th>Average Temp. [°C]</th>
<th>23,8</th>
<th>23,5</th>
</tr>
</thead>
<tbody>
<tr>
<td>PCB-Congener</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WHO-TEF</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PCB in indoor air [ng/m³]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>77</td>
<td>0,0001</td>
<td>0,7</td>
</tr>
<tr>
<td>105</td>
<td>0,0001</td>
<td>2,5</td>
</tr>
<tr>
<td>114</td>
<td>0,0005</td>
<td>0,1</td>
</tr>
<tr>
<td>118</td>
<td>0,0001</td>
<td>14,2</td>
</tr>
<tr>
<td>126</td>
<td>0,1</td>
<td>0,02</td>
</tr>
<tr>
<td>156</td>
<td>0,0005</td>
<td>2,7</td>
</tr>
<tr>
<td>157</td>
<td>0,0005</td>
<td>0,3</td>
</tr>
<tr>
<td>167</td>
<td>0,0000 1</td>
<td>1,1</td>
</tr>
<tr>
<td>169</td>
<td>0,01</td>
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</tr>
<tr>
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<td>21,4</td>
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<tr>
<td>Σ (max) ng/m³</td>
<td>22,4</td>
<td>21,5</td>
</tr>
<tr>
<td>WHO-TE min in pg/m³</td>
<td>5,2</td>
<td>5,5</td>
</tr>
<tr>
<td>WHO-TE max in pg/m³</td>
<td>5,2</td>
<td>5,6</td>
</tr>
</tbody>
</table>

- Checking the compliance with a guideline or limit value. Usually the sampling conditions for this problems are clearly fixed by standards as for example in the VDI 4300 Bl. 3.
- Determination of an average concentration during longer periods of time while using the building. Therefore it is often required to define the measurement and sampling conditions, depending on the specific situation of the building and the way how this buildings are used. For example usual ventilation habits differ between classrooms and offices or living spaces. This means, however, that a sampling procedure that is suitable for continuous sampling during longer periods has to be chosen. Usually passive collecting systems are suitable for this. If no passive collecting systems are available it
is necessary to develop a strategy for representative sampling under consideration of seasonable variability [6,8].

- Determination of a maximum concentration ("worst case"), where the conditions are to be co-ordinated to the physicochemical characteristics of the parameter which should be analysed, e.g. vapour pressure (equilibrium concentration in indoor air at normal room temperature).

2. PRELIMINARY TEST METHODS

In normal cases the expected low concentrations of organic pollutants in indoor air do not permit the use of direct monitoring measuring systems. Methods like flame ionisation detector (FID), photo ionisation detector (PID) and photo-acoustic sensor (PAS) for volatile organic compounds can be applied with restrictions. Under defined conditions, these methods may give a fast overview of a possibly existing air pollution. In practice however, these methods show substantial deviations from the actual concentrations in indoor air. These methods just give hints. For semivolatile organic compounds (SVOC) like pesticides, polychlorinated biphenyls (PCB), softeners (phthalates) and various flame retardands in indoor air no directly indicating methods exist up to know. Test tubes which are commercial available, are normally used for occupational air. The detection limits for those tubes are normally within the range of mg/m³. Those tubes are suitable as preliminary test methods, if high indoor air concentrations are expected. Known cross-sensitivities for example for compounds of the classes of aliphates, aromatics, ketones or esters have to be regarded (VDI 4300 Bl. 6). In general preliminary test cannot be used to determine an exact concentration of organic compounds in indoor air.

3. SAMPLING METHODS

The low concentrations of organic pollutants in indoor air requires an enrichment step via a sampling system normally. Sampling of indoor air is usually accomplished at average indoor temperatures of approximately 18 - 24 °C and air humidity between 30 and 60 % (comfort range (VDI 4300 Bl. 6). In principle active and passive sampling techniques are applicable. Active sampling techniques with a pump are usually preferred. The disadvantage of these sampling techniques are, that they give just a “snapshot” of the indoor air situation during the sampling period. Usual changes of the concentration of organic pollutants in indoor air, resulting from user behaviour (ventilation habits) and changes of the
source strength can not be detected. The sampling volume depends on the sampling technology and the expected concentration and varies between 0.001 – 400 m³. Small volumes from 1 to max. 10 l are used in combination with thermal desorption techniques to determine various VOC. Air volumes up to 400 m³ are necessary to determine polychlorinated dibenzodioxins and dibenzofurans (PCDD/F) in indoor air. When collecting volumes of 400 m³ it has to be noticed that the sampling air volume drawn in per hour shall not exceed 1/10 of the room volume. Otherwise a dilution via ambient air can not be excluded. Using passive collecting systems an overview of the average indoor air concentration of a longer period is possible. Passive sampling systems are available for aliphatic and aromatic organic compounds as well as aldehydes [9]. These methods do not allow to control the indoor conditions (ventilation habits, room climate) during the sampling period. Temperature and air humidity can be measured additionally using small continuous registratrating systems. It has to be regarded, that users very often are interested to create worst-case situations to prove that their health problems do have an objective background. Applications and limits of these sampling techniques and aspects of the measuring strategy are given in the standards VDI 4300 Bl. 1 - 8, DIN ISO 16000 Bl. 1-4 as well as E DIN 14412 in detail.

4. ANALYTICAL METHODS

Organic compounds in indoor air were usually detected after absorption on fixed phases, either directly by thermal desorption techniques or after extraction with different solvents with capillary gas chromatography (GC) with different detection systems (e.g. flame ionisation detector FID; Photo ionisation detector PID, mass-spectrometers (MS with low or high resolution). For reactive compounds, such as aldehydes reaction absorption systems (2.4 Dinitrophenylhydra- zin DNPH as fixed phase) are used which forms a stable hydrazone with aldehyde. After extraction the determination of the aldehydes is carried out with high performance liquid chromatography (DIN ISO 16000 Bl.3 and 4). Meanwhile, the appropriate technical standards for a set of indoor air pollutants are existing and are constantly extended. For common volatile organic compounds (VOC) the standard deviation for the total procedure (thermal desorption sampling and determination) in concentration-ranges between 110 µg/m³ (n-hexane) and 16.8 µg/m³ (m-xylene) vary between 5.6 % (n-hexane) and 0.9 % (m-xylene). (DIN ISO 16017 Bl.1). Comparable data can be achieved by sampling
on activated charcoal and desorption with different solvents (VDI 2001 Bl. 2). This shows that the common accuracy of the determination procedures including the sampling procedures are sufficient for common VOC. The determination of semivolatile organic compounds (e.g. biocides, flame retardands) by gaschromatography/ massspectrometry after absorption on PU foam shows recovery rates between 92 % (γ-hexachlorocyclohexane; γ-HCH) and 100 % (p, p, DDT) with a standard deviations between 8 % (γ-HCH) and 21 % (DDT). The use of PU foams for compounds with a boiling point below 200 °C (e.g. 1,2,3 trichlorobenzene) is not adequate. The recovery rates drops under 10 % (EPA TO-4A).

Examples of the use, the problems and the limits of the existing methods are given in the available standards DIN ISO 16000 Bl. 3,4 and 6; DIN EN 16017 Bl. 1 and 2; E DIN 14412; VDI 2001 Bl. 1-3; VDI 3864 Bl. 1und2; VDI 3875 Bl. 1 VDI 3498 Bl. 1 and 2; VDI 4301 Bl. 1 - 3 as well as EPA Compendium Methods TO-4A and TO -10A in detail. The use of these standards is recommended. The detection limits of the analytic procedures applied varies as a function of the method, the compounds and the sampling volume between few pg/m³ (PCDD/F or dioxin-like PCB) [6] up to the µg/m³ range (aldehydes, usual VOCs)[2-4,6,9].

5. ERRORS AND LIMITS

In practice, despite of existing technical standards the results obtained in the same polluted building often vary significantly. In common these differences are not the result of an analytic error but due to the dynamics of indoor air. Very often it is possible to explain those differences as a result of different climatic conditions during sampling in combination with the physicochemical behaviour of the interesting compound. The concentration of organic compounds in indoor air depends on a line-up of varying factors. Indoor air is in principle a dynamic system. The parameters, which determine the concentration in indoor air are the source strength, the physicochemical characteristics of the emitter and the dilution with ambient air depending on the air exchange rates or ventilation habits given. The source strength depends on the diffusion potential and the vapour pressure of the emitter. Therefore, emission is as a function of the temperature of indoor the air and the source, which might be different because of influences of structural conditions like heaters/radiators or direct sunshine on buildings materials. The strength of sources is often also depending on the relative humidity of indoor air, especially for all organic compounds which are steam volatile.
Tab. 2 and 3 give examples for the effects on the concentration of dioxin-like PCBs and PCP in indoor air.

Table 2: Concentration of selected dioxin-like PCBs in indoor air of 5 PCB contaminated buildings in dependence on the season (winter times/summer times) [6]

<table>
<thead>
<tr>
<th>SAMPLING PERIOD</th>
<th>T [°C]</th>
<th>PCB 105</th>
<th>PCB 118</th>
<th>PCB 126</th>
<th>Σ PCB*</th>
<th>WHO-TEQ</th>
</tr>
</thead>
<tbody>
<tr>
<td>A summer</td>
<td>23.6</td>
<td>4.8</td>
<td>38.2</td>
<td>0.02</td>
<td>56.2</td>
<td>10.0</td>
</tr>
<tr>
<td>A winter</td>
<td>22.5</td>
<td>3.8</td>
<td>17.2</td>
<td>0.02</td>
<td>20.6</td>
<td>5.6</td>
</tr>
<tr>
<td>B summer</td>
<td>25.0</td>
<td>4.9</td>
<td>64.1</td>
<td>0.02</td>
<td>85.7</td>
<td>13.6</td>
</tr>
<tr>
<td>B winter</td>
<td>23.0</td>
<td>4.1</td>
<td>41.3</td>
<td>&lt; 0.01</td>
<td>57.4</td>
<td>8.1</td>
</tr>
<tr>
<td>C summer</td>
<td>25.1</td>
<td>5.5</td>
<td>66.6</td>
<td>0.01</td>
<td>90.2</td>
<td>13.7</td>
</tr>
<tr>
<td>C winter</td>
<td>21.5</td>
<td>2.1</td>
<td>18.8</td>
<td>&lt; 0.01</td>
<td>25.6</td>
<td>3.5</td>
</tr>
<tr>
<td>D summer</td>
<td>21.8</td>
<td>2.5</td>
<td>37.1</td>
<td>&lt; 0.01</td>
<td>48.5</td>
<td>7.1</td>
</tr>
<tr>
<td>D winter</td>
<td>17.5</td>
<td>1.2</td>
<td>13.4</td>
<td>&lt; 0.01</td>
<td>18.2</td>
<td>2.6</td>
</tr>
<tr>
<td>E summer</td>
<td>23.9</td>
<td>4.6</td>
<td>12.1</td>
<td>&lt; 0.01</td>
<td>21.8</td>
<td>2.5</td>
</tr>
<tr>
<td>E winter</td>
<td>18.9</td>
<td>2.3</td>
<td>3.8</td>
<td>&lt; 0.01</td>
<td>8.5</td>
<td>1.0</td>
</tr>
</tbody>
</table>

T = temperature; Σ PCB* = Sum dioxin-like PCB

Table 3: Concentration of Pentachlorophenole (PCP) in indoor air of a public library measured under standardised conditions (VDI 4300 Bl. 2)

<table>
<thead>
<tr>
<th>ROOM TEMPERATURE</th>
<th>REL. HUMIDITY</th>
<th>DATE OF SAMPLING</th>
<th>CONCENTRATION OF PCP</th>
</tr>
</thead>
<tbody>
<tr>
<td>28 °C</td>
<td>52 %</td>
<td>August</td>
<td>480 ng/m³</td>
</tr>
<tr>
<td>24 °C</td>
<td>45 %</td>
<td>November</td>
<td>75 ng/m³</td>
</tr>
<tr>
<td>19 °C</td>
<td>36 %</td>
<td>January</td>
<td>15 ng/m³</td>
</tr>
</tbody>
</table>
In many cases it is not considered, that the temperature of the sources depends on structural condition and can vary between 10 and 60 °C depending on the season. Besides these factors, the dilution with ambient air depending on varying air exchange rates influences the concentration of organic pollutants in indoor air basically. This has a substantial influence on the determinable contents of the emitters in indoor air at comparable ambient temperatures. These effects increase with the boiling point of the emitters (see tabs. 2 and 3). This dominating influence can be shown in detail for polychlorinated Biphenyls, especially for dioxinlike PCBs (tab. 2). The PCB-concentration in the indoor air increases with the temperature and is significantly higher in summer as in winter even at almost similar room temperatures. An increase of temperature of approximately 5 °C doubles the PCB concentration in indoor air (tab. 2 and fig. 1) [6,7].

![Fig. 1: Temperature dependence of the emission of dioxin-like PCBs. Average temperature during the sampling period. [6]](image)

The dependence of the emission characteristics on vapour pressure/boiling point can be demonstrated by the different emission characteristics of n-propylbenzole (boiling point 159°C) versus diethyleneglykole (boiling point 245 °C) (see fig. 2) [5]. VOCs with boiling points between 80 and 180 °C lead in general to short term, intensive pollution of the indoor air immediately after application of the products. Compounds with high boiling-points (> 250 °C) lead to smaller, but long lasting emission, as well as to the generation of secondary contaminated surfaces.
Besides emission processes especially for semivolatile compounds adsorption processes have to be considered. Sinks like material surfaces, dust particles have to be taken into account. The lower the vapour pressures of semivolatile compounds is, the more important are these sink processes. The surface concentration of these secondary contaminated surfaces increases over the time. Especially for persistent organic compounds like PCB this effect leads to an indoor situation, where after several years the secondary sources form about 50 % of the emission and thus 50 % of the indoor air concentration [10,11]. As already mentioned house dust is a very effective sink. Thus house dust may directly influence the measurable concentration of semivolatile organic compounds in indoor air.

An example for the significant influence of whirled up dust as a result of indoor activities, here for example a cleaning procedure with a broom, is given in table 4. In house dust the concentrations of PCBs with lower vapour pressures such as PCB 101, PCB 153, PCB 138 and PCB 180 are enriched. These PCB contaminated dust particles whirled up, can be collected on PU-foam as well as the PCBs in the gaseous phase emitting from the given sources [12]. The effect of indoor activities is an significant increase of the PCB concentration in indoor air. Whirling up house dust can also be achieved by normal activities in
class rooms during lessons. The result of this activities may be an increase of the indoor air concentration, compared to measurements carried out under standardised conditions without any inside activities.

Table 4: Polychlorinated Biphenyls (PCB) in indoor air; Comparison of the sampling technique VDI 4301 Bl. 2 and EPA TO-10 A 2; A = Conditions according VDI 4301 Bl. 2; B = During the sampling a cleaning procedures with a broom was performed [12]

<table>
<thead>
<tr>
<th>PARAMETER</th>
<th>SAMPLING METHOD ACCORDING TO</th>
<th>Concentration in ng/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>VDI 4301 BL. 2</td>
<td>EPA COMPENDIUM METHODE TO-10A</td>
</tr>
<tr>
<td>PCB 28</td>
<td>A 25 B 30 A 50 B 25</td>
<td></td>
</tr>
<tr>
<td>PCB 52</td>
<td>A 35 B 40 A 25 B 50</td>
<td></td>
</tr>
<tr>
<td>PCB 101</td>
<td>A 15 B 120 A 10 B 100</td>
<td></td>
</tr>
<tr>
<td>PCB 153</td>
<td>A 10 B 210 A 8 B 180</td>
<td></td>
</tr>
<tr>
<td>PCB 138</td>
<td>A 5 B 240 A 5 B 310</td>
<td></td>
</tr>
<tr>
<td>PCB 180</td>
<td>A 5 B 170 A 5 B 240</td>
<td></td>
</tr>
<tr>
<td>Sum PCB</td>
<td>A 95 B 790 A 103 B 905</td>
<td></td>
</tr>
</tbody>
</table>

The concentration of volatile and semivolatile organic compounds in indoor air is the result of a complex interaction of sources and the specific situation inside of contaminated buildings and the climatic conditions in the surrounding of these buildings. Therefore it varies depending on the present situation. It is influenced by a set of factors which should be considered to obtain comparable results. Even the same sources lead to different indoor air concentrations depending on structural situations, room climate, ventilation habits, activities and the season. Hygienic appraisal of indoor air are normally based on an average annual indoor air concentrations. The given examples for the influences of various parameters on the indoor air concentration prove the difficulties of any estimations of an average annual indoor air concentration.
REFERENCES

[1] EPA United States Environmental Protection Agency – Indoor Environments Division Indoor Air Quality [www.epa.gov/iaq]


STAINLESS STEEL REINFORCEMENT – A SURVEY

NICHTROSTENDE BETONSTÄHLE – EIN ÜBERBLICK

ARMATURES EN ACIER INOXYDABLE – UN APERÇU

Ulf Nürnberger

SUMMARY

World-wide numerous stainless steel reinforcing steels are applied for the purpose of preventive corrosion protection of reinforced concrete structures. In this report the performance characteristics, the corrosion behaviour, practical experiences and existing standards are dealt with in a survey. It is shown that reinforced concrete structures, reinforced with suitable stainless steels can be classified as durable.

ZUSAMMENFASSUNG


RESUME

Mondialement, de nombreuses armatures en acier inoxydable sont utilisées comme mesure de prévention contre la corrosion des constructions en béton ar-mé. Ce rapport donne un aperçu sur les caractéristiques de fonctionnement, le comportement face à la corrosion, des expériences pratiques et les normes exis-tantes. Il s'avère que les constructions en béton armé avec des armatures inox appropriées peuvent être classifiées durables du point de vue de la corrosion.

KEYWORDS: Stainless steel, reinforcement, corrosion, properties, specifications
1 INTRODUCTION

In reinforced concrete structures the concrete guarantees chemical and physical corrosion protection of the unalloyed reinforcement.

Loss of durability in reinforced concrete apart from problems caused by poor design and construction only occurs if the passivation oxide layer is rendered unstable (if depassivation occurs) due to carbonation of the concrete reducing the alkalinity of the pore solution in the hardened cement paste around the steel or to the ingress of chlorides to the steel/concrete interface [1-2].

There are several conventional options open to the designer when long life is required or corrosion is anticipated. One attractive technical solution is to apply a stainless steel based reinforcement [3-8].

Although the initial cost of stainless steel is much higher than that of carbon steel, its use can be justified on the basis that the increase in total project cost is small and is easily overtaken by the benefits of lower maintenance and repair costs, particularly where disruption times and costs for such work are taken into consideration.

Corrosion resistant materials for reinforcement may be used in the following applications:

- structures are exposed to attack of corrosion promoting substances,
- the concrete cover and the concrete quality is – by design or otherwise – reduced relative to the necessary values for the surrounding environmental conditions (e.g. in extremely slender elements),
- special structures have to be built, e.g. connections between precast and cast in place elements or heat insulated joints between the structure and external structural elements (e.g. balconies),
- prefabricated wall- and roof-elements where the reinforcement connects the outer and inner walls,
- non-dense or dense lightweight concrete is designed to reach a required thermal insulation as well as low ownweight,
- cases where access to the structure is strongly limited, making future inspection and maintenance costly, such as in underground structures in aggressive soil
- and where future maintenance is possible but may cause extreme indirect costs due to non-availability, such as in bridges in the main traffic arteries of densely populated areas.

There exist recommendations for a convenient use of stainless steel reinforcement [3, 5, 9, 10]. The decision on which type of stainless steels to use depends on:
- the degree of corrosion protection required,
- cost aspects,
- workability and application characteristics (mechanical and physical properties, weldability).

Typical applications where reductions in maintenance costs warrant the use of ferritic-austenitic and austenitic stainless steels include offshore structures, piers at the sea coast, parts of highway structures subject to de-icing salts or splash, multi-storey car parks, plants for the desalination of sea-water, concrete elements in thermal bath and each kind of repair work. A guidance on locations where use of stainless steel reinforcement is recommended in new highway structures is published in [10]. It is possible to substitute all carbon steel reinforcement on a structure with corrosion resistant reinforcement but this would nearly always too expensive to justify. Replacement with stainless steel reinforcement should be limited to those major components where the consequences of future repair are likely to be highly disruptive and costly and the possibility of chloride attack is likely.

Components of highway structures that may meeting these requirements include
- Decks of bridges carrying heavily trafficked roads over busy railway lines with limited possessions for repair,
- exposed piers and columns in centre reserves (but not deeply buried elements),
- deck slabs where access for maintenance is going to be very difficult because of traffic levels.

An alternative approach is to use stainless steel reinforcement selectively in conjunction and also contact with carbon steels. It is not envisaged that stainless steel will replace any really significant part of the massive tonnage of the present carbon steel reinforcement output.
The applications of stainless steels must not be restricted to chromium-nickel-(molybdenum) steels with austenitic and ferritic-austenitic structure. Ferritic chromium-alloyed steels might be the best choice in moderate aggressive environments, e.g. in carbonated normal and lightweight concrete if chloride attack can be excluded, where the higher resistance of the more expensive stainless steels is not necessary.

2 STEEL TYPES [1,11]

The term stainless steel does not refer to a single specific material but rather to a group of corrosion resistant high alloyed steels, which in contrast to unalloyed steels do not show general corrosion and noticeable rust formation in normal environmental conditions (atmosphere, humidity) and in aqueous, nearly neutral to alkaline media. Basic requirement for the before-said reaction is a minimum concentration of that steel on particular alloying elements and the existence of an oxidising agent (e.g. oxygen) in the surrounding medium. This causes a passivation of the surface. „Passivity“ describes a condition that produces a strong inhibition of the reaction of resolving iron after forming a passive layer on the surface. Chromium, in particular, is an element that tends to passivation. A self-forming inert chromium oxide layer on the surface of the material protects against corrosion. In the event of the protective surface layer being damaged, it is self-healing in the presence of oxygen. This property is transmitted on iron resp. steel through alloying: General corrosion decreases in corrosion-promoting media contrary to the content of chromium (see Fig. 1). The content of chromium that causes passivity when exceeded depends on the attacking agent. The content of chromium in water and in the atmosphere should at least be 12 M.-%. Corrosion resistance may be further improved by additions of further alloying elements. Chromium, molybdenum and nitrogen are important elements in relation to pitting corrosion. Nickel especially increases corrosion resistance in acid media.
Changing the balance of the alloying elements (chromium, nickel, molybdenum, nitrogen, titanium and others) will influence the structure as well as the other properties such as corrosion behaviour, mechanical and physical properties and weldability. Therefore members of the stainless steel family are usually grouped in groups having the same metallographic structure. Within the area of concrete reinforcement three types of stainless steels are in question and are available in the adequate product form. These are ferritic, austenitic and ferritic-austenitic (duplex). Interest in the use of these alloys as reinforcing steel for concrete is due to their increased resistance to corrosion particularly in chloride containing media, but particular technological characteristics are aimed at with regard to processing and application, as well. However increasing the alloy level the cost of the material will also increase. Therefore it is important to select steel types at an alloy level which are sufficiently corrosion resistant for the job to be done and with sufficient mechanical properties and weldability.

In common conditions, that prevail in construction engineering (attack of light acid to alkaline aqueous media), ferritic steels with about 11 to 30 M.-% of chromium have a sufficient resistivity against general corrosion. With an addition of a sufficient content of chromium and molybdenum up to about 2 M.-%, resistivity against pitting corrosion can be achieved as well. Besides, ferrites have a high resistivity to stress corrosion cracking in an environment containing chlorides. Above all, if you assume comparable contents of chromium, the reac-
tion of ferritic steels towards crevice corrosion is much more adverse than it is e.g. at austenitic steels.

Ferritic steels are ferromagnetic. An advantage of these steels in comparison with austenites is the higher yield stress in the as-rolled condition. Advers is the low fracture-elongation, the more difficult workability and the brittleness at low temperatures. The workhardening during cold forming is low in comparison with austenitic steels. They are not so readily weldable as the other types.

**Austenitic steels** have between 17 to 25 M.-% of chromium and 8 to 26 M.-% of nickel. These steels are especially used because of their positive corrosion properties and their superior workability in comparison with other stainless steels. In case of a proper content of alloy, they have got a high resistivity to general corrosion, pitting corrosion and crevice corrosion, but are sensitive to stress corrosion cracking in their typical compound with about 10 M.-% nickel. The resistance to pitting corrosion, crevice corrosion and stress corrosion cracking can be improved with an addition of chromium, molybdenum and nickel.

Austenitic steels are not ferromagnetic. They have a higher toughness and a much better weldability but a lower yield stress in the as-rolled condition than ferritic steels. The tendency to workhardening is very pronounced. For that reason austenites can increase their strength evident by means of cold forming without unacceptable reduction of deformability. The ductility of austenitic stainless steel always exceeds that of conventional bars and they have a very high toughness and good ductility properties at low temperatures. In seismic areas, austenitic steels are often used in reinforced concrete structures, as their strength and ductility intensify the material’s specific deformation energy. This is advantageous for absorbing the impact of a violent earthquake.

**Ferritic-austenitic (duplex) steels** have a binary structure of ferrite and austenite. The typical range of their chemical analysis is 22 to 28 M.-% of chromium, 4 to 8 M.-% of nickel. Molybdenum can be added in order to improve the corrosion resistivity. These steels combine good properties of ferritic steels (high yield strength) and austenitic steels (good ductility, improved corrosion properties). Owing to their excellent mechanical properties (high yield strength, good ductility) in already the as-rolled condition and the very high resistivity to chloride attack, duplex steels are of interest as material for reinforcement.
### Table 1a: Ferritic steel - chemical composition according to [13]

<table>
<thead>
<tr>
<th>Steel designation</th>
<th>Steel No.</th>
<th>C</th>
<th>Si</th>
<th>Mn</th>
<th>P</th>
<th>S</th>
<th>N</th>
<th>Cr</th>
<th>Mo</th>
<th>Ni</th>
<th>Ti</th>
<th>Others</th>
<th>Grades</th>
</tr>
</thead>
<tbody>
<tr>
<td>X3CrNb17</td>
<td>1.4511</td>
<td>≤ 0.05</td>
<td>≤ 1.00</td>
<td>≤ 1.00</td>
<td>≤ 0.040</td>
<td>≤ 0.015</td>
<td>-</td>
<td>16.0 to 18.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Nb: 12xC to 1.00</td>
<td>InE235</td>
</tr>
<tr>
<td>X2CrNi12</td>
<td>1.4003</td>
<td>≤ 0.03</td>
<td>≤ 1.00</td>
<td>0.50 to 1.50</td>
<td>≤ 0.040</td>
<td>≤ 0.015</td>
<td>≤ 0.03</td>
<td>10.5 to 12.5</td>
<td>-</td>
<td>0.30 to 1.00</td>
<td>-</td>
<td>C+N≤ 0.03</td>
<td>InE500</td>
</tr>
</tbody>
</table>

### Table 1b: Austenitic steels - chemical composition according to [13]

<table>
<thead>
<tr>
<th>Steel designation</th>
<th>Steel No.</th>
<th>C</th>
<th>Si</th>
<th>Mn</th>
<th>P</th>
<th>S</th>
<th>N</th>
<th>Cr</th>
<th>Cu</th>
<th>Mo</th>
<th>Ni</th>
<th>Others</th>
<th>Grades</th>
</tr>
</thead>
<tbody>
<tr>
<td>X5CrNi18-10</td>
<td>1.4301</td>
<td>≤ 0.07</td>
<td>≤ 1.00</td>
<td>≤ 2.0</td>
<td>≤ 0.045</td>
<td>≤ 0.030</td>
<td>≤ 0.11</td>
<td>17.0 to 19.5</td>
<td>-</td>
<td>-</td>
<td>8.0 to 10.5</td>
<td>-</td>
<td>InE235, InE500, InE650</td>
</tr>
<tr>
<td>X2CrNi18-10</td>
<td>1.4311</td>
<td>≤ 0.030</td>
<td>≤ 1.00</td>
<td>≤ 2.0</td>
<td>≤ 0.045</td>
<td>≤ 0.030</td>
<td>0.12 to 0.22</td>
<td>17.0 to 19.5</td>
<td>-</td>
<td>-</td>
<td>8.0 to 11.5</td>
<td>-</td>
<td>InE235, InE500, InE650</td>
</tr>
<tr>
<td>X5CrNiMo17-12-2</td>
<td>1.4401</td>
<td>≤ 0.07</td>
<td>≤ 1.00</td>
<td>≤ 2.0</td>
<td>≤ 0.045</td>
<td>≤ 0.030</td>
<td>≤ 0.11</td>
<td>16.5 to 18.5</td>
<td>-</td>
<td>2.0 to 2.5</td>
<td>10.0 to 13.0</td>
<td>-</td>
<td>InE235, InE500, InE650</td>
</tr>
<tr>
<td>X2CrNiMoN17-13-3</td>
<td>1.4429</td>
<td>≤ 0.030</td>
<td>≤ 1.00</td>
<td>≤ 2.0</td>
<td>≤ 0.045</td>
<td>≤ 0.015</td>
<td>0.12 to 0.22</td>
<td>16.5 to 18.5</td>
<td>-</td>
<td>2.5 to 3.0</td>
<td>11.0 to 14.0 b</td>
<td>-</td>
<td>InE235, InE500, InE650</td>
</tr>
<tr>
<td>3CrNiMo17-13-3</td>
<td>1.4436</td>
<td>≤ 0.05</td>
<td>≤ 1.00</td>
<td>≤ 2.0</td>
<td>≤ 0.045</td>
<td>≤ 0.030</td>
<td>≤ 0.11</td>
<td>16.5 to 18.5</td>
<td>-</td>
<td>2.5 to 3.0</td>
<td>10.5 to 13.0 b</td>
<td>-</td>
<td>InE235, InE500, InE650</td>
</tr>
<tr>
<td>X6CrNiMoTi17-12-2</td>
<td>1.4571</td>
<td>≤ 0.08</td>
<td>≤ 1.00</td>
<td>≤ 2.0</td>
<td>≤ 0.045</td>
<td>≤ 0.030</td>
<td>-</td>
<td>16.5 to 18.5</td>
<td>-</td>
<td>2.0 to 2.5</td>
<td>10.5 to 13.5 b</td>
<td>Ti:5xC to 0.70</td>
<td>InE235, InE500</td>
</tr>
<tr>
<td>X1NiCrMoCu25-20-5</td>
<td>1.4539</td>
<td>≤ 0.020</td>
<td>≤ 0.70</td>
<td>≤ 2.0</td>
<td>≤ 0.030</td>
<td>≤ 0.010</td>
<td>≤ 0.15</td>
<td>19.0 to 21.0</td>
<td>1.2 to2,0</td>
<td>4.0 to 5.0</td>
<td>24.0 to 26.0</td>
<td>-</td>
<td>InE235, InE500, InE650</td>
</tr>
<tr>
<td>X8CrMoNaB17-8-3</td>
<td>1.4597</td>
<td>≤ 0.10</td>
<td>≤ 2.00</td>
<td>6.5 to 8.5</td>
<td>≤ 0.040</td>
<td>≤ 0.030</td>
<td>0.015 to 0.30</td>
<td>16.0 to 18.0</td>
<td>2.0 to3,5</td>
<td>≤ 1.00</td>
<td>≤ 2.0</td>
<td>0.0005 to 0.0015</td>
<td>-</td>
</tr>
</tbody>
</table>

### Table 1c: Ferritic-austenitic (duplex) steels - chemical composition according to [13]

<table>
<thead>
<tr>
<th>Steel designation</th>
<th>Steel No.</th>
<th>C</th>
<th>Si</th>
<th>Mn</th>
<th>P</th>
<th>S</th>
<th>N</th>
<th>Cr</th>
<th>Cu</th>
<th>Mo</th>
<th>Ni</th>
<th>Others</th>
<th>Grades</th>
</tr>
</thead>
<tbody>
<tr>
<td>X2CrNiMoN222-5-3</td>
<td>1.4462</td>
<td>≤ 0.030</td>
<td>≤ 1.00</td>
<td>≤ 2.00</td>
<td>≤ 0.035</td>
<td>≤ 0.015</td>
<td>0.10 to 0.22</td>
<td>21.0 to 23.0</td>
<td>-</td>
<td>2.50 to 3.50</td>
<td>4.50 to 6.50</td>
<td>-</td>
<td>InE500, InE650, InE800</td>
</tr>
</tbody>
</table>
Table 1 lists the chemical composition of the main stainless steels suitable for the manufacturing of reinforcing stainless steels. This survey was taken from a common European standard [12] which at this time is in preparation. It was added by the ferritic steel type 1.4003 which is used in Germany and other countries. The numbers of the steels are according to European Standard codes as given in EN 10088 [13].

For particular types of corrosion, e.g. pitting corrosion and stress corrosion cracking, the existence of a passive layer is a necessary requirement. Because of that, passive steels are resistant against general corrosion, but are sensitive to local corrosion in presence of specific media (e.g. chloride ions) in case of an insufficient content of alloy. There are four types of corrosion of stainless steels to be observed: general corrosion, intergranular corrosion, pitting corrosion and stress corrosion cracking.

3 PRODUCTION OF STAINLESS STEEL REINFORCEMENT [3,1,16]

For application in concrete structures, ferritic, austenitic and ferritic-austenitic (duplex) steels can be produced as ribbed bars within the normal range of strength and deformability requirements. Up to 14 mm diameter the bars are available in rings permitting the confection of any shape and length of reinforcing bar. Above 14 mm diameter the bars are supplied in straight lengths (length up to 12 m are available in the UK).

One of the initial problems in producing stainless steel reinforcing bars was that the yield strength $R_{p0.2}$ of ferritic and above all austenitic as-rolled bares were approximately the same as those of mild steel. The general mechanical properties in the annealed condition are such that the yield strength of ferritic and austenitic types are about 300 N/mm$^2$ respectively 200 N/mm$^2$ whereas the corresponding values for duplex steels are higher (400 - 480 N/mm$^2$). Therefore no ferritic or austenitic standard steel in the normal as-rolled condition would have sufficient strength.

However, in order to meet the requirement for use as reinforcement in concrete the strength of the steels must be increased. As these steels had a metallurgical structure incapable hardened significantly by heat treatment other methods of increasing strength had to be pursued.

Subsequent treatment, either special heat treatment or cold and warm working, the latter also with a nitrogen addition, will enable high yield reinforcement
strength to be reached. These processes are however complicated and increase the high material cost of stainless steel.

**Ferritic steels** in the as-rolled condition have a higher yield strength than austenitic steels. There is a certain probability that the bars may be further strengthened by cold twisting [14] or drawing and cold rolling [15]. These processes can be facilitated by employing a special alloy composition. In this, the carbon and nitrogen contents are limited to avoid hardening after cooling from the austenite phase. The steel retains sufficient strength and deformation properties after cold deforming from 4 to 14 mm diameter. In addition to strengthening the bars, twisting is also an effective method of removing millscale, which has been found to aggravate pitting corrosion and was previously removed by pickling and shot blasting.

Acceptable high yield reinforcing bar strengths can be obtained from **austenitic stainless** steels. The lower dimensions from 4 to 14 mm may be strengthened by means of cold working (drawing and rolling) [15-17]. For the austenitic types cold working results in a reduction of the elongation from 40% to 20 - 25%, which is beneficial for the function of the rebars in concrete.

The literature [18] sometimes makes reference to the possibility of a somewhat reduced corrosion resistance of cold worked austenitic stainless steel whereas in duplex materials this is not the case. Cold working of austenitic stainless steel may cause a transformation of some of the austenite into martensite. Alloys with a lower content of alloying elements (e.g. 1.4301) are more prone to develop martensite than alloys with a higher content (e.g. 1.4539) (see section 4.2). Martensite is in the position to favour pitting corrosion. However, the amount of cold work of reinforcing steel does not exceed about 35% which results not in a damaging martensite formation and a reduction of the pitting corrosion resistance [15].

For small dimensions (< 12 mm) also warm working at reduced temperature may be used for increasing the strength of austenitic steels resulting in mechanical properties similar to those obtained by cold working [16,17]. An effective solution for large diameter bars up to 40 mm for ribbed bars and 50 mm for plain bars is the combination of using a modified composition (an addition of 0.15/0.20 M.-% nitrogen) and the warm working process.

Owing to their excellent mechanical properties in the as-rolled condition, duplex stainless steels are of interest as materials for reinforcements. In Ger-
many [15] such wires are cold deformed, in Italy [19] they are as-rolled or cold deformed.

In principle manufacture of stainless steel reinforcement by hot and cold deforming does not distinguish from production of mild steel reinforcement. Another development, which can significantly reduce the cost, involves producing a stainless steel clad reinforcing bar [20]. In this approach, a core of ordinary steel is encapsulated in a stainless steel sheath to resist corrosion. However, the difficulties associated with inserting the core and fusing the metals together added to the cost which thereby offset the savings resulting from the use of a cheaper core. Furthermore, of pinholes were present in the cladding there was a potential problem of 'undercutting' corrosion [17]. At that time improved products are on the market in UK.

4 STRUCTURAL PROPERTIES

Mechanical and physical properties as well as welding behaviour are very important in order to evaluate the ability of any material to withstand the expected loads during the designed service life. These depend on the method of manufacture, material composition respectively microstructure and bar size.

4.1 Mechanical properties

The mechanical properties of stainless steels that are of main concern to the designer are characteristic strength, ultimate tensile strength and elongation. The stress-strain-behaviour of austenitic and duplex grades differs from that of carbon steels in that they do not exhibit a well-defined yield point when test pieces are submitted to tensile load. To characterise the design strength of such materials, proof strengths are used and are determined as the stress $R_{p0.2}$ of 0.2 %. After [9] a modulus of elasticity for austenitic and ferritic-austenitic stainless steel reinforcement of 200 KN/mm$^2$ may be used in design, except for the austenitic steel 1.4529, which has a modulus of 195 KN/mm$^2$. The Tables 2 – 4 show typical properties for different steel grades from UK, Germany and Italy.

Temperature influence

Austenitic stainless steels in the warm deformed condition retain considerably higher strength than carbon steels, ferritic and ferritic-austenitic(duplex) stainless steels at elevated temperatures [4,9]. At temperatures up to 500 °C, there is negligible reduction in the 0.2 % proof stress. This suggests that con-
crete elements reinforced with austenitic stainless steel will behave better in fire than conventionally reinforced elements with the same depth of cover.

The increase in strength of stainless steels from cold working process gradually reduces with increasing temperature. At 500 °C, austenitic stainless steels exhibit a marginal decrease in the 0.2 % proof strength and a significant reduction in the ultimate tensile strength. The strength of heated cold deformed reaches the strength of annealed material at a little over 800 °C.

Table 2: Mechanical properties of stainless reinforcing steels in UK (from [16] and steel maker information)

<table>
<thead>
<tr>
<th>steel grade</th>
<th>chemical composition</th>
<th>condition</th>
<th>bar size</th>
<th>yield stress</th>
<th>tensile stress</th>
<th>elongation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.4401 austenitic</td>
<td>X5CrNiMo 17-12-2</td>
<td>warm worked</td>
<td>10</td>
<td>865</td>
<td>1000</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>20</td>
<td>745</td>
<td>880</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>32</td>
<td>620</td>
<td>775</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>40</td>
<td>550</td>
<td>685</td>
<td>25</td>
</tr>
<tr>
<td>1.4401 austenitic</td>
<td>X5CrNiMo 17-12-2</td>
<td>as rolled</td>
<td>25</td>
<td>279</td>
<td>579</td>
<td>52</td>
</tr>
<tr>
<td></td>
<td>X5CrNiMo 17-12-2</td>
<td>cold twisted</td>
<td>20</td>
<td>660</td>
<td>780</td>
<td>28</td>
</tr>
</tbody>
</table>

1) minimum values  2) values of specific specimens

Table 3: Mechanical properties of stainless reinforcing steels in Germany (from [15] and steel maker information)

<table>
<thead>
<tr>
<th>steel grade</th>
<th>chemical composition</th>
<th>condition</th>
<th>bar size</th>
<th>yield stress</th>
<th>tensile stress</th>
<th>elongation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.4429 austenitic</td>
<td>X2CrNiMoN 17-13-3</td>
<td>hot rolled</td>
<td>10</td>
<td>880</td>
<td>990</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>20</td>
<td>790</td>
<td>900</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>32</td>
<td>630</td>
<td>790</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>40</td>
<td>550</td>
<td>790</td>
<td>30</td>
</tr>
<tr>
<td>1.4571 austenitic</td>
<td>X6CrNiMoTi 17-12-2</td>
<td>cold rolled</td>
<td>10¹)</td>
<td>456</td>
<td>599</td>
<td>39</td>
</tr>
<tr>
<td>1.4462 ferr.-aust.</td>
<td>X2CrNiMoN 22-5-3</td>
<td></td>
<td>7¹)</td>
<td>870</td>
<td>934</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8¹)</td>
<td>518</td>
<td>608</td>
<td>16</td>
</tr>
<tr>
<td>1.4003 ferritic</td>
<td>X2CrNi 12</td>
<td>hot rolled</td>
<td>~350</td>
<td>~490</td>
<td>~25</td>
<td></td>
</tr>
</tbody>
</table>

1) 6-14 mm is possible  2) no reinforcing steel  3) values of specific specimens  4) minimum values
Table 4: Mechanical properties of stainless steels in Italy (from [19] and steel maker information)

<table>
<thead>
<tr>
<th>steel grade</th>
<th>chemical composition</th>
<th>condition</th>
<th>bar size</th>
<th>yield stress(^1)</th>
<th>tensile stress(^1)</th>
<th>elongation(^1)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>mm</td>
<td>N mm(^{-2})</td>
<td>N mm(^{-2})</td>
<td>%</td>
</tr>
<tr>
<td>1.4301</td>
<td>X5CrNi 18-10</td>
<td>cold finished</td>
<td>10</td>
<td>671</td>
<td>831</td>
<td>21.4</td>
</tr>
<tr>
<td>1.4307</td>
<td>X2CrNi 18-9</td>
<td>hot rolled</td>
<td>20</td>
<td>761</td>
<td>864</td>
<td>27.9</td>
</tr>
<tr>
<td>1.4401</td>
<td>X5CrNiMo 17-12-2</td>
<td></td>
<td>32</td>
<td>754</td>
<td>863</td>
<td>25.9</td>
</tr>
<tr>
<td>1.4404</td>
<td>X2CrNiMo 17-12-2</td>
<td></td>
<td>40</td>
<td>717</td>
<td>878</td>
<td>31.1</td>
</tr>
<tr>
<td>1.4571</td>
<td>X6CrNiMoTi 17-12-2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.4462</td>
<td>X2CrNiMoN 22-5</td>
<td>cold finished</td>
<td>10</td>
<td>950</td>
<td>1059</td>
<td>14.0</td>
</tr>
<tr>
<td>1.4362</td>
<td>X2CrNiN 23-4</td>
<td>as rolled</td>
<td>18</td>
<td>485</td>
<td>668</td>
<td>-</td>
</tr>
</tbody>
</table>

\(^1\)Values of specific specimens

For example, liquefied natural gas is stored at a **low temperature** of -165 °C and liquid oxygen below -190 °C. Any materials forming part of a containment system for such gases must have satisfactory and predictable properties at these temperature to avoid failure. Austenitic stainless steel reinforcement, which retains ductility to temperatures as low as -196 °C, is suitable for use in such applications [4,9], unlike carbon steel, which exhibits a transition from ductile to brittle behaviour well above this temperature. The ultimate tensile strength and the 0.2 % proof stress increase slightly with descending temperature. The elongation decreases. Ferritic-austenitic (duplex) and above all ferritic stainless steels undergo a marked decrease in toughness at sub-zero temperatures. These steels are not recommended for cryogenic applications.

### 4.2 Physical properties

The important physical properties of stainless steel considered in relation to application in concrete are: density, thermal conductivity, coefficient of thermal expansion and magnetic permeability. In Table 5 typical values of these parameters for different types of stainless steel in the annealed condition are collected.

From the structural point of view, the most important physical property is the coefficient of linear thermal expansion [3]. The coefficients of thermal expansion of ferritic steel and concrete are more or less the same (1.2 and 1.1 x 10\(^{-5}\) °C\(^{-1}\) respectively). In comparison, the coefficient of thermal expansion of austenitic stainless steel is higher (1.7 x 10\(^{-5}\) °C\(^{-1}\)). If a concrete structure with austenitic reinforcement is exposed to high temperatures, tensile stresses will be produced in the uncracked concrete as a consequence of the different thermal...
coefficient of steel and concrete. This may in theory cause some minor defects in the contact zone and expansion cracking, particularly in heavily reinforced sections. However, there is no practical evidence or laboratory results supporting this assumption. Compared to carbon steels, the higher coefficients of thermal expansion for the austenitic steels, and the lower thermal conductivities, may rise to greater welding distortions (section 4.3).

Table 5: Physical properties of stainless steel

<table>
<thead>
<tr>
<th></th>
<th>Density g/cm³</th>
<th>Thermal conductivity W/m·°C</th>
<th>Specific heat J/g·°C</th>
<th>Coefficient of thermal expansion cm/cm·°C</th>
<th>Magnetic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ferritic steel</td>
<td>7,7</td>
<td>23</td>
<td>0.46</td>
<td>1,2x10⁻⁵</td>
<td>Yes</td>
</tr>
<tr>
<td>Austenitic steel</td>
<td>7,8-8,0</td>
<td>12-15</td>
<td>0.44</td>
<td>1,7x10⁻⁵</td>
<td>No</td>
</tr>
<tr>
<td>Martensitic steel</td>
<td>7,7</td>
<td>23</td>
<td>0.46</td>
<td>1,2x10⁻⁵</td>
<td>Yes</td>
</tr>
<tr>
<td>Duplex steel</td>
<td>7,7</td>
<td>20</td>
<td>0.44</td>
<td>1,3x10⁻⁵</td>
<td>Yes</td>
</tr>
</tbody>
</table>

Ferritic stainless steels are (ferro-)magnetic, as are carbon steels. The magnetic behaviour of the various types of austenitic steel varies, but they have low magnetic permeabilities compared to other ferrous steels and are generally considered to be non-magnetic.

Relative magnetic permeability is defined as the ratio of the magnetic flux density produced in the material to that produced in free space by the same magnetising force; thus the lowest achievable magnetic permeability is 1. For austenitic stainless steels magnetic permeability depends on chemical composition and production process. Concerning the steel grade magnetic permeability decreases in the designation order 1.4301 - 1.4401 - 1.4436 - 1.4429 - 1.4529. However, the values varies with production process, e.g. values for cold-drawn steel bar are greater than for bar that has been warm-worked. Heavy cold working, particularly of the lean alloyed austenitic steel, can also increase magnetic permeability; subsequent annealing would restore the non-magnetic properties. Cold working produces phase transformation from austenite to martensite (section 3). These strain induced martensite phases are magnetic and increases magnetic permeability. In particular the more highly with chromium, manganese, nickel, molybdenum and nitrogen alloyed grades have a increased austenite stability and are effectively non-magnetic after cold deformation (lit. in [21]). So, bars required to have a low permeability (~ 1.005) must be hot-rolled and/or of a specific composition.
4.3 Weldability

All stainless steel can be welded either to themselves or to carbon steel provided that necessary precautions are taken [4,9]. However, welding method and type of weld should be considered. Welding of reinforcement can be made by resistance welding as well as metal arc welding. As most materials used for reinforcement have been strengthened by cold working, reduction of strength at the weld is possible depending of the heat input applied.

Resistance welding is the most widely used welding method in factories. For instance, it is used for prefabrication of mesh reinforcement. Resistance welding having generally the lowest heat input will have the least effect on the properties. On the other hand, it requires well adjusted parameters in order to obtain a mechanical connection which is able to transfer sufficient force. This is done by optimising the electrical parameters along with the press force by welding.

Gas metal arc welding (MIG/MAG) is the most frequently used method for welding carried out on site. It is a very rational method for joining crossing rebars. When arc welding reinforcing bars some loss of tensile and yield strength may result from the welding heat. Consequently it is advisable to adjust the welding parameters resulting in shortest possible welding time and the best possible gas shielding. The latter is in order to minimise oxide formation. Gas mixture used is 96 % argon, 3 % CO$_2$ and 1 % hydrogen. If the weld products (tem-per colours) followed by high heat input metal arc welding are not completely removed, corrosion resistance is reduced. Pickling or shot-blasting the weld can often solve this problem, but is not always on construction sites.

The weldability of stainless steel depends on its structure and chemical composition. The weldability of the steel types is best for the austenitic types, similar but more restricted for the duplex materials and very limited for the ferritic ones. Weldability is improved by decreasing the carbon content, increasing the nickel content and by stabilisation. As a rule low carbon grades of stainless steel with max. C = 0.03 % or with titanium or niobium stabilised grades can be welded without fear of any detrimental effect.

In comparison with carbon steel, the higher thermal expansion of austenitic stainless steel coupled with its lower value of thermal conductivity, increases the possibility of distortion occurring during the welding process. However, the higher electrical resistance of stainless steel is an advantage because it results in
the generation of more heat for the same current. Together with the low heat conductivity this can be advantageous when resistance welding processes are used.

When welding the duplex stainless steels, it is the cooling rate which controls the microstructure, therefore the heat input should be controlled in conjunction with the material thickness to obtain the correct weld structure.

Because stainless steel concrete reinforcing bars have different chemical compositions it is important to select welding electrodes or wires which result in welds with identical or better composition to those of the bars. That provide weld filler with corrosion resistance properties as nearly identical to the base metal. Proper weld rod selection not only preserves corrosion resistance properties, but is also important in achieving optimum mechanical properties.

When welding stainless steel to carbon steel the electrode or wire has to be higher alloyed than the stainless steel that is to be welded in order to compensate for the diluting effect of the carbon steel. The chemical composition of weld, depending only on the welding electrode or wire used, shall not be too lean in alloying elements as otherwise brittle welds are the result. As a minimum the weld should have the composition of stainless steel type 1.4301. This can be achieved with an electrode or wire that contains at least 23 % chromium and 12 % nickel.

5 CURRENT SPECIFICATIONS

UK

BS 6744 [22] was one of the first standards covering stainless steel reinforcement. This standard specifies stainless hot-rolled and cold-worked steel bars to achieve characteristic strength levels of 500 N/mm² or higher. Strength grades are defined in Table 6. The 200 grade steel is only available as plain bar.

In the UK stainless steel is currently produced from the austenitic and ferritic-austenitic materials 1.4301, 1.4436, 1.4429, 1.4462, 1.4501, 1.4529. They are listed from left to right in order of increasing corrosion resistance and, consequently, of increasing initial cost. In most situations standard austenitic grades 1.4301 or 1.4436 will provide an acceptable solution when designing against corrosion. The higher grade austenitic and ferritic-austenitic steels should be considered when the possibility of high levels of chloride build-up in concrete over time is anticipated (e. g. marine structures, traffic structures heavy con-
taminated with de-icing salts). The mentioned materials are typically available in all three strength grades; however the duplex steel designation 1.4462 is only available in 650 grade.

The range of sizes of bars shall be from 3 mm to 50 mm. Typical mechanical properties are listed in Table 2.

**Table 6: Minimum tensile properties**

<table>
<thead>
<tr>
<th>Strength grade</th>
<th>0.2 proof strength $R_{p0.2}$ (N/mm$^2$)</th>
<th>Stress ratio $R_m/R_{p0.2}$ (N/mm$^2$)</th>
<th>Elongation at fracture $A_3$ (%)</th>
<th>Total elongation at maximum force $A_m$ (%)</th>
<th>Nominal size (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>200</td>
<td>1.10</td>
<td>22</td>
<td>5</td>
<td>3-50</td>
</tr>
<tr>
<td>500$^{1)}$</td>
<td>500</td>
<td>1.10</td>
<td>14</td>
<td>5</td>
<td>6-50</td>
</tr>
<tr>
<td>650</td>
<td>650</td>
<td>1.10</td>
<td>14</td>
<td>5</td>
<td>3-25</td>
</tr>
</tbody>
</table>

$^{1)}$Recommended grade

**Germany**

In Germany there exist an approval of the Deutsches Institut für Bautechnik in Berlin concerning stainless reinforcing steels [23]. The application of these steels has up to now been limited because of the high price.

Small diameters of 4 to 14 mm are cold rolled plain or ribbed bars and are of the ferritic type 1.4003, the austenitic type 1.4571 and the ferritic-austenitic (duplex) type 1.4462. The wires are weldable and also used for welded wire mesh. Typical mechanical properties are documented in Table 3. The strength grade corresponds to the British strength grade 500, but the elongation at fracture is 10%. It is recommended to use the grades 1.4571 and 1.4462 if the possibility of high levels of chloride are to be expected.

The steel grade 1.4003 may be used if quick carbonation of concrete cover can not be excluded reliable.

Further in Germany bars of 10 to 40 mm are offered in the hot rolled condition. With the austenitic steel grade of 1.4429 a yield stress of 550 to 880 N/mm$^2$ can be reached (Table 3).

**USA**

In the USA, stainless steel reinforcement is specified in ASTM A955M - 2001 [20], which covers deformed bar in a wide range of alloys and plain stain-
stainless steel clad carbon steel bars from 9.5 to 57.3 mm diameter. In particular austenitic stainless steels with designation numbers 1.4429 and 1.4404 are often used, an typical ferritic-austenitic (duplex) stainless steels are types equivalent to 1.4462. They are generally of one of three minimum yield levels, 300, 420 and 520 N/mm$^2$, designated as grade 300, 420 and 520, respectively.

**Other countries**

In Denmark, cold rolled weldable austenitic stainless steel smooth and profiled bars of the types 1.4301 and 1.4401 are in use [24]; dimensions from 4 - 16 mm are available. Resistance welding is the most widely used welding method. For instance, it is used for prefabrication of mesh reinforcement. MIG/MAG welding is the most frequently used method for welding carried out on site. In other Scandinavian countries also steel types 1.4301 and 1.4401 are specified. In particular in Norway and Finland the steel type 1.4436 has been used.

In Italy, mainly austenitic stainless steels 1.4301 and 1.4401 and ferritic-austenitic (duplex) steels of grade 1.4462 and 1.4362 have been used in reinforced concrete structures (Table 4).

In France, the low austenitic carbon steel types 1.4307 and 1.4404 are specified.

Many specifications in the Middle East are based on BS 6744 [22], particularly using 1.4401 steel. Duplex steel 1.4462 has been used in repair contracts in the Middle East. In parts of Far East, such as China, Japan and India, the American codes are generally used.

**European standard**

At present a common European standard [12] is in preparation. This standard specifies the requirements for the chemical composition, mass per unit length, dimensional, mechanical, technological and shape properties of bars and coils (wire rod and wire) of reinforcing stainless steel, smooth of grade InE235 and smooth, ribbed or indented of grades InE500, InE650 and InE800, with a nominal diameter between 5 mm and 50 mm. The designation of reinforcing stainless steels covered by this standard consists of the indication of the specified proof strength of the product.
The tensile mechanical properties shall be in accordance with the requirements of Table 7. The specified values are a 0.05 fractile for $R_{p0.2}$ and 0.10 fractiles for the ratio $R_m/R_{p0.2}$ and $A_{gt}$, to which minimum values are associated.

<table>
<thead>
<tr>
<th>Steel grade</th>
<th>0.2 % Proof strength $R_{p0.2}$ (Mpa)</th>
<th>Ratio $R_m/R_{p0.2}$</th>
<th>Total elongation at maximum force $A_{gt}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fractile value</td>
<td>Minimum value</td>
<td>Fractile value</td>
<td>Minimum value</td>
</tr>
<tr>
<td>InE235</td>
<td>235</td>
<td>220</td>
<td>1.15</td>
</tr>
<tr>
<td>InE500</td>
<td>500</td>
<td>475</td>
<td>1.10</td>
</tr>
<tr>
<td>InE650</td>
<td>650</td>
<td>625</td>
<td>1.10</td>
</tr>
<tr>
<td>InE800</td>
<td>800</td>
<td>775</td>
<td>1.10</td>
</tr>
</tbody>
</table>

The Table 1a - c lists the chemical composition of the main stainless steels, suitable for the manufacturing of reinforcing stainless steels. For each of these steels, the grades that are possible to get are indicated in the last column of the tables; getting these grades depends on the diameter of the product, its manufacturing process (hot or cold rolling) and its profile (smooth, indented or ribbed). The steels mentioned may be welded under certain conditions.

For the proposed steels in Table 1 (excepting the steel 1.4003)), the standard gives guidelines for the selection of stainless steels depending on the conditions of use and environment as well as example of application. However, this recommendation for use is not quite straight, too complicated and seems not to be covered by research and practical experience.

6 PRACTICAL EXPERIENCES WITH APPLICATION

Stainless steel reinforcement have been used in concrete structures in UK, USA, Italy, France, Denmark, Norway, Sweden, Finland, Germany, in the Middle and Far East and South Africa. Typical applications of stainless steel reinforcement are structures which are exposed to very aggressive environments.

Only relatively small quantities of stainless steel reinforcement have been used in the past. However, an increasing amount of austenitic or ferritic-austenitic (duplex) steel reinforcement is to be found in bridge engineering, multi-storey car park decks, tunnels and underpasses, retaining walls, marine
structures like piers at the sea coast, where influence of seawater or de-icing salt cannot be excluded, and historic buildings and buildings with long service lives [4,5,9,10,25,26]. Further these steels are generally located at construction joints or critical gaps between columns and deck.

Ferritic stainless steels are used as reinforcement in pre-cast elements of normal-weight and light-weight concrete. Another typical application is in pre-fabricated wall elements with inner heat insulation where the reinforcement connects the outer and inner concrete walls [6].

The experiences are positive; core samples taken after some years and long-term monitoring of embedded corrosion probes showed no sign of corrosion of the stainless steel reinforcement [5,9]. However, there exist no extensive long-term experiences with the use of stainless steels as reinforcement in concrete. In [27] a case of long-term application of stainless steel reinforcement (steel grade 1.4301) from Mexican Gulf is reported. Due to the harsh environmental exposure of concrete piers (hot and humid marine environment) it was decided to use stainless steel in selected areas. 60 years after construction no significant corrosion was found for the reinforcement with a cover larger than approx. 20 mm, despite the extremely high chloride contents of up to 1.9 % Cl$^-$ of dry concrete weight. For other piers at the same place reinforced with ordinary carbon steel serious chloride and/or carbonation-induced corrosion problems occurred.

1.4571 (X6CrNiMoT 17-12-2) had been stored and sprayed under conditions of parking decks and walls by the road side exposed to chloride containing water. The concrete was of medium quality; the concrete cover was 2.5 and 5.0 cm and the crack widths 0.05 to 1 mm. The cracks were carbonated artificially.

During the storage the corrosion potential of the steel was measured continuously, to detect the start of corrosion inside concrete cracks. Some beams were opened to reveal the state of the bars.

In the case of unalloyed steel, there existed an essential drop of corrosion potential, when the chloride reached the reinforcement in the concrete cracks and the steel became active after 1 to 3 months. Concerning the corrosion resistant reinforcement, the steel remained passive over the whole testing time of 2.5 years.

After breaking up some beams strong corrosion was found in the concrete cracks if the crack width exceeded 0.1 mm in the case of unalloyed steel. No
serious corrosion was detected on the high alloyed steels up to a crack of 1 mm. Stainless steel reinforcement of type 1.4462 and 1.4571 is suitable for the very unfavourable case of highly chloride contaminated cracked concrete.

7 CORROSION BEHAVIOUR

The informations collected in [3] have shown that stainless steel offers excellent resistance to corrosion in concrete structures exposed to aggressive environment.

As opposed to carbon steels which is protected by a passive film only in alkaline environments, the protective film which forms on stainless steel is stable in alkaline to neutral and slightly acid environments. Consequently, stainless steels do not suffer general corrosion and will not corrode even in carbonated concrete.

Stainless steel reinforcement has a much higher corrosion resistance against chloride attack and can withstand much higher chloride contents compared to the normal carbon steel; however also stainless steels can be subjected to localised corrosion if the chloride content in the concrete resulting from seawater or de-icing salts exceeds a certain critical value.

Such threshold values depend on the chemical composition and microstructure of the stainless steels, surface finishing and the presence of welding scale, the pH-value of the concrete solution and environmental conditions (humidity and temperature). The intensity of the pitting corrosion increases with increasing chloride content. Carbonation of the concrete will lead to a significant reduction in the critical chloride concentration for pitting initiation.

The unalloyed steel commonly leads to widespread corrosion in chloride-contaminated environments with spalling of the concrete cover while for stainless steel only locally concentrated attack may occur. It was noted that a corrosion attack on a not sufficient resistant type of stainless steel develops different than on black steel. On stainless steel the attack does not spread in the same way as on black steel, but grows more like a pinhole attack. This might lead to a quick reduction in the cross section and consequently in the load bearing capacity if corrosion occurs under extreme conditions if the stainless steel is not highly enough alloyed with respect to the environment.
Depending on the actual corrosion attack, ferritic or austenitic steel as well as ferritic-austenitic (duplex) steel can be used. The corrosion resistance increases in the sequence:

- **unalloyed**
  - ferritic e.g. Cr12 .... Cr17
  - austenitic e.g. CrNi 18-10
  - ferritic-austenitic e.g. CrNiN 23-4
  - austenitic e.g. CrNiMo 17-12-2
  - ferritic-austenitic e.g. CrNiMoN 22-5-3

These steels used as concrete reinforcement will not corrode at all provided they are selected in accordance with the expected conditions.

The corrosion properties appear to be extremely dependent on the state of the steel surface. In particular, all scale and temper colours can aggravate pitting corrosion and therefore the usual welding procedure will lead to a significant reduction in the corrosion resistance; it reduce the level of chloride contamination at which corrosion can take place. This problem can be anticipated by higher alloying the steel or removing millscale and temper colours by pickling or shot blasting. However all studies also indicated that there was no corrosion of welded molybdenum alloyed steel type 1.4571 and 1.4462 steel under practical conditions of strongly chloride-contaminated uncarbonated and carbonated concrete (chloride concentrations up to 5 M.-% and higher).

Fig. 2 summarises the results of the literature in [3] and draws the corrosion degree based on pitting depth and loss of weight. Areas without and weld are separated:

- As expected mild steel bars corrode in carbonated and/or in chloride contaminated concrete. The strongest attack occurs in carbonated plus chloride-contaminated concrete; cracking and spalling of the concrete specimen are common.

- The unwelded low-chromium ferritic steel of type 1.4003 shows a distinctly better behaviour than unalloyed steel when embedded in carbonated or in alkaline concrete containing low chloride levels. The critical chloride content for pitting corrosion is about 1.5 to 2.5 M.-% depending on state of surface, type of cement (pH-value of pore liquid) and concrete quality. However, at higher chloride contents this steel suffers pitting attack, which is concen-
trated at a few points on the surface. The tendency to concrete cracking is distinctly lower than for corroding mild steel. In chloride contaminated concrete the (unwelded) steel may suffer a stronger attack if carbonation had reached the steel surface.

For the welded steel within the weld line, chlorides in the order of \(\geq 0.5 \text{ M.-\%}\) produce locally distinct pitting corrosion. The depth of pitting increases with increasing chloride content and is more pronounced in chloride-containing carbonated concrete. However, for the ferritic chromium steel the pitting at weld lines is deeper than for unalloyed steel, but the overall general corrosion (loss of weight) is significantly smaller.

All the higher alloyed stainless steels have a very high corrosion resistance in all the environments tested. No corrosion appeared with the austenitic steel CrNiMo 17-12-2 (1.4571) and the ferritic-austenitic (duplex) steel CrNiMoN 22-5-3 (1.4462). These properties are also maintained at the highest chloride levels that appear in practice and when these steel types are welded.

The ferritic-austenitic (duplex) steels offer even better properties. These materials may provide a suitable solution to the problem of concrete structures requiring rebars with high mechanical strength and good corrosion resistance.

The corrosion properties of austenitic and ferritic-austenitic Cr-Ni-Mo-steels are better than for Cr-Ni-steels. Some results [28, 29] suggest that, within this group of stainless steels, bars without molybdenum are sufficiently resistant and therefore suitable for application in chloride contaminated concrete. Nevertheless, after results of [24], welded bars without molybdenum seems not to be sufficiently resistant and not suitable for application in presence of more than 3 M.-\% chloride in concrete (related to the amount of cement).

Concluding one can say that ferritic stainless steel with at least 12 M.-\% of chromium might be the best choice in moderately aggressive environments (carbonated concrete or exposed to low chloride levels), where the higher resistance of the more expensive austenitic stainless steels is not necessary. Austenitic stainless steel of type CrNiMo 17-12-2 and ferritic-austenitic (duplex) steel CrNiMoN 22-5-3, even in the welded state, proved to give excellent performance in chloride-containing concrete, even at the highest chloride levels that appear in practice. Austenitic stainless steel of type CrNi 18-10 may be satisfactory in many cases with 'normal' exposure to chlorides and no welding of the
reinforcement. Higher alloyed steels than the mentioned types seem not to be necessary unlike the recommendations in [4,10,12].

<table>
<thead>
<tr>
<th>Steel</th>
<th>Concrete</th>
<th>Alkaline</th>
<th>Carbonated</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cr M. - %</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>Unalloyed</td>
<td>Unwelded</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Welded</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ferritic 12 Cr</td>
<td>Unwelded</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Welded</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Austenitic 18 Cr - 10 Ni</td>
<td>Unwelded</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Welded</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Austenitic 17 Cr - 12 Ni - 2 Mo</td>
<td>Unwelded</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Welded</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ferr.-aust. 22 Cr - 5 Ni - 3 Mo</td>
<td>Unwelded</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Welded</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1) Chloride content in concrete related to cement

![Corrosion Behaviour Chart]

**Fig. 2: Corrosion behaviour of steel in concrete (survey)**

Stainless steels can be used for complete or partial substitution of carbon steel in new reinforced concrete structures exposed to aggressive environments or when a very long service life is required.

Due to the very high cost of stainless steel reinforcement it is not likely that the entire reinforcement, for example in a large marine structure, would be made of stainless steel. A possible alternative is to use stainless steel only as the outer reinforcement in the splash zone. Stainless steel and unalloyed steel will then probably be in electrical contact and this could lead to a theoretical risk of galvanic corrosion. Furthermore, in the rehabilitation of corroding reinforced concrete structures, stainless steel are often used in structures reinforced with normal carbon steel and galvanic coupling can occur.

As long as both metals are in the passive state, i.e. not corroding, their potentials will be more or less the same when embedded in concrete and galvanic coupling does not produce appreciable effects. Even if there should be minor differences in potential, both black and stainless steels can be polarised signifi-
cantly without serious risk of corrosion, i.e., their potentials will approach a common value without the passage of significant current.

In situations where the unalloyed carbon reinforcement is corroding and the stainless steel is passive, the galvanic coupling will give rise to accelerated corrosion. However, the coupling of corroding carbon steel with stainless steel is generally without risk and is negligible compared to coupling to passive carbon steel which always surrounds the corroding area [30-32]. Fig. 3 shows that the macrocouple current density (increase in corrosion) was almost one order of magnitude lower when corroding carbon steel in 3 M.-% Cl\(^-\) concrete was connected with passive stainless steel, compared to the current density measured during the tests with a passive bar of carbon steel. That means that the increase in corrosion rate of corroding carbon steel embedded in chloride-contaminated or carbonated concrete, due to galvanic coupling with stainless steel, is significantly lower than the increase brought about by coupling with passive carbon steel. Stainless steel has in the absence of welding scale (see below) a higher over-voltage for cathodic reaction of oxygen reduction (the cathodic oxygen reaction is a very slow process) with respect to carbon steel. That means, the increase in corrosion rate on carbon steel embedded in chloride-contaminated concrete due to galvanic coupling with stainless steel is significantly lower than the increase brought about with passive carbon steel. Therefore, coupling with stainless steel seems to be less dangerous than coupling with passive areas on carbon steel that always surround the area where localised corrosion takes place. Thus, assuming the ‘correct’ use of the stainless steel, i.e. stainless steel is used at all positions where chloride ingress and subsequent corrosion might occur, the two metals can be coupled without problems.

Nevertheless, a worse behaviour was observed in the presence of a welding scales (see Fig. 3). Oxide scale produced at high temperature increases the macrocouple current density generated by stainless steels, to the same order of magnitude or even higher than that produced by coupling with carbon steel.

The fact that stainless steel is a far less effective cathode in concrete than carbon steel, makes stainless steel a useful reinforcement material for application in repair projects. When part of the corroded reinforcement, e.g. close to the concrete cover, is to be replaced, it could be advantageous to use stainless steel instead of carbon steel. In being a poor cathode, the stainless steel should minimise any possible problems that may occur in neighbouring corroding and passive areas after repair.
Fig. 3: Macrocouple current density in a corroding bar of carbon steel in 3% chloride contaminated concrete when it was coupled
- with a passive bar of unalloyed steel in chloride free concrete,
- bars of 1.4571 stainless steel in chloride free concrete,
- bars of 1.4571 stainless steel in 3% chloride contaminated concrete
Results on stainless steel bars also with the surface covered with oxide scale produced by heating at 700 °C in order to simulate a welding scale [31]

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[12] Corrosion resistant (stainless) reinforcing steels. ECISS/TC 19/SC 1/WG6, preliminary European standard, draft 2.11.2005


[22] BS 6744:2001 Stainless steel bars for the reinforcement of and use in concrete – requirements and test methods


EXPERIMENTAL IDENTIFICATION OF THE GRADE OF PRESTRESSING OF A REINFORCED CONCRETE SLAB WITH UNBONDED PRESTRESSING

EXPERIMENTELLE ERMITTlung DES VORSPANNGRADES EINER STAHLBETONDECKENPLATTE MIT VORSPANNUNG OHNE VERBUND

DETERMINATION EXPERIMENTALE DU DEGRE DE PRECONTRAINTE D'UNE DALLE EN BETON ARME SOUS PRECONTRAINTE NON ADHERENTE

Thomas Jahn

SUMMARY

After removing the formwork of a monolithic fabricated and prestressed (unbonded prestressing) concrete slab with a cantilever, the measured deformations in the direction of the applied load at the end of the cantilever did not correspond to the calculated values for the required prestressing force. Therefore a non-destructive test in order to determine the existing prestressing force was necessary. The subsequent grab of the ends of the tendons was not possible, because the strands have been cut almost directly behind the wedges. This report describes a procedure, which offers the possibility to identify the existing prestressing force by measuring an applied transversal displacement of the tendon under a defined force.

ZUSAMMENFASSUNG

Nach dem Ausschalen einer monolithisch gefertigten, vorgespannten (Vorspannung ohne Verbund), auskragenden Stahlbetondeckenplatte wurden am Kragarmende Verformungen in Belastungsrichtung gemessen, die nicht den berechneten Werten für die planmäßig aufzubringenden Vorspannung entsprachen. Aus diesem Grund wurde eine möglichst zerstörungsfreie Prüfung der vorhandenen Vorspannkraft erforderlich. Ein nachträgliches Fassen der Spanngliedenden hinter der Ankerplatte mittels Spannpresse war nicht mehr möglich, da die Litzen bereits unmittelbar hinter der Keilverankerung abgetrennt wurden. In diesem Beitrag wird ein Verfahren beschrieben, das es ermöglicht, die im Spann-
glied vorhandene Spannkraft durch eine Querauslenkung, unter Messung des Auslenkweges und der dazugehörigen Kraft, zu bestimmen.

**RESUME**

Après le décoffrage d'une dalle monolithique en encorbellement en béton précontraint (précontrainte non adhérente), les déformations mesurées à l'extrémité de la dalle ne correspondaient pas aux valeurs calculées pour la précontrainte projetée. Un dispositif d'essai non destructif permettant de déterminer la force de précontrainte réelle fut nécessaire car les extrémités des câbles n'offraient plus de prise, ayant été sectionnées derrière les coins d'ancrage. Dans cet article est présenté un procédé permettant de déterminer la force de précontrainte du câble en mesurant le déplacement transversal dû à une force définie.

**KEYWORDS**: Unbonded prestressing, grade of prestressing, measurement of prestressed loads

1. **INTRODUCTION**

Prestressing of structures is normally applied in order to superpose the stresses due to external loading with intended counteracting stress states.

Besides a possible increase in capacity of slender structural elements, the advantage of prestressed concrete is that even under working loads no cracks emerge respectively only such with minor crack width. Depending on the bond between steel and concrete the following methods of prestressing may be applied.

- Unbonded prestressing: the prestressed tendons are located outside (external) or inside (internal) the concrete member without bond in the concrete cross section.
- Prestressing with immediate bond: after prestressing the tendons, they are incorporated into the concrete in that manner that during the hardening of the concrete the bond will develop.
- Prestressing with subsequent bonding: at first the prestressing force is applied to the hardened concrete structural element without bond between the prestressed tendons and concrete; the bond will develop afterwards grouting of the conduits.
The structural element described in this report is a reinforced concrete slab with unbonded prestressing in a building of an electronic data processing center of the University of Stuttgart. This slab is the roof of the building, see fig. 1.

The structural system of the slab is a single-span girder with a cantilever. At the end of the cantilever the loads are applied by tensioned columns (Fig. 1).

In order to reduce the selfweight of the roof slab it was designed as a hollow core slab with unbonded prestressing, see fig. 3.

The configuration of the prestressed tendons is linear. The prestressed tendons are anchored at the neutral axis of the slab by anchor plates and conical wedges. Above the supporting columns the distance between prestressed tendons and neutral axis of the slab is 32 cm.

The bending moment curve (fig. 2) due to prestressing results from equation (1):

\[ M_{cp} (x) = P(x) \cdot z_{cp} (x) \]  

with:

- \( P(x) \) ... value of the prestressing force at position \( x \)
- \( z_{cp} (x) \) ... distance between the prestressed tendon and the neutral axis of the reinforced concrete slab

Fig. 1: longitudinal section of the building
After prestressing, mounting the tensioned columns at the cantilever and subsequent removal of the temporary bearings of the slabs in the ground floor and in the first floor, a vertical deflection of more than 20 mm could be observed at the end of the cantilever.
The consulting engineers office stated that these deformations could not be expected according to static calculations and exceed the determined tolerances for the planned cladding structure. Based on a comparison between the measured and the calculated values it was stated that the deflection of the cantilever meets the expected values for a non-prestressed slab.

Reliable records of prestressing were not available. It was supposed that the applied prestressing force did not reach the calculated values [1].

2. PERFORMED TESTS AND RESULTS

The easiest way of determining the actual prestressing force would have been to engage a hydraulic ram at the ends of the tendons and measure force and displacement of the ram. The slope of the load-displacement-curve changes when the force of the ram exceeds the actual prestressing. This method was not possible because the ends of the tendons have been cut short behind the wedges after prestressing.

The following alternative method was chosen in order to carry out a mostly non-destructive test of the actual prestressing force:

- The upper surface of the concrete slab was stemmed at a length of 1,2 m (in the axis 204, 229 and 256 of the building) and the tendons were uncovered, see fig. 4.

- A transversal force was applied to the tendons and the corresponding displacement was measured using a tensiometer procep MS 150 (see Fig. 5). The tensiometer works on the principle shown in fig. 6, but must be calibrated on the diameter of the strand or wire and on additional boundary conditions. The displacement was chosen to reach certain displayed values on the tensiometer of 80, 90, 100 and 110 which correspond to the range of expected prestressing forces according to calibration tests. Four measurements were taken at different positions along each strand in order to minimise the influence of strand twist.

- The actual prestressing forces were determined from the comparison between measured values on site and values obtained on a reference tendon in the laboratory.
Fig. 4: stemmed surface of the slab in the area of tendons (axis 204)

Fig. 5: tensiometer proceq MS 150
Experimental identification of the grade of prestressing of a reinforced concrete slab with unbonded prestressing

Fig. 6: force-triangle as a result of transversal displacement of the tendon by the tensiometer proceq

The results of the measurements on site are summarized in table 1.

Table 1: measurement results of detecting prestressed loads by proceq and the corresponding average values of transversal displacements of tendons in the concrete slab

<table>
<thead>
<tr>
<th>displayed tensiometer value</th>
<th>average transversal displacements [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>prestressed tendon axis 204</td>
</tr>
<tr>
<td>80</td>
<td>1.60 1.56 1.82 1.56</td>
</tr>
<tr>
<td>90</td>
<td>2.14 2.06 2.35 2.23</td>
</tr>
<tr>
<td>100</td>
<td>2.72 2.70 2.82 2.72</td>
</tr>
<tr>
<td>110</td>
<td>3.22 3.12 3.40 3.26</td>
</tr>
</tbody>
</table>

The reference tendon (St 1570/1770 ø 15.7 mm, 4 m long) was first prestressed to the maximum admissible tensile load of $P_{\text{tmax}} = 186$ kN, established by static calculation. Then the tensiometer was applied and the prestressing force was reduced step by step to 170, 160, 150 and 140 kN in order to gain dis-
placement values for a reasonable range of prestressing forces. A loss of
prestressing force of 15 % due to creep and shrinkage would yield a remaining
force of

\[ P_{1\to\infty} = 186 \cdot 0,85 = 158 \text{ kN} \]

which is about in the middle of the tested range.

The transverse displacements were chosen to achieve displayed values of
80, 90, 100 and 110 like on site and again four positions were measured along
the strand to account for the strand twist.

The results of the measurements of the reference tendon are shown in table 2.

*Table 2: test results obtained on the reference tendon*

<table>
<thead>
<tr>
<th>displayed tensiometer value</th>
<th>prestressing loads applied by the load equipment [kN]</th>
<th>average transversal displacements [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>186</td>
<td>170</td>
</tr>
<tr>
<td>80</td>
<td>1,42</td>
<td>1,97</td>
</tr>
<tr>
<td>90</td>
<td>1,95</td>
<td>2,41</td>
</tr>
<tr>
<td>100</td>
<td>2,40</td>
<td>2,95</td>
</tr>
<tr>
<td>110</td>
<td>2,88</td>
<td>3,40</td>
</tr>
</tbody>
</table>

The real prestressing force of the tendons in the building were calculated
by interpolating with the deformation values measured on site between the
prestressing forces of the reference tendon and averaging a total of 16 values for
each tendon.

The results of this evaluation are given in table 3.
Table 3: identified prestressing forces

<table>
<thead>
<tr>
<th>axis</th>
<th>$P_{tl}$ [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>204</td>
<td>175</td>
</tr>
<tr>
<td>229</td>
<td>178</td>
</tr>
<tr>
<td>256</td>
<td>148</td>
</tr>
</tbody>
</table>

The measured prestressing forces of the axis 204 and 229 are approximately 6% smaller than $P_{t=0}$.

By taking into consideration the loss of prestressing forces by creep and shrinkage the values correspond to the anticipated forces with a loss of 5-7% of the initial prestressing. The determined prestressing force $P_{tl} = 148$ kN of axis 256 is approximately 20% lower than the value $P_{t=0}$.

The deviation of the prestressing force in one of the tested strands from the nominal value might be due to a bigger sag of the strand after mounting of the hydraulic ram but before prestressing or by a bigger slip of the wedge than those of the other 3 strands.

In order to gain information on possible differences between prestressing forces of the single strands of a 4-strand-tendon, 4 strands were mounted in an anchorage plate and loaded with a tensile force of:

$$P_{compl} = 4 \cdot 186 = 744 \text{ kN}$$

The transversal displacements of each of the strands at defined tensile forces were measured with the tensiometer procedure like in the tests described earlier. The results are listed in table 4.
Table 4: average transversal displacements of 4 single strands simultaneously stressed

<table>
<thead>
<tr>
<th>measured values of prestressed forces by proceq [kN]</th>
<th>average transversal displacements [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>strand 1</td>
</tr>
<tr>
<td>90</td>
<td>1.97</td>
</tr>
<tr>
<td>100</td>
<td>2.46</td>
</tr>
<tr>
<td>110</td>
<td>2.95</td>
</tr>
</tbody>
</table>

The largest deviation of single transversal displacements compared to the mean value of all strands is equal to 7%. For small deviations an almost linear relation between prestressing force and displacement can be assumed and therefore this values describes the deviation of forces as well.

Assuming that at the time of measurement the loss of prestressing force due to creep and shrinkage of the concrete slab was approximately 5-7%, \( P_{t1} \approx 0.94 \times 186 = 175 \text{ kN} \) the prestressing force of the strand in axis 256 (minimum value obtained) is about 15% less compared to the expected value.

In the most unfavorable case that this minimum value of the prestressing force (148 kN) is valid for every strand, the calculated value of the deflection at the end of the cantilever would still be significantly smaller than the measured deflection of 20 mm.

Consequently insufficient prestressing of the concrete can be precluded as cause of the appeared deformations. The bearing capacity of the concrete slab isn’t impacted by the deformation.

A possible reason of the measured cantilever deflection may be a pre-deformation caused by lowering the formwork when casting the fresh concrete. Further tests are recommended.
3. CONCLUSION

Because of unexpected deformations in a prestressed reinforced concrete slab (unbonded prestressing) it was necessary to determine the existent grade of prestressing by a non-destructive test. Therefore the strands were uncovered at three places by stemming the upper surface of the concrete slab.

By using the load measuring equipment proeq the transversal displacements of the strands were measured at certain transversal loads. In order to determine the existent prestressing forces a calibration of the results was performed on a reference tendon. Transversal displacements at defined transversal and prestressing loads were measured in those reference tests in a stationary equipment. By comparing these measurement values with those of the concrete member the existent prestressing forces in the concrete slab could be determined.

One of the strands measured on site showed a prestressing force which was roughly 15 % less compared to the other strands measured and 20 % less compared to the nominal initial prestressing. Assuming this reduced prestressing for all tendons would still yield much smaller displacement of the cantilever than the value of 20 mm obtained on site. Therefore a lower grade of prestressing of the concrete slab than required can be excluded as reason of the measured deformations provided the static calculations submitted by the consulting engineers have been correct. Other possible reasons for the deflection may e. g. be deformations of the formwork during pouring of the concrete.

4. REFERENCES

SUMMARY

A fire model ("reference fire scenario") is described on which the new European fire classification system for construction products is based upon. The adoption of this new classification system within the existing national classification system and its safety level requirements for the use of building construction products due to national regulations with or without reaction to fire testing is explained. Information on the new introduced European central fire testing method “SBI”, run at the fire testing department of MPA, is given in comparison to the existing national used Brandschacht test.

ZUSAMMENFASSUNG

Es wird das dem neuen europäischen Klassifizierungssystem des Brandverhaltens von Baustoffen zu Grunde liegende Brandmodell ("Referenz-Brandszenarien") beschrieben und die Umsetzung der europäischen Baustoffklassifizierung bei der Verwendbarkeit von Baustoffen unter Beachtung der bisherigen nationalen Schutzziele mit oder ohne Brandprüfung erläutert. Die dabei völlig neu eingeführte und in der Brandprüfstelle der MPA vorgehaltene europäische Zentralprüfmethode für das Brandverhalten “SBI” wird im Vergleich mit der seitherigen nationalen Brandschachtprüfung beschrieben.
RESUME

Nous décrivons ici d’une part le nouveau système de classement au feu européen des matériaux de construction basé sur le modèle de réaction au feu («scénario de référence de réaction au feu»), d’autre part la transcription de la classification européenne dans l’utilisation des matériaux de construction dans le cadre des objectifs de prévention nationaux – avec ou sans test au feu – actuellement en vigueur. Nous décrivons la méthode centrale européenne « SBI » de test au feu, nouvellement introduite et présentée au département de contrôle de réaction au feu de MPA, en comparaison avec la méthode de test «Brand-schacht» jusqu’à présent en vigueur au niveau national.

KEYWORDS: Approval, Bauregelliste, building product directive, classification, commission decision, CWFT, fire growth, fire scenario, fixing & mounting, flame spread, flashover, ignitability, non combustibility, SBI, smoke

1 INTRODUCTION

Protection against fire hazard and assessment of reaction to fire as well as fire resistance is a basic presumption when planning and erecting buildings and is therefore an imperative requirement from both, national and EC regulations, given e.g. in the construction products directive (CPD). Assessment of reaction to fire solely is possible on basis of a prescriptive fire model and resulting fire test methods. Reaction to fire for building products historically is given by classification within national standards such as DIN 4102. The new reaction to fire classification system for building products according to EN 13501-1, set up by the European Commission, now requires reaction to fire assessment on basis of new introduced fire test methods, especially the new “central” test method, “SBI”. These new european reaction to fire classes have to be set in relation to the existing national classification system and its legal requirements, thus rendering possible the use of the products due to national building regulations.
2 EUROPEAN FIRE CLASSIFICATION SYSTEM

2.1 Fire scenario – Fire model

European commission Construction Products Directive (CPD) 89/106 gives the basic provisions for limitation of the generation and spread of fire and smoke within the building, one of them being the limitation of the building products´ contribution to a fully developed fire.

To render possible the estimation and limitation of the building products´ contribution to a fire, the implementation of a fire classification system was recommended for the description of their reaction to fire. The classification system itself is not included in the Construction Products Directive but has been published first time within Commission Decision 94/611/EG [1] and finally was published with Commission Decision 2000/147/EG [2].

The 7 fire classes for building products laid down in the Commission Decision are specified in the standard EN 13501-1 "Fire classification of construction products and building elements" [3]. These european fire classes in principle as well cover the requirements of the as yet existing 5 national German reaction to fire classes according to DIN 4102 [4].

Classification is a means to consider the construction product´s contribution to the generation and spread of fire and smoke within the room of origin or in a given area. A simplifying assumption is made to apply the same classification to different orientations and geometries and to product types other than room surface products. Products are generally considered in relation to their end use application.

For all building products, this classification is based upon fire models ("reference scenarios"), the principal validity of which has been shown from various fire disasters or results from real large scale fire tests. The latter ones are based on the consideration of a fire, initiated in a room, which can grow and eventually reach flashover. This scenario includes three fire situations corresponding to three stages in the development of a fire (Fig. 1):

- First stage includes initiation of the fire by ignition of a building product, with a small flame, within a limited area of the product.
Second stage addresses fire growth eventually reaching final flashover situation with heat release > 1 MW to 2 MW and temperature levels of 800 - 1200°C.

This stage generally is simulated for building products by a single burning item in the corner of a room, creating a heat flux onto the adjacent surfaces within the room. For floorings, a fire is seen to grow in the room of its origin, creating a heat flux on the floorings in an adjacent room or corridor, through a door opening. Smoke development and smoke density have as well to be taken in account.

In the post-flashover phase all combustible building products present finally are contributing to the fire load (fully developed fire).

Different classes address exposure of the product to the fire at different stages of the fire development in the reference scenario.

According to EN 13501-1, the relation between fire classes for building products excluding floorings (Table 1), and for floorings ("fl", Table 2), and the above shown reference fire scenario generally is described as follows:
### Table 1: Relationship between the classes and the reference fire scenarios for building products excluding floorings acc. to EN 13501-1

<table>
<thead>
<tr>
<th>Euro-class</th>
<th>Contribution to fire / aspired safety level</th>
<th>Classification acc. to DIN 4102</th>
</tr>
</thead>
<tbody>
<tr>
<td>F</td>
<td>Products for which no reaction to fire performances are determined or which cannot be classified in one of the classes A1, A2, B, C, D, E.</td>
<td>B3</td>
</tr>
<tr>
<td>E</td>
<td>Products capable of resisting, for a short period, a small flame attack without substantial flame spread.</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>Products satisfying criteria for class E and capable of resisting, for a longer period, a small flame attack without substantial flame spread. In addition, they are also capable of undergoing thermal attack by a single burning item with sufficiently delayed and limited heat release.</td>
<td>B2</td>
</tr>
<tr>
<td>C</td>
<td>As class D but satisfying more stringent requirements. Additionally under the thermal attack by a single burning item they have limited lateral spread of flame.</td>
<td>B1</td>
</tr>
<tr>
<td>B</td>
<td>As class C but satisfying more stringent requirements.</td>
<td></td>
</tr>
<tr>
<td>A2</td>
<td>Satisfying the same criteria as class B for the SBI-test according to EN 13823. In addition, under conditions of a fully developed fire these products will not significantly contribute to the fire load and fire growth.</td>
<td>A2</td>
</tr>
<tr>
<td>A1</td>
<td>Class A1 products will not contribute in any stage of the fire including the fully developed fire. For that reason they are assumed to be capable of satisfying automatically all requirements of all lower classes.</td>
<td>A1</td>
</tr>
</tbody>
</table>

**Additional classifications for smoke production:**

- **s3:** No limitation of smoke production required
- **s2:** The total smoke production as well as the ratio of increase in smoke production are limited
- **s1:** More stringent criteria than s2 are satisfied

**Additional classifications for flaming droplets/particles:**

- **d2:** No limitation
- **d1:** No flaming droplets/particles persisting longer than a given time allowed
- **d0:** No flaming droplets/particles are allowed
Table 2: Relationship between the classes and the reference fire scenarios for floorings acc. to EN 13501-1

<table>
<thead>
<tr>
<th>Euro-class</th>
<th>contribution to fire / aspired safety level</th>
<th>classification acc. to DIN 4102</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-fl</td>
<td>Products for which no reaction to fire performances are determined or which cannot be classified in one of the classes A1-fl, A2-fl, B-fl, C-fl, D-fl, E-fl.</td>
<td>B3</td>
</tr>
<tr>
<td>E-fl</td>
<td>Products capable of resisting a small flame attack without substantial flame spread.</td>
<td>B2</td>
</tr>
<tr>
<td>D-fl</td>
<td>Products satisfying E-fl and in addition capable of resisting, for a certain period, a heat flux.</td>
<td></td>
</tr>
<tr>
<td>C-fl</td>
<td>As class D-fl but satisfying more stringent requirements.</td>
<td>B1</td>
</tr>
<tr>
<td>B-fl</td>
<td>As class C-fl, but satisfying more stringent requirements.</td>
<td></td>
</tr>
<tr>
<td>A2-fl</td>
<td>Satisfying the same requirements as class B-fl relating to heat flux. In addition, under conditions of a fully developed fire these products will not significantly contribute to the fire load and fire growth.</td>
<td>A2</td>
</tr>
<tr>
<td>A1-fl</td>
<td>Class A1 products will not contribute in any stage of the fire including the fully developed fire. For that reason they are assumed to be capable of satisfying automatically all requirements of all lower classes.</td>
<td>A1</td>
</tr>
</tbody>
</table>

Additional classifications for smoke production:
- s2: No limit
- s1: The total smoke production is limited

2.2 Testing and classification of reaction to fire of building products according to European standards (EN)

The 7 reaction to fire classes A1, A2, B, C, D, E, F for construction building products/floorings, published in the above noted commission decision, are given in standard EN 13501-1 "Fire classification of construction products and building elements – Part 1: Classification using test data from reaction to fire tests". These European fire classes principally as well cover the yet existing 5 German building product´s classes (Baustoffklassen) A1, A2, B1, B2, B3 according to DIN 4102.

For assessment of reaction to fire, as far as possible test methods had been choosen based on already existing international standards (ISO) [7].
Some of the European fire testing methods are mainly comparable to the existing German fire tests in terms of testing and sample mounting procedures, thus giving sufficient equivalence in test results. A greater number of products is therefore assumed to meet at least the existing national safety levels when classified into Euro-class E (ignitability test/“Kleinbrennerprüfung“, EN ISO 11925-2 [8]) or classes A1/A2 (non-combustibility test EN ISO 1182 [9] and heat of combustion test EN ISO 1716 [10]). The test method for floorings (EN ISO 9239-1 [11]) substantially compares to DIN 4102-14 [12, 13], too.

Reaction to fire tests for classification purposes of building construction products according to EN 13501-1, are therefore generally possible to be performed, based on European standardized fire test methods, since all of the testing standards had already been published. Nevertheless, the problem still is existing, that for some of the fire tests according to European standard test methods, to a great extent still the rules are missing on how samples are to be prepared, or fixed and mounted for testing purposes, with regard to the general requirement of consideration the end use application as given in the commission decisions and the classification standard (“fixing & mounting”). The same is true for the still wanting guidance on the validity of test results for variations in one or more of the product properties and/or intended end use applications (“extended application”). Another still unsolved problem addresses the yet completely missing European rules for testing the smouldering hazards in fires, which are fire safety classification demand in some member states, e.g. Germany. The introduction of these rules and guidances, into the corresponding technical product specifications such as harmonized product (EN) standards, European Technical Approval Guidelines (ETAG) etc. or additional test directions (recommendations, technical guidance papers etc.) are but unconditional presumptions beforehand the new euroclass system can totally be adopted.
3 NEW EUROPEAN “CENTRAL” TEST METHOD

3.1 “SBI“ according to EN 13823

As was said before, for assessment of reaction to fire, as far as possible test methods had been chosen based on already existing international standards (ISO). Both, the national and the new European classification, are but partly based on different test methods, therefore a direct comparability between both systems is not given. Proof of correct relation of the classification limits, and comparability of European to national reaction to fire classification, solely can be given by direct comparison of test results on a product by product method.

Especially for assessment of the “central” European classes (B, C, D), a totally new test method had been established: the so called SBI-Test (*Single Burning Item acc. to EN 13823 : 2002-02 : Reaction to fire tests for building products. Building products excluding floorings exposed to the thermal attack by a single burning item [14]*) . The original aim of this standard has been, comparable to the well known German Brandschacht-test according to DIN 4102-1/-15 [4, 15], to determine flame spread under the attack of a single burning item such as e.g. a burning paper basket in a room corner. Conception, test arrangement, and test performance, however, are fundamentally different to the existing national Brandschacht test method. Thus, direct comparability definitively is not given at all.

Figures 2 a through c show -somewhat simplified- main construction parts of the SBI test arrangement.
European fire classification of construction products, new test method “SBI”, and introduction of the European classification system into German building regulations

Fig. 2 a - c: SBI – test apparatus
3.2 Comparison between both “central” test methods

The comparison between the both “central” test methods, the existing national Brandschacht acc. to DIN 4102 and the new european testing method SBI acc. to EN 13823 (Table 3), shows the main parameter flame spread now is replaced by the measurement of heat release being the essential european classification criterion.

**Table 3: Brandschacht compared to SBI test**

<table>
<thead>
<tr>
<th>szenario/ fire model</th>
<th>Brandschacht - DIN 4102</th>
<th>SBI - EN 13823</th>
</tr>
</thead>
<tbody>
<tr>
<td>scenario/ fire model</td>
<td>single burning item in a room corner (Paperbasket)</td>
<td>single burning item in a room corner</td>
</tr>
<tr>
<td>requirement</td>
<td>flame spread not substantial outside primary area + heat release limited smoke - temperature (density) flaming droplets/particles (smouldering)</td>
<td>sufficient retarded and limited heat release + limited lateral flame spread smoke - rate development + release flaming droplets/particles</td>
</tr>
<tr>
<td>additional:</td>
<td>residual length smokegas-temperature smokegas-density flaming droplets/particles</td>
<td>FIGRA (heat release) SMOGRA (smoke rate) LFS (lateral flame spread) flaming droplets/particles</td>
</tr>
<tr>
<td>test period</td>
<td>10 min</td>
<td>20 min</td>
</tr>
</tbody>
</table>
3.3 SBI testing facilities at MPA-Otto-Graf-Institute fire department

These differences require re-development of most of the common used building products in terms of fire safety performance. Thus, it could be foreseen, that most of the building products on the market would be subject to new-/re-testing.

MPA-Otto-Graf-Institute fire department was able to install an SBI-test apparatus and now is in a position to provide this central european fire test method for about 80% of the european approved building products. The SBI-test apparatus requires more room, extent, test period, and sample size etc. than the Brandschachtprüfung, and putting it into operation made necessary a new fire testing laboratory and the installation of an expensive smoke cleaning system which is required by legislation.

In combination with the whole of all the other european fire test methods available at MPA-Otto-Graf-Institute fire department, all of the fire tests for bulding products and floorings necessary for both, national and european testing, are now provided.

Figure 3a shows the SBI-test apparatus run at MPA-Otto-Graf-Institute fire department and Figure 3b shows a real SBI test on slab material.

Fig. 3a: MPA-Otto-Graf-Institute fire department test laboratory with SBI-test apparatus
4 REACTION TO FIRE CLASSIFICATION WITHOUT TESTING

In a further European Commission decision 96/603/EG [16], those building products are listed which are classified into categories A1/A1\textsubscript{fl} -no contribution to a fire in any stage including the fully developed fire- without testing.

These products, classified A1 from their well known reaction to fire behaviour (“deemed to satisfy”), are listed in an annex to this decision. The list covers, amongst others: concrete, mineral fibres, foamglass, fibrecement, lime, metals (iron, steel, copper, zinc, aluminum, lead), gypsum, mortars with inorganic binders (rendering and plastering mortars and floor screeds with one or more inorganic binders, as are e.g. cement, lime, masonry mortar or gypsum), clay (bricks, slabs, chimney claddings), calcium-silicate materials, natural stone and slate materials, glass, ceramics (including glass-ceramics, and glazed or unglazed extruded products).

Nevertheless, none of the listed products may contain more than 1% per weight or volume (whatever is the lower value has to be taken into account) of homogeniously dispersed organic content. Products, which are made by laminat-
ing one or more of the listed materials, are classified into class A1 without testing, if the laminating glue does not exceed 0.1 % per weight or volume.

Products with one or more **organic** layers or containing not homogeniously dispersed **organic material**, cannot be classified based on this commission decision. These products can only be classified with tests.

Products made with coating of one of the listed materials with an **inorganic** layer (e.g. coated metal products) can but classified in class A1 without testing.

Additional lists of classified building products (“CWFT“ - "classified without further testing") are intended to be issued for classes below class A1 [17], too. This corresponds to well known national practice in many of the european member countries (e.g. Germany: DIN 4102 Teil 4 [18]).

## 5 INTRODUCTION OF THE EUROPEAN CLASSIFICATION SYSTEM INTO GERMAN BUILDING REGULATIONS

As was explained both, the national and the new european classification are partly based on different test methods, therefore a direct relationship between both systems is not possible. For some of the hitherto German fire testing methods being comparable to the european fire tests, the safety level according to DIN 4102 as required in German building regulations “Landesbauordnungen”, for greater number of products is therefore also met when the products are classified into european fire classes. Building regulations consequently already accept application of euroclasses for reaction to fire of building construction products [19]. For this purpose, Deutsches Institut für Bautechnik (DIBt) published, in the “Bauregelliste” (building regulation list), a table of related requirements giving relations of classifications according to EN 13501-1 to the still, and within a foreseeable space of time, required legal fire classification nominations (e.g. “schwerentflammbär”) to the classifications (Baustoffklassen) according DIN 4102. The generally corresponding classes (without respect to additional smoke and flaming droplets/particles classification requirements) are shown in above tables 1 and 2, respectively.

By reason of still missing rules for sample preparation, fixing and mounting and extended application, as well as assessing the smouldering hazards in fires, which are fire safety classification presumptions when performig european fire tests (see 2.2), these rules have to be fixed nationally in any case to comply with the existing fire safety level requirements in national building regulations.
The current situation of generally accepted and applied technical rules for testing and assessment of reaction to fire of building products not yet being available, results in the necessity of proofing the usability of European tested and classified products by a national approval. Many of such approvals, covering European reaction to fire classification, have already been issued by Deutsches Institut für Bautechnik, mainly for floorings and thermal insulating products.

For a number of building products, however, rules for reaction to fire testing have already been laid down by the responsible European technical (e.g. group of notified bodies GNB FSG SH02, EOTA PT 04 etc.) or product standardisation committees after publication of the harmonized technical specifications such as mandated EN-product-standards, ETA-Guidelines, CUAPs etc. This more and more completes the European reaction to fire classification system and allows the implementation into national legislation step by step. As a consequence, Deutsches Institut für Bautechnik decides on a case by case procedure on if all relevant safety requirements are definitively regulated in these technical product specifications, and issues the list of “European usable” products within the “Bauregelliste”. Such products may then be used without the need of any additional approval.

REFERENCES

European fire classification of construction products, new test method “SBI”, and introduction of the European classification system into German building regulations


[14] EN 13823: 2002-02: Reaction to fire tests for building products - Building products excluding floorings exposed to the thermal attack by a single burning item.


HEAT CURING OF SELF-COMPACTING CONCRETE (SCC)

WÄRMEBEHANDLUNG VON SELBSTVERDICHTENDEM BETON (SVB)

TRAITEMANT THERMIQUE DES BETONS AUTOPLAÇANTS (BAP)

Michael Stegmaier

SUMMARY

In precast production, heat treatment of concrete is of great significance for economic utilisation of operating resources. In this paper the effect of heat treatment on the mechanical properties of self-compacting concrete (SCC) is examined. For this purpose, different types of SCC ranging in strength classes from C20/25 to C70/85 were subjected to heat treatment of different intensity, corresponding to durable storage of three days at 20 °C, and compared to specimens stored under standard conditions to the same maturity. It was found that the (w/c)$_{eq}$-ratio of the concretes played an important role in achievable strength and that the strength is significantly influenced by the pore space.

ZUSAMMENFASSUNG

Bei der Herstellung von Fertigteilen ist für eine wirtschaftliche Auslastung der Betriebsmittel die Wärmebehandlung der Betone von großer Bedeutung. In dieser Arbeit werden die Auswirkungen einer Wärmebehandlung auf die mechanischen Eigenschaften von Selbstverdichtendem Beton (SVB) untersucht. Dazu wurden unterschiedliche SVB-Typen im Bereich der Festigkeitsklassen von C20/25 bis C70/85 mit verschieden scharfen Wärmebehandlungen auf eine Reife gebracht, die einer dauerhaften Lagerung von 3 Tagen bei 20 °C entspricht, und mit normgelagerten Proben der selben Reife verglichen. Dabei wurde festgestellt, dass der (w/z)$_{eq}$-Wert der Betone eine wichtige Rolle bei der erreichbaren Festigkeit spielt und dass die Veränderung des Porenraumes einen beträchtlichen Einfluss auf die Festigkeit hat.

RESUME

Dans la production du béton préfabriqué, le traitement thermique joue un rôle important dans l'exploitation économique des ressources. Cet article traite
de l'influence d'un traitement thermique sur les propriétés mécaniques des bétons autoplaçants (BAP). Différents types de BAP, dont les classes de résistance allaient de C20/25 à C70/85, ont été soumis à des traitements thermiques d'intensités différentes jusqu'à obtention d'une maturité correspondant à une cure de 3 jours à 20°C, et ont été comparés à des échantillons de même maturité conservés selon les conditions standard. Il s'est avéré que le rapport eau/ciment équivalent joue un rôle important pour la résistance réalisable et que la résistance est fortement affectée par le volume des pores.

**KEYWORDS:**  SCC, heat curing, maturity, compressive strength, splitting tensile strength, Young’s modulus

**OBJECTIVE**

The effect of heat treatment on the mechanical properties of conventionally vibrated concrete is well known [1, 2]. Whether this knowledge can also be applied to SCC had been called into question when the code of practise on self-compacting concrete [3] was drawn up by the German Association of Structural Concrete (DAfStb). For this reason, heat treatment was not included in the first edition of this code. To close this gap, comprehensive investigations have been carried out, extracts of which will be described in the following [4]. In the new edition of the code, heat treatment has been included.

**HEAT TREATMENT**

**Calculation of maturity**

The maturity was calculated in accordance with the de Vree (CEMIJ) method [5], in which a cement-specific factor C is introduced. Based on the Portland cement clinker content, a C-value of 1,3 was assumed [6]. The calculation of the weighted increase in maturity per hour was performed using equation 1 [5]:

\[
\Delta R_w = \frac{10 \times [C^{(0,1T_i - 1,245)} - C^{-2,245}]}{\ln C}
\]

where:
- \( \Delta R_w \) = weighted maturity per hour
- \( T_i \) = mean hardening temperature of the concrete during this hour [°C]
C = cement specific factor [-]

For determining a concrete’s total maturity, the individual increases in maturity are added up in accordance with equation 2:

\[ R_g = \sum \Delta R_g \times \Delta t_i \quad [°Ch] \]  

(2)

where:
\( \Delta R_g \) = weighted maturity per hour
\( \Delta t_i \) = time interval observed, here 1 hour [h]

Heat treatment process and storage of the specimens

For performing the heat treatment the limiting values of the maximum temperature, the temperature during preliminary storage and the rate of heating and cooling given in the DAfStb code of practice on heat treatment of concrete [79] were employed. The procedure adopted for the different curing temperatures is summarised in Table 1.

\[ \text{Table 1: Temperature management for heat treatment with different maximum temperatures} \]

<table>
<thead>
<tr>
<th>Maximum temperature ( T_{\text{max}} )</th>
<th>40 °C</th>
<th>60 °C</th>
<th>80 °C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Preliminary storage time [h]</td>
<td>3.0</td>
<td>3.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Preliminary storage temperature [°C]</td>
<td>30.0</td>
<td>30.0</td>
<td>30.0</td>
</tr>
<tr>
<td>Heating rate [K/h]</td>
<td>10.0</td>
<td>10.0</td>
<td>10.0</td>
</tr>
<tr>
<td>Maintained at ( T_{\text{max}} ) [h]</td>
<td>14.0</td>
<td>5.0</td>
<td>3.0</td>
</tr>
<tr>
<td>Cooling rate [K/h]</td>
<td>10.0</td>
<td>10.0</td>
<td>10.0</td>
</tr>
<tr>
<td>Storage at 20 °C prior to testing [h]</td>
<td>30.6</td>
<td>27.4</td>
<td>4.1</td>
</tr>
<tr>
<td>Total treatment time [d]</td>
<td>2.1</td>
<td>1.8</td>
<td>0.8</td>
</tr>
</tbody>
</table>

Heat treatment of the concretes took place by hot-air method in two programmable heating cabinets and with steel forms with good conductive properties. Prior to filling, the forms in the heating cabinets were preheated to the preliminary storage temperature. To ensure sufficient air humidity during the heat treatment and to keep the specimens from drying out on their upper open side, containers filled with water were placed in the cabinets. The evaporating water
was to ensure sufficient air humidity. The reference specimens not subjected to heat treatment were placed after manufacture in a climate chamber at a temperature of 20 °C and a relative humidity of 100 %. The heat-treated specimens were placed in the specific heating cabinets immediately after manufacture and the heat treatment has been carried out. Subsequent storage after the heat treatment until testing took place in a climate chamber at 20 °C and a relative humidity of 100 %.

Mix compositions used

The following nomenclature was adopted to ensure clear identification of the concretes: “M” denotes the powder type, “K” the combination type and “S” the viscosity-agent type. The capital letter is followed by a number, which indicates the target strength (compressive strength measured on cubes with an edge length of 150 mm) of the concrete. The second number states the maximum temperature applied during the heat treatment and/or storage temperature. The third number stands for the aimed maturity in °C days (at storage at 20 °C). Accordingly, a sample marked M85-60-3, for example, identifies the specimen as being of powder type with a projected compressive cube strength of 85 N/mm², subjected to heat treatment at a maximum temperature of 60 °C in order to achieve a maturity that corresponds to a storage of 3 days at 20 °C.

Table 2: Mix compositions of the combination types investigated

<table>
<thead>
<tr>
<th>Constituents</th>
<th>K25</th>
<th>K45</th>
<th>K65</th>
<th>K85</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement type</td>
<td>CEM II/ A-LL 32.5R</td>
<td>CEM II/ A-LL 32.5R</td>
<td>CEM II/ A-LL 42.5R</td>
<td>CEM II/ A-LL 42.5R</td>
</tr>
<tr>
<td>Cement content [kg/m³]</td>
<td>240</td>
<td>300</td>
<td>350</td>
<td>500</td>
</tr>
<tr>
<td>Water [kg/m³]</td>
<td>170</td>
<td>166</td>
<td>170</td>
<td>185</td>
</tr>
<tr>
<td>Limestone powder [kg/m³]</td>
<td>316</td>
<td>104</td>
<td>79</td>
<td>0</td>
</tr>
<tr>
<td>Fly ash [kg/m³]</td>
<td>0</td>
<td>99</td>
<td>119</td>
<td>129</td>
</tr>
<tr>
<td>Sand [kg/m³]</td>
<td>746</td>
<td>775</td>
<td>751</td>
<td>705</td>
</tr>
<tr>
<td>Gravel [kg/m³]</td>
<td>878</td>
<td>900</td>
<td>873</td>
<td>819</td>
</tr>
<tr>
<td>Powder content [kg/m³]</td>
<td>569</td>
<td>516</td>
<td>560</td>
<td>643</td>
</tr>
<tr>
<td>Plasticizer content [% by mass of cement]</td>
<td>1.25</td>
<td>1.35</td>
<td>1.35</td>
<td>1.60</td>
</tr>
<tr>
<td>Viscosity-agent content [% by mass of cement]</td>
<td>0.20</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
</tr>
<tr>
<td>(w/c) eq [-]</td>
<td>0.71</td>
<td>0.49</td>
<td>0.43</td>
<td>0.34</td>
</tr>
</tbody>
</table>
For the purpose of the investigations presented here, combination types and powder types ranging in strength classes from C20/25 to C70/85 were designed. In addition to these a viscosity–agent type of strength class C20/25 was included in the program. The exact composition of the SCCs investigated is summarised in Tables 2 and 3.

### Table 3: Mix compositions of the powder types and the viscosity-agent type investigated

<table>
<thead>
<tr>
<th>Constituents</th>
<th>M25</th>
<th>M45</th>
<th>M65</th>
<th>M85</th>
<th>S25</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement type</td>
<td>CEM II/ A-LL 32.5R</td>
<td>CEM II/ A-LL 32.5R</td>
<td>CEM II/ A-LL 42.5R</td>
<td>CEM II/ A-LL 42.5R</td>
<td>CEM II/ A-LL 32.5R</td>
</tr>
<tr>
<td>Cement content [kg/m³]</td>
<td>240</td>
<td>300</td>
<td>350</td>
<td>500</td>
<td>240</td>
</tr>
<tr>
<td>Water [kg/m³]</td>
<td>168</td>
<td>166</td>
<td>170</td>
<td>183</td>
<td>192</td>
</tr>
<tr>
<td>Limestone powder [kg/m³]</td>
<td>338</td>
<td>134</td>
<td>66</td>
<td>0</td>
<td>145</td>
</tr>
<tr>
<td>Fly ash [kg/m³]</td>
<td>0</td>
<td>99</td>
<td>119</td>
<td>137</td>
<td>0</td>
</tr>
<tr>
<td>Sand [kg/m³]</td>
<td>752</td>
<td>763</td>
<td>751</td>
<td>705</td>
<td>815</td>
</tr>
<tr>
<td>Gravel [kg/m³]</td>
<td>856</td>
<td>887</td>
<td>873</td>
<td>819</td>
<td>928</td>
</tr>
<tr>
<td>Powder content [kg/m³]</td>
<td>594</td>
<td>545</td>
<td>548</td>
<td>650</td>
<td>402</td>
</tr>
<tr>
<td>Plasticizer content</td>
<td>1.25</td>
<td>1.25</td>
<td>1.35</td>
<td>1.45</td>
<td>1.50</td>
</tr>
<tr>
<td>[% by mass of cement]</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Viscosity-agent content</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.45</td>
</tr>
<tr>
<td>[% by mass of cement]</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(w/c)_{eq} [-]</td>
<td>0.70</td>
<td>0.49</td>
<td>0.43</td>
<td>0.34</td>
<td>0.80</td>
</tr>
</tbody>
</table>

The (w/c)_{eq}-ratio given in Tables 2 and 3, was calculated with the following equation 3:

\[
(w/c)_{eq} = \frac{w}{c+0.4f}
\]

where:

- \(w\) = water content [kg/m³]
- \(c\) = cement content [kg/m³]
- \(f\) = fly ash (f allowable ≤ 0.33 z) [kg/m³]

The water content of the superplasticizer was taken into account for the mixing water. The superplasticizer used is a product based on polycarboxylate
ether (PCE) with a solids content of 35% (Woerment FM/BV 375). In addition, a stabilising admixture was added to the concretes of the viscosity-agent type and the combination type. The product used is a Woermann underwater compound (ST). This admixture is largely water insoluble and is characterised by a high swelling capacity. The raw materials basis are natural and synthetic polymers. The limestone flour used, Calcit MS 14, was obtained from the company Schön und Hippelein and has a CaCO$_3$ content larger than 99%. Another admixture used was fly ash. Here, too, fly ash from Altbach power plant was constantly used to avoid fluctuations in the composition. The water used as mixing water originated from the city of Stuttgart.

**SCOPE OF THE MEASUREMENT**

The aim of the investigations presented here was to investigate the effect of heat treatment of different temperature on the mechanical properties of SCC. In order to be able to make the assessment, reference test specimens were manufactured of the SCC in addition to the specimens subjected to heat treatment. These specimens were stored in accordance with the standard at a temperature of 20 °C and a relative humidity of 100% to the age of 3 days. With the aid of the heat treatments administered at various temperatures the concretes were brought to the same maturity as attained under durable storage conditions of 3 d at a temperature of 20 °C (1822 °Ch after the de Vree’s method). All of the concretes described here are therefore of the same maturity, independent of the temperature applied for heat treatment, and can as such be directly compared with each other.

**Fresh concrete tests**

On the concretes manufactured, the typical fresh concrete test procedures for SCC were applied to determine the flowability (slump), viscosity (V-funnel test) and inclination to blocking (slump test with J-ring). The test procedure adopted is described in detail in [3, 8]. The SCCs inclination to sedimentation was assessed on the specimens for the splitting tensile strength. When the split specimens contained coarse aggregate also in the uppermost area of the cube, no inclination to sedimentation was in evidence. Apart from the special fresh concrete procedures adopted for investigating the SCC as described above, the fresh concrete air content was in addition determined on every SCC by the pressure
equalisation method specified in DIN EN 12350-Part 7 and the temperature of the fresh concrete determined with a commercially available thermometer.

** Manufacture of the concretes **

In order to ensure an as good as possible comparability among the fresh concrete varieties, the same scheme was adopted for testing and manufacturing for all the concretes. Following fresh concrete testing, the test specimens for determining the mechanical parameters were manufactured. The forms were filled using a channel of 1.5 m length. In this way it could be ensured that the concrete would be able to adequately deaerate during the flow process. For practical reasons it was not possible to cast the test specimens in one batch for all temperature treatments. Every concrete variety had to be manufactured separately for every curing temperature. Testing for the rheological properties was performed only on the first batch of every SCC type; the air content and the fresh concrete temperature were determined in every casting process.

** Determination of the mechanical properties of SCC **

The compressive strength of the SCCs was determined as specified in DIN EN 12390-Part3 on cubes with an edge length of 150 mm and a loading rate of 0.5 N/mm²s. For determining the splitting tensile strength of the SCCs, cubes with an edge length of 150 mm were manufactured in accordance with DIN EN 12390-Part 6 and tested at a loading rate of 0.05 N/mm²s. The static Young’s modulus was measured as specified in DIN 1048 Part 5 on cylinders of 150 mm diameter and a height of 300 mm.

** TEST RESULTS AND DISCUSSION **

** Fresh concrete properties **

The results of the SCC-specific fresh concrete investigations are summarised in Table 4. The fact that the slump measured with the J-ring is partly higher than without J-ring can be explained by the inherent scattering and is quite possible for individual values and the measuring accuracy employed for this procedure.
Table 4: Results of the fresh concrete tests

<table>
<thead>
<tr>
<th></th>
<th>K25</th>
<th>K45</th>
<th>K65</th>
<th>K85</th>
<th>M25</th>
<th>M45</th>
<th>M65</th>
<th>M85</th>
<th>S25</th>
</tr>
</thead>
<tbody>
<tr>
<td>V-funnel time [s]</td>
<td>10.5</td>
<td>13.0</td>
<td>18.0</td>
<td>14.0</td>
<td>11.0</td>
<td>12.0</td>
<td>15.5</td>
<td>12.0</td>
<td>5.0</td>
</tr>
<tr>
<td>Slump [mm]</td>
<td>750</td>
<td>720</td>
<td>690</td>
<td>780</td>
<td>780</td>
<td>740</td>
<td>745</td>
<td>770</td>
<td>720</td>
</tr>
<tr>
<td>Slump with J-Ring [mm]</td>
<td>750</td>
<td>725</td>
<td>690</td>
<td>765</td>
<td>785</td>
<td>730</td>
<td>750</td>
<td>730</td>
<td>685</td>
</tr>
<tr>
<td>$t_{500}$ [s]</td>
<td>5.0</td>
<td>6.0</td>
<td>10.0</td>
<td>8.0</td>
<td>6.0</td>
<td>7.0</td>
<td>6.0</td>
<td>8.0</td>
<td>4.0</td>
</tr>
</tbody>
</table>

The air contents and the temperatures of the fresh concretes measured on the SCC manufactured on different dates are presented in Table 5 and Table 6.

Table 5: Air contents of the fresh concretes for the different curing temperatures

<table>
<thead>
<tr>
<th></th>
<th>20 °C</th>
<th>40 °C</th>
<th>60 °C</th>
<th>80 °C</th>
</tr>
</thead>
<tbody>
<tr>
<td>K25</td>
<td>2.0</td>
<td>2.2</td>
<td>3.2</td>
<td>2.8</td>
</tr>
<tr>
<td>K45</td>
<td>0.8</td>
<td>2.4</td>
<td>2.2</td>
<td>0.7</td>
</tr>
<tr>
<td>K65</td>
<td>2.1</td>
<td>3.6</td>
<td>2.2</td>
<td>1.5</td>
</tr>
<tr>
<td>K85</td>
<td>0.7</td>
<td>0.8</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>M25</td>
<td>3.9</td>
<td>4.4</td>
<td>5.6</td>
<td>2.7</td>
</tr>
<tr>
<td>M45</td>
<td>1.9</td>
<td>2.0</td>
<td>1.8</td>
<td>0.6</td>
</tr>
<tr>
<td>M65</td>
<td>2.5</td>
<td>0.5</td>
<td>1.2</td>
<td>0.5</td>
</tr>
<tr>
<td>M85</td>
<td>1.3</td>
<td>1.8</td>
<td>1.4</td>
<td>0.7</td>
</tr>
<tr>
<td>S25</td>
<td>0.7</td>
<td>0.4</td>
<td>2.4</td>
<td>0.4</td>
</tr>
</tbody>
</table>
Table 6: Fresh concrete temperatures of the concretes for the different curing temperatures

<p>| Fresh concrete temperature for curing temperature [°C] |</p>
<table>
<thead>
<tr>
<th>20 °C</th>
<th>40 °C</th>
<th>60 °C</th>
<th>80 °C</th>
</tr>
</thead>
<tbody>
<tr>
<td>K25</td>
<td>25.4</td>
<td>19.9</td>
<td>20.2</td>
</tr>
<tr>
<td>K45</td>
<td>27.9</td>
<td>20.1</td>
<td>21.2</td>
</tr>
<tr>
<td>K65</td>
<td>28.0</td>
<td>20.8</td>
<td>21.4</td>
</tr>
<tr>
<td>K85</td>
<td>26.6</td>
<td>20.8</td>
<td>22.0</td>
</tr>
<tr>
<td>M25</td>
<td>23.8</td>
<td>20.8</td>
<td>20.7</td>
</tr>
<tr>
<td>M45</td>
<td>25.0</td>
<td>19.5</td>
<td>20.1</td>
</tr>
<tr>
<td>M65</td>
<td>25.2</td>
<td>20.3</td>
<td>20.6</td>
</tr>
<tr>
<td>M85</td>
<td>24.0</td>
<td>21.8</td>
<td>19.6</td>
</tr>
<tr>
<td>S25</td>
<td>21.4</td>
<td>20.0</td>
<td>19.1</td>
</tr>
</tbody>
</table>

Mechanical parameters

The effect of the curing temperatures on the compressive strength of the combination types is presented in Fig. 1 and Fig. 2. The high-strength concretes K65 and K85 are hardly affected by the heat treatment. The low fluctuations at 40 °C and 60 °C in respect of the strength achieved under standard storage ranged within the possible scattering that is possible when measuring the compressive strength and can not be attributed to the heat treatment.

Fig. 1: Comparison of compressive strengths of the combination types at an effective age of 3 d and different maximum curing temperatures
At 80 ° the high-strength concretes K65 and K85 even reached higher compressive strengths compared to standard storage. The results obtained in mixes K25 and K45 present a completely different picture. Both SCCs exhibit up to a curing temperature of 60 °C a continuous decrease in compressive strength of approx. 22 % relative to standard storage. The loss in strength continues with K25 as the curing temperatures increase and reaches approx. 30 %. Combination type K45 shows at 80 °C again a higher compressive strength. This value, however, ranges about 13 % below the strength achieved with standard storage.

![Graph showing percentage difference in compressive strength of combination types related to standard storage at an effective age of 3 d and different maximum curing temperatures.](image)

Fig. 2: Percentage difference in the compressive strength of the combination types related to standard storage at an effective age of 3 d and different maximum curing temperatures

The powder types investigated were observed to have a behaviour similar to the combination types. At a curing temperature of 40 °C, the compressive strength is lower by approx. 12 % for all powder types compared to standard storage at 20 °C. An increase in curing temperature to 60 °C leads for the M25 to a further loss of strength, while the remaining powder types experienced no further decrease in compressive strength. At 80 °C, all powder types exhibited once again a higher compressive strength than at 60 °C. The values of the high-strength powder types even surpass the values attained under standard conditions.

Viscosity-agent type S25 exhibits a comparatively poorer behaviour than combination type K25. The compressive strength of this concrete continues to decrease with rising curing temperature and ranks at 80 °C about 32 % below the values achieved at standard storage.
With regard to the effect of the heat treatment on the compressive strength, the SCCs with a low \((w/c)_{eq}\)-ratio are in general better suited for heat treatment than SCC with a high \((w/c)_{eq}\)-ratio. In literature [9] the same observations are described also for vibrated concretes. At a comparable cement content and \((w/c)_{eq}\)-ratio, the combination types are better suited for heat treatment, with regard to the compressive strength, than the remaining SCC types.

All combination types, except for K45, show an approximately identical behaviour with regard to the influence of heat curing temperature on the splitting tensile strength (see Fig. 5 and 6). Up to a maximum temperature of 60 °C, the change in splitting tensile strength is insignificant compared to standard storage.
conditions. These deviations from standard storage lie within the possible scatterings for the determination of the splitting tensile strength. At 80 °C, a decrease in splitting tensile strength, in excess of the known scattering, is observed on all combination types. Concrete K45, in contrast, experiences a continuous decrease in splitting tensile strength with rising curing temperature. For the powder types, no such uniform behaviour is observed as for the combination types (see Fig. 7 and 8).

**Fig. 5:** Comparison of splitting tensile strengths of the combination types at an effective age of 3 d and different maximum curing temperatures.

The viscosity-agent type shows analogously to the compressive strength a continuous decrease in splitting tensile strength with rising curing temperature and achieves at 80 °C only approx. 60 % of the splitting tensile strength of the standard storage.

**Fig. 6:** Percentage difference in the splitting tensile strength of the combination types related to standard storage at an effective age of 3 d and different maximum curing temperatures.
For the splitting tensile strength, the effect of (w/c)_eq-ratio on the achievable strength compared to standard storage is not as pronounced as for the compressive strength. In general, however, the splitting tensile strength is not as markedly influenced as the compressive strength, independent of the strength class of the concretes. Only the viscosity-agent type is an exception to this.

Looking at the results of the Young’s modulus measurements, it is clearly apparent that both the powder types and the combination types show hardly any difference in the values measured on the concretes of the strength classes C55/67 and C70/85. This came as a surprise, as the compressive strengths of some of the mixes of these strength classes clearly differ from each other.

![Fig. 9: Comparison of the Young's modulus of the combination types at an effective age of 3 d and different maximum curing temperatures.](image)

The effect of the curing temperature on the Young’s modulus is for the combination types essentially the same as for the compressive strength. On the concretes with low (w/c)_eq-ratio, the curing temperature effects hardly a change. The Young’s modulus, in contrast, decreases compared to storage at 20 °C with the concretes with high (w/c)_eq-ratio with increasing curing temperature, with no difference between treatment at 60 °C and 80 °C. The difference in respect of durable storage at 20 °C is approx. 17 % for K25 and 15 % for K45.
Fig. 10: Percentage difference in the Young’s modulus of the combination types related to standard storage at an effective age of 3 d and different maximum curing temperatures

In the case of the powder types, the concretes with high (w/c)$_{eq}$-ratio were found to be equal in respect of the Young’s modulus up to a temperature of 60 °C compared to storage of 20 °C and/or even somewhat better than the concretes with a low (w/c)$_{eq}$-ratio (see Fig. 11 and 12).

Fig. 11: Comparison of the Young’s modulus of the powder types and the viscosity-agent type at an effective age of 3 d and different maximum curing temperatures

When the curing temperature is further increased to 80 °C, the reverse takes place, in that the loss in Young’s modulus in the high-strength concretes with a low (w/c)$_{eq}$-ratio is less compared to normal storage.
Heat curing of SCC

The viscosity-agent type, too, shows with regard to the Young’s modulus, a definite dependency on the curing temperature. The Young’s modulus decreases with increasing curing temperature of the heat treatment and lies at 80 °C at only approx. 70% of the value achieved under standard storage conditions.

SUMMARY AND CONCLUSIONS

The objective of these investigations was to study the effect of heat treatment on the mechanical properties of SCC within the limits set by the DAfStb code of practice. With the aid of curing at different temperatures, various SCCs were brought to a maturity corresponding to a durable storage of the concretes for 3 days at 20 °C. On these concretes, the compressive strength, the splitting tensile strength and the static Young’s modulus were determined and compared to reference concretes that had been stored for 3 days under standard conditions.

The concretes with a low (w/c)$_\text{eq}$-ratio, which are typically used in the precast industry, are hardly affected by the heat treatment as regards the compressive strength and the Young’s modulus, related to the values of standard storage conditions. This applies independent from the curing temperature. The positive effect of low (w/c)$_\text{eq}$-ratio on heat treatment is also known for vibrated concretes [9]. A high (w/c)$_\text{eq}$-ratio leads in part to marked loss of strength, which in most cases increases with increasing curing temperature. For the splitting tensile strength, heat curing temperatures up to 60 °C can be regarded as uncritical. Beyond this temperature, strength losses compared to standard storage have to be reckoned with. When comparing different SCC types of identical strength

Fig. 12: Percentage difference in the Young’s modulus of the powder types and the viscosity-agent type related to standard storage at an effective age of 3 d and different maximum curing temperatures
classes for their suitability for heat treatment, the combination type can be said to be the most suitable. The viscosity-agent type of strength class C20/25 showed the highest loss of strength for all mechanical parameters and must be regarded as being unsuitable for heat treatment. Heat treatment influences the pore space of the concretes [10, 11]. This process, however, does not increase the total porosity. The cause of the loss of strength determined in testing as compared to strength after standard storage must, instead, be attributed to a change in the distribution of pore radii towards larger pores. That this is in fact the case could be shown by mercury intrusion porosimetry and helium pyknometry [10, 11]. The partly higher strength values measured on the mixes with a low (w/c)$_{eq}$-ratio subjected to heat treatment can be explained by the lower total porosity of these concretes compared to standard storage.

**LITERATURE**


FRICION STIR WELDING – INNOVATIVE TECHNOLOGY FOR JOINING ALUMINIUM COMPONENTS

RÜHRREIBSCHWEISSEN (FRICION STIR WELDING) – EINE INNOVATIVE VERFAHRENSTECHNOLOGIE ZUM FÜGEN VON HALBZEUGEN AUS ALUMINIUM

SOUDAGE FSW – UNE TECHNOLOGIE INNOVANTE POUR LA JONCTION DE PRODUITS SEMI-FINIS EN ALUMINIUM

Martin Josef Greitmann, Peter Deimel

SUMMARY

Friction stir welding is suitable for joining semi-finished aluminium products with a wall thickness from 0.3 mm to 35 mm (one side welded, butt joint or lap joint).

In the context of investigations carried out at the MPA University of Stuttgart welded joints of EN AW-6082 and EN AW-5083 (t = 6 and 10 mm) were manufactured and examined regarding the microstructure and tensile test behaviour. High-quality welding seams could be manufactured.

In the year 2004 the MPA University of Stuttgart installed a new system of the type "ESAB FSW Legio 3ST" for friction stir welding. This welding machine is suitable for the production of welded joints of aluminium components with a seam depth up to 12 mm (copper materials up to 3 mm). This offers innovative potential also for applications in civil engineering (metal construction) in the context of research and development on customers order.

ZUSAMMENFASSUNG

Das Rührreibschweißen (Friction Stir Welding) ist geeignet zum Fügen von Halbzeugen aus Aluminiumwerkstoffen von 0,3 mm bis 35 mm Wanddicke (einsichtig ausgeführte Naht, Stumpfstoß- oder Überlappstoßverbindung).

Im Rahmen von Untersuchungen an der Materialprüfungsanstalt Universität Stuttgart wurden Schweißverbindungen an EN AW-6082 und EN AW-5083, (t = 6 und 10 mm) hergestellt und hinsichtlich des Gefügeaufbaus und der Festigkeitsverhaltens untersucht. Dabei konnten fehlerfreie Schweißnähte hoher Qualität hergestellt werden.
Das im Jahr 2004 an der Materialprüfungsanstalt Universität Stuttgart beschaffte System vom Typ „ESAB FSW Legio 3ST“ für das Rührreibschweißen ist geeignet zur Herstellung von Schweißverbindungen an Aluminiumwerkstoffen bis zu 12 mm Nahttiefe (Kupferwerkstoffe bis zu 3 mm). Dieses bietet ein entsprechend innovatives Potenzial auch für Anwendungen im Bereich des Bauingenieurwesens (Metallbau) und wird im Rahmen von Forschungsvorhaben und Entwicklungsarbeiten im Kundenauftrag für innovative Produktentwicklungen eingesetzt.

RESUME

Le soudage FSW (Friction Stir Welding) est recommandé pour la jonction de produits semi-finis en aluminium ayant une épaisseur de paroi variant de 0,3 à 35 mm (cordon de soudure d´un côté, joint droit, soudage par recouvrement).

Dans le cadre d’études au MPA de l’université de Stuttgart, des jonctions par soudure entre EN AW-6082 et EN AW-5083, (t = 6 et 10 mm) ont été réalisées et étudiées au niveau micro structurel et au niveau de leur solidité. Des cordons de soudures de haute qualité présentant une haute solidité ont ainsi été réalisés.

Le système de soudage FSW de type « ESAB FSW Legio 3ST » que s’est procuré le Materialprüfungsanstalt de l’université de Stuttgart en 2004 convient parfaitement à la réalisation de jonctions par soudure pour des matières aluminium avec une profondeur de soudure allant jusqu’à 12mm (et jusqu’à 3mm pour des matériaux en cuivre). Ce système présente également un potentiel innovant pour le génie civil (constructions métalliques) et sera employé pour des développements de produits innovateurs dans le cadre de projets de recherche et de travaux de développement pour le compte de clients.

KEYWORDS: Friction stir welding, joining, aluminium, components

INTRODUCTION - FRICTION STIR WELDING

In 1991 Wayne Thomas developed and patented a new welding method, the Friction Stir Welding (FSW) [ 1 ]. The principle of the FSW-procedure is shown in Fig. 1. This technology is a modification of the friction welding proc-
Friction stir welding – innovative technology for joining aluminium components

The frictional heat is produced by a wear resistant rotary tool, which is pressed under high axial force into the welding zone.

By the use of the described welding tool it is possible to weld butt or lap joints (seam welding) [2-4]. Similar as in the rotation friction welding process, in friction stir welding the join is also formed below the melting temperature of the assembly parts, thus in solid state welding. In the literature the process of FSW is therefore compared with warm forming technologies such as extrusion and forging.

By means of this analogy, the characteristics of this welding process can be explained, e.g. the fine-grained very homogeneous microstructure of the welded seam as well as the joint properties resulting from it. Most advantages of FSW result from the simple process as well as from the solid state welding at comparatively low temperatures [5-7].

Fig. 1: Friction stir welding - procedure principle [2]
Summarizing the following advantages arise:

- simple and robust process
- simple joint preparation
- no fused-on spatters, fume or dust
- no inert protective gas
- low power requirement
- low distortion of the welded parts
- very good mechanical behaviour of the welds
- material mix of different aluminium alloys

The application of the FSW process could be impeded mainly by the high axial process forces and thus the required support of the welding zone as well as the limited possibility for three-dimensional seam welds.

**FRICITION STIR WELDING (ALUMINIUM: EN AW-6082, EN AW-5083)**

At the MPA University of Stuttgart first investigations on the fundamentals of the friction stir welding process were carried out on aluminium materials [2, 8-10].

For a workable aluminium alloy AlMg4,5Mn (EN-AW-5083) and an age-hardenable aluminium alloy AlMgSi1 (EN-AW-6082) several 10 mm thick friction stir seam welds were realized. During the welding process both the number of revolutions of the tool and the welding speed were varied, in order to get information about the influence of these parameters on the quality of the welding seam. A compilation of the tool and processing parameters is contained in Fig 2.
Metallographical examinations including hardness measurements, Fig. 3, and mechanical-technological testing were carried out for the evaluation of the material condition of the welded joints.

Both materials showed defect-free welds, with structures like annual rings ("onion structure") in the centre. For both materials, EN AW-6082 and EN AW-5083, the particles of the phases are arranged rectilinear in the base materials and partly stretched in the direction of the lines. In the region of the welding seam these lines disappear, the particles have a rather globular shape and their size is comparable to the one of the unaffected basic material. In welded EN AW-6082 material the hardness continuum transverse to the welding seam shows a decrease in hardness. In the case of the material EN AW-5083 the hardness did not change transverse to the welding seam. For the material EN AW-5083 flat tensile specimens were taken from a welded joint of higher welding speed range. The fracture of these tensile specimens took place as shear fracture under 45° in the region of the base material. The comparison of the results of the tensile tests on flat specimens with those obtained for round tensile specimens shows a somewhat lower reduction in area of the welded joint sample.
The investigations performed showed that the material behaviour and process which lead to the formation of the FSW-welded joint are very complex and must be further examined.

In view of the industrial application, the friction stir welding offers advantages in particular for the joining of aluminium components of larger wall thickness (one-side welded: up to 40 mm, both side welded: up to 70 mm), Fig. 4, and for the production of mixed constructions.
CONCLUSION

The friction stir welding process has a large development and improvement potential in the area "lightweight construction". Further topics such as "joining of modern materials" (metal matrix composites (MMCs), material mix) and "environmentally friendly production technology" are for many areas of technology (civil engineering, machine and equipment construction, lightweight construction in aviation, vehicle construction and ship building) of increasingly current significance. To achieve these goals, friction stir welding, which was developed at the beginning of the nineties, will make an innovative contribution. In this context it can be assumed that research results in this area will within short time have a broad effect on and also a direct conversion to a lot of possible applications in the industrial manufacturing.
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WATER SATURATED SUPER-ABSORBENT POLYMERS USED IN HIGH STRENGTH CONCRETE

WASSERGESÄTTIGTE SUPERABSORBIERENDE POLYMERE ALS ZUSATZ FÜR HOCHFESTEN BETON

POLYMERES SUPER ABSORBANTS SATURES EN EAU UTILISES COMME ADJUVANTS POUR BETONS HAUTES PERFORMANCES

Sven Mönnig

SUMMARY

As part of a DFG research program the influence of water-saturated admixtures and aggregates on high strength concrete attributes was investigated. Some of the results concerning super-absorbent polymers, used as admixture, are presented in the following sections.

ZUSAMMENFASSUNG

Im Rahmen eines DFG-Forschungsprojektes wurde der Einfluss wasserge- sättigter Zusätze und Leichtzuschläge auf die Eigenschaften hochfester Betone untersucht. Einige der Ergebnisse, die mit superabsorbierenden Polymeren, verwendet als Zusatzmittel, erzielt wurden, werden im folgenden gezeigt.

RESUME

Dans le cadre d'un projet de recherche DFG, l'influence sur les propriétés des bétons hautes performances des adjuvants et granulats légers saturés en eau a été analysée. Quelques résultats concernant les polymères super absorbants utilisés comme adjuvants sont présentés.

KEYWORDS: Super-absorbent polymers, high-strength concrete
1 INTRODUCTION

Internal water storage systems can be subdivided into two categories. Firstly there are the water saturated lightweight aggregates (LWA). They can be considered to be volume stable and they participate in the concrete strength. They have been closely examined by Reinhardt and Weber [1,2,3]. Secondly there are super-absorbent polymers (SAP). They came recently under investigation and it was expected that they perform likewise as internal water sources. For both water sources the determinant material parameters were investigated. Based on the found results an approach for modelling is presented.

2 MATERIAL PROPERTIES

Polymer attributes

In 2000, Jensen [8,9] introduced a new possibility to avoid self-desiccation of high strength concretes by using super-absorbent polymers, which reduce autogenous shrinkage and early age cracking of concrete. The polymers extract water after they have been dryly added to the mortar or concrete mixture. They provide the absorbed water during hydration functioning as internal water source. SAPs were known and used much earlier mainly by the health care industry. These polymers can absorb up to 5000 times of their own weight of pure water. Picture 1 shows a polymer in dry and swollen state.

Different types of polymers are known to function as super-absorber. Most often polyacrylic acids are used since their water absorption ratio (water absorption in relation to self-weight) is higher than the one of other known polymers. The polymers are covalently cross-linked by di-functional molecules [10]. The swelling capacity of commercially available polymers is highly reduced in the presence of ions because ions can cause a collapse of the gel by over cross-
linking the polymers. This fact circumvented the use of SAPs in concrete until a new generation of stable polymers was recently developed and investigated. The used polymers were grinded to spherical particles with a medium grain size distribution of approximately 125 µm (Figure 1).

**Figure 1: Particle size distribution of a used super-absorbent polymer**

**Water absorption**

The polymers influence the mortar or concrete on two different ways, dependent on the way how they get added to the mixture. Firstly, in case of being dryly inserted, they extract water from the fresh mix and by doing this the density of the matrix increases. Thereby they can consume such high amounts of water that their diameter increases dependent on their ability to consume water producing air pores with diameters of up to 400 µm. Secondly, they provide the absorbed water during hydration and act as an internal water source. For measuring the absorption capacity in an alkaline environment the polymer industry developed different testing procedures, for example the bag test [5]. Since no published research work of the behaviour of polymers under mixing conditions was found, a simplified testing procedure was developed. The absorbed water content of the polymers was determined by comparing different slump flow measurements of reference mixtures with varying water content with the one of the polymer mixture. Thus the absorption coefficient $\chi$ gets determined by the following equation:
\[ \chi = \frac{m_{\text{water,added}} - m_{\text{water,available}}}{m_{\text{polymers}}} \]  

\( \chi \) (ml/g): water sorption per gram polymer; m: mass (g)

The mass of available, remaining after absorption by the polymers, water \( (m_{\text{water,available}}) \) is identical to the water content of the reference mixture having a similar slump flow. The names of the reference lines in figure 3 refer to the added water amount. The mass of polymers and \( m_{\text{water,added}} \) are taken from the polymer mixture.

![Figure 3: Slump flow measurements of reference mortars, without any polymers, with different water content in comparison to a polymer-1 mortar containing 297 ml water and 0.6 g polymers.](image)

From behaviour of the polymer mixture it can be concluded that the water uptake is ongoing between the measurement at 0 and 5 minutes. The slope of the curve changes after 5 minutes additional mixing time and resembles afterwards to the one of stiffening reference mortar. The final slump flow is close to the mixture containing 270 ml water. Thus the water uptake can be estimated with about 45 ml/g polymer for polymer-1. For the third type of polymer, polymer-3, the water absorption was measured with about 10 ml/g. Assumed that the polymer particles are spherical and the water uptake increases the volume steadily, the radius of a swollen polymer will increase by \( \frac{3}{\chi} \). The medium sieve passing was measured with 125 micrometers. Thus, the emerging pores will have diameters between 270 and 445 \( \mu m \). Indirectly the effect of polymers can be shown by the resulting densification of the mortar matrix. The second examined polymer was in alkaline environment unstable. It is a commercially available product.
Water desorption

The desorption behaviour of polymers is mainly governed by diffusion. For most of the drying process the water delivery rate is constant and dependent on the relative humidity, e.g. at a humidity of 40% the rate was measured to be 0.10 ml/min and for 80% the rate dropped down to 0.032 ml/min.

Mercury Intrusion porosity measurement

The mercury intrusion porosity measurements (MIP), as Figure 4 shows, point up the smaller amount of pores at a radius of 0.04 µm for the mixture with polymers. The total measured pore volume was smaller for the polymer mixture. However, the air pores left after desorption of the polymers were not measured since the maximum size measurable by mercury intrusion is 58 µm.

![Figure 4: MIP results for a reference and a polymer-1 mortar](image)

Compressive strength

The compressive strength development of mortars mixed with different w/c-ratios and different types of polymers (Figure 5) did not show any significant difference between mixtures with or without polymers. If porosity is taken into account by a Bal’sin [7] calculation the difference in the compressive strength of the mixtures with polymer-2 and -3 compared to the reference mixture disappear. Except for polymer-1 the compressive strength of a mixture after 28 days were very close to each other. The smaller value for polymer-1 can be ascribed to the influence of larger pores on the compressive strength. Recent research work showed that not all pore sizes have an equal influence on the strength. Larger pores lead to higher strength reduction [6]. This could be an ex-
planation for the performance of the polymer-1 mortar. Polymer-2 was a polymer that should have collapsed during hydration hence a small difference was measurable for all examinations. This difference can be a result of a slightly smaller slump flow caused by a stiffer mortar matrix, compared to the reference mixture, due to the water absorption that occurred until the critical ion concentration was reached and the polymers collapsed.

![Graph](image)

*Figure 5: Compressive strength development of a mixture with polymers and a reference mixture. All mixtures were produced with a w/c-ratio of 0.55.*

**Freeze-thaw resistance**

By adding polymers of a known particle size distribution and absorption ratio the pore sizes developing during the water uptake can be estimated. Based on the assumption that air pores smaller 300 µm improve the freeze-thaw resistance a CDF test with a polymer-3 concrete was performed. The medium particle size was 125 µm and the absorption ratio 10 g/g. The resulting air pores had an estimated size of approximately 270 µm. The aspired exposure class was XF4. Table 1 shows the used concrete mixtures. The average spacing between two polymer particles was approximately 11 mm.
Table 1: Used concrete mixtures for CDF

<table>
<thead>
<tr>
<th></th>
<th>Reference mixture</th>
<th>Polymer mixture</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement (CEM I 42.5)</td>
<td>333.4 kg/m³</td>
<td></td>
</tr>
<tr>
<td>Water</td>
<td>160.3 l/m³</td>
<td></td>
</tr>
<tr>
<td>Water / cement-ratio</td>
<td>0.48</td>
<td></td>
</tr>
<tr>
<td>Superplasticizer (FM/BV 375)</td>
<td>500 g/m³</td>
<td>2000 g/m³</td>
</tr>
<tr>
<td>Polymer content (polymer-3)</td>
<td>- g/m³</td>
<td>1300 g/m³</td>
</tr>
<tr>
<td>0 - 2 mm</td>
<td>518 kg/m³</td>
<td></td>
</tr>
<tr>
<td>2 - 4 mm</td>
<td>263 kg/m³</td>
<td></td>
</tr>
<tr>
<td>4 – 8 mm</td>
<td>371 kg/m³</td>
<td></td>
</tr>
<tr>
<td>8 – 16 mm</td>
<td>586 kg/m³</td>
<td></td>
</tr>
</tbody>
</table>

For the determination of the scaling resistance a CDF test was used. Therefore 28 cycles freezing and thawing were performed. Each cycle lasted 12 hours. After 2, 6, 14 and 28 cycles the test specimens were cleansed by ultrasonic and weighed. Figure 6 shows the results of the test.

![Figure 6: Results of scaling determination](chart)

The average density of the tested specimens for the reference mixture was 2.36 kg/dm³ and for the polymer mixture 2.33 kg/dm³. Table 2 shows the results of the strength tests after 29 days.
Table 2: Results of the strength tests

<table>
<thead>
<tr>
<th></th>
<th>Reference mixture</th>
<th>Polymer mixture</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Compressive strength</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>65.6</td>
<td>67.3</td>
</tr>
<tr>
<td>Standard deviation</td>
<td>0.39</td>
<td>1.42</td>
</tr>
<tr>
<td><strong>Tensile strength</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>6.4</td>
<td>5.1</td>
</tr>
<tr>
<td>Standard deviation</td>
<td>0.30</td>
<td>0.44</td>
</tr>
</tbody>
</table>

The scaling test did show a significant reduction of scaling. After 28 cycles the total amount of scaling was 445 g/m² for the reference mixture and 267 g/m² for the polymer mixture that is 40% less than the value for the reference mixture. The slightly higher compressive strength is within the range of the standard deviation thus the polymer admixture did not have a negative effect on the concrete strength presumably the strength was increased by the dense concrete matrix structure. The results of the porosity distribution measured by mercury intrusion are very similar to the ones presented in figure 4. The result for the total porosity was for the polymer mixture 57 mm³/g and 62 mm³/g for the reference mixture. Since the density of the reference mixture was the higher one the measurement might not be representative but it shows that the difference is just a very small one.

3 MODEL

The influence of super-absorbent polymers was modelled based on DuCOM [4]. DuCOM calculates for a representative distribution of cement grains and a maximum free growth radius for each grain which is dependent on the total amount of free water. To consider the presence of dryly added polymers the equations describing the initial situation were changed. Equally distributed cement grains have a packing density of 0.74 under the assumption of face-centred cube packing. If water is present the distance between two cement grains is equal to s. This is the free growing radius for each hydrating cement grain too. The influence of polymers can be considered by a reduction of the free water content and a resulting smaller distance between the cement grains.
s = 2 \cdot r_{\text{Cement grain}} \left( 3 \cdot \sqrt{0.74 \cdot \frac{V_{\text{Cube}}}{V_{\text{Cement}}}} - 1 \right) \quad (2)

r: cement grain size; s: distance between two cement grains, V_{\text{Cube}}: volume of reference element; V_{\text{Cement}}: volume of cement

The cement size distribution is simplified and a smeared approach calculates the medium size of a grain based on the Blaine value. The volume of a reference element (V_{\text{Cube}}) consists of cement and water. The volume of water changes as equation 3 shows. The initial water being present is dependent on the water/cement-ratio, the polymer/cement-ratio and the mass of cement used.

\[
V_{\text{Water}} = \left( \frac{m_w}{m_w} \cdot \frac{1}{\rho_w} \right) - \left( \chi \cdot \frac{m_p}{m_p} \cdot \frac{1}{\rho_w} \right)
\]

\[
V_{\text{Water}} = \frac{m_c}{\rho_w} \left( \frac{m_w}{m_c} - \chi \cdot \frac{m_p}{m_c} \right)
\]

\(\rho_w: \) water density; \(m_i: \) mass (g); Index i: c: cement; w: water; p: polymer; \(\chi\) (ml/g): water sorption per gram polymers

4 CONCLUSION

The use of super-absorbent polymers did improve the freeze thaw resistance significantly. The compressive strength tests did not show a detrimental influence. On the contrary, the polymer mixture strength was slightly higher than the strength of the reference mixture. The mercury intrusion examinations did show a decrease of small pores while the total pore volume did not change. This is the result of a compacted concrete matrix by the water absorption of the polymers. The water absorption takes place during the mixing procedure, which can result in a mixture with significantly reduced slump flow. For high strength concrete with small water/cement ratios the amount of needed superplasticizer is increased. Examinations to investigate the influence on the shrinkage behaviour are currently performed. A preliminary model of the polymer influence on the concrete matrix structure was introduced. Examinations of a polymer mixture compared with reference mixtures with different water/cement ratios should prove the correctness of the model. The performed mortar tests did show that the absorption ratio and the particle size distribution of polymers have a significant
influence. A high absorption ratio may cause a stiff mixture and inserts a high amount of large pores resulting in a lower compressive strength.

The future research will investigate the shrinkage behaviour of polymer mixtures and further improve the model to account for changing material attributes due to desorption of water filled polymers.

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Layer thickness determination of polymer concrete

Summary
The intention of the work described in this article is to determine the thickness of a layer of 4 cm polymer concrete which covers a concrete cube of 15 cm edgelength. For this purpose electromagnetical (Ground Penetrating Radar) and acoustic echo techniques were used. The results show that only the Impulse-Echo technique was able to identify the thickness of the polymer concrete layer. If the target thickness of the layer is known, the Impulse-Echo technique can be adjusted to that thickness. This is also shown here.

Zusammenfassung
Ziel der in diesem Artikel beschriebenen Untersuchungen war die Schichtdickenbestimmung einer 4 cm dicken Lage aus Polymer Beton, die auf einen Betonwürfel (15 cm Kantenlänge) aufgebracht wurde. Hierfür kam Bodenradar als elektromagnetisches Verfahren sowie akustische Echo Verfahren zum Einsatz. Die Ergebnisse zeigten, dass nur mit dem Impuls-Echo Verfahren die Dicke der Schicht aus Polymer Beton bestimmt werden konnte. Bei vorheriger Kenntnis der Soll-Dicke einer solchen Lage, kann das Impuls Echo Verfahren auf diese Dicke „geeicht” werden. Dieser Ansatz wird ebenfalls vorgestellt.

Resume
L'objectif des investigations qui ont décrivent dans ce article était la détermination de la grosseur d'une couche de béton polymère. Cette couche de béton polymère a une grosseur de 4 cm et est situé sur un cube de béton avec un longueur des arêtes de 15 cm. Les méthodes utiliser était le Ground Penetrating Ra-
1. INTRODUCTION

Layer thickness determination in solid bodies is a challenge in different scientific areas. Example applications range from the macro scale e.g. in exploration geophysics to the micro-scale e.g. in construction materials. However, the physical principle of layer thickness determination is similar at all scales. Using electromagnetic or elastic waves, it is possible to determine the thickness of a layer by the reflected waves from the boundary between two layers.

Here, the investigated test specimen consists of a concrete cube (15 cm edge length) with a 4 cm thick polymer concrete layer on top. A detailed description of the specimen will be given in the next section. Different methods were applied for layer thickness determination. These techniques will be discussed in the next but one paragraph.

2. TEST SPECIMEN

The test specimen is a concrete cube of 15 cm edge length covered by a layer of polymer concrete which has a thickness of 4 cm. A photo and a schematic diagram are shown in Fig. 1. The sound velocities of the two materials were estimated using the traveltime of ultrasonic waves through each layer. The concrete has a velocity of approximately 4000 m/s and the polymer concrete a velocity of 3500 m/s.
The densities of the two materials do not differ significantly since only the matrix material is different. The determined velocities are also relatively close to each other. Therefore, the impedance contrast between the layers is expected to be small.

The task regarding this test specimen is if it is possible to determine the thickness of the polymer concrete layer despite the low impedance contrast of the two materials. Several methods which will be described in the following were tested.

3. APPLIED METHODS

The velocity measurements showed that the impedance contrast of the two materials of the specimen (concrete and polymer concrete) is not very high. This is not surprising, since only the matrix material of the two materials is different. However, the low impedance contrast is a problem for the non-destructive evaluation of the thickness of the polymer concrete layer. Several methods were applied because it was not clear in advance which one might be successful.
3.1 Ground Penetrating Radar

Finck [1] and Beutel et al. [2] showed that this electromagnetic method is able to detect structures within concrete and it is also often used for geophysical near surface investigation e.g. for groundwater aquifer detection. However, the georadar principle only works if the relative dielectricities of the investigated materials are significantly different from each other. The results showed that this is obviously not the case and therefore, the layer boundary could not be detected.

3.2 Impact-Echo

The principle of the Impact-Echo technique is to generate elastic waves by a mechanical impact. This generates ultrasonic waves and audible sound waves. The generated P- and S-Waves travel through the medium and are reflected back and forth between the layers or inhomogenities in the material. The broad frequency spectrum generated by the impact generally activates resonance frequencies of the structure or specimen. Recording the elastic waves e.g. with accelerometers and transforming the digitized signal in the frequency domain, the resonance frequencies of the structure or specimen can be identified. With the velocity of the material \( v \) and the determined resonance frequency \( f \) the thickness \( d \) of the specimen can be calculated by:

\[
d = \frac{v}{2f}
\]  

A detailed description of the Impact-Echo method and technical details about the usable impacts can be found in [3] and [4].

Concerning the polymer concrete – concrete specimen described in the last section the resonance frequencies corresponding to the layer thickness are expected. Several measurements using different impactors and sensors were made.

3.3 Impulse-Echo

The principle of the Impulse-Echo technique is identical to Impact-Echo. However, the source is different. As the name already indicates the source is an ultrasonic impulse. Therefore, an ultrasonic transmitter and a receiver are placed on the same side of the test sample (on top or at bottom). The layer boundaries or inhomogenities reflect the ultrasonic signal and an additional signal is generated.
In our application several measurements from the top and the bottom of the test specimen were made.

4. RESULTS

The time series of two Impulse-Echo measurements are shown in Fig. 2 and Fig. 3. In the first case (Fig. 2) the sensors were placed on top of the specimen (7 cm distance between the sensors) i.e., on the layer of polymer concrete. The second case (Fig. 3) shows the results with sensors placed on the underside of the specimen. The arrival times of the reflected signal from the layer boundary can easily be calculated from the layer thickness and the measured velocities. These calculated reflection times are marked as vertical lines in the time series.

**Fig. 2:** Time series of a measurement done with transmitter and receiver placed on the top of the test sample.

**Fig. 3:** Time series of a measurement done with transmitter and receiver on the underside of the test sample.
Neither in the measurements from the top (Fig. 2) nor from the bottom (Fig. 3), the echo from the layer boundary between concrete and polymer concrete is clearly identifiable. However, the reflection from the rear plane of the specimen is clearly detectable.

Analyzing the resonance frequencies of the specimen as described for the Impact-Echo method the recorded time series from the Impulse-Echo measurements were transformed to the frequency domain. Fig. 4 and Fig. 5 show the mean of all amplitude- and power-spectra calculated from the measurements. It is distinguished between the measurements from top (Fig. 4) and from the lower side (Fig. 5) of the specimen.

![Measurement from the top](image)

**Fig. 4**: Mean amplitude and power spectrum of the measurements done from the top of the test specimen. The resonance frequencies of the whole specimen and the polymer concrete layer are marked by vertical lines.

All calculated spectra show the echo from the back side of the specimen at approx. 10 kHz. Using Equ. 1 the thickness of the whole specimen can be calculated. The measured thickness is 19.2 cm and the calculated 19.5 cm.
The theoretical resonance frequency of the polymer concrete layer is about 43 kHz. Looking at the spectra measured from top of the specimen (Fig. 4) a weak resonance can be found at 43 kHz in the amplitude and power spectra. This resonance is not identifiable in the spectra calculated from the measurements from the lower side of the specimen.

The geometry of the specimen is definitely not ideal for detecting a thin layer with the Impact-Echo method which is created for analysing large specimen. The layered specimen used here, is relative small. Therefore, many reflections from the edges of the block disturb the analysis. Another problem is, that for the detection of the layer boundary in 4 cm depth an impact is required which generates high frequency signals. Three different impacts and two different sensors were used for the measurements. The spectra of the recorded signals
are shown in Fig. 6. The lowermost graph presents a spectrum from the Impulse-Echo measurements for comparison.

It has to be stated at first, that there is not much energy transferred beyond 28 kHz in all shown impact echo measurements (Fig. 6). Therefore, the expected resonance of the layer of polymer concrete at about 43 kHz is not detectable.

Fig. 6: Amplitude Spectra of the different impact echo measurements, using different impacts and sensors and for comparison one impulse echo measurement.

However, the consistent peak at about 10 kHz is the resonance from the back side of the specimen. The spectra of the Impact-Echo measurements show further resonance peaks. One is at about 13.5 kHz which corresponds to a thickness of 15 cm. This resonance frequency is also observable in the results of the Impulse-Echo measurements. The concrete cube of this compound specimen has a thickness of 15 cm.
Due to the question if the measured 13.5 kHz is a resonance frequency of the concrete cube, further investigations using a LMS PIMENTO MODAL ANALYSIS soft- and hardware were performed. These measurements showed that the 13.5 kHz correspond to the concrete cube. Furthermore, this resonance frequency was clearly identifiable and of equal strength on all components of the triaxial modal analysis sensor. This fact leads to the interpretation of these results that the 13.5 kHz resonance frequency belongs solely to the concrete cube and reflects the isotropic dilatation of the concrete layer. This behaviour is strongly related to the high symmetrical geometry and a spacious material isotropy.

DISCUSSION AND CONCLUSION

The relative dielectricities and the acoustic impedance between concrete and polymer concrete are very small. Most of the electromagnetical and acoustical energy is transmitted and only a small part is reflected. Therefore, the determination of the thickness of the relative small polymer concrete layer was difficult. The Ground Penetrating Radar measurements lead to no results.

The Impulse-Echo measurements lead to better results than the GPR application. However, it was not easy to determine the thickness of the polymer concrete. The polymer concrete layer is thin compared to the concrete cube. The reflection from the boundary of the two layers is hidden in the direct arrival of the elastic wave (measuring from top of the specimen) or in the reflections from the edges of the block (measuring from the underside). Therefore, it is impossible to identify the layer thickness in the time domain.

In the frequency domain the resonance frequencies are used for calculating the thickness of a layer or the whole specimen. The resonance frequency of the whole block can be clearly identified at approx. 10 kHz. This corresponds to a thickness of 19.5 cm. Considering the coarse values for the velocities the assumed thickness of 19.2 cm is well hit.

A layer of 4 cm thickness would create a resonance at approx. 43 kHz. In the measurements from top of the specimen a small peak at 43 kHz is observable. This peak can not be seen in the spectra from the measurements from the lower side of the specimen. The reflected energy is too small and the signals vanishe within the noise due to scattering and attenuation on the way back through the concrete. The weak resonance peak at 43 kHz gained from the measurements from top of the specimen can be amplified. Herefore, the signal
can be filtered with a narrowband filter. However, it might not always be possible to get a stable filter. If the expected thickness of the investigated layer is known, the frequency coefficients outside the narrow band of interest can be set to zero. Then the measurements along several lines on a plate can be compared and areas with no resonance peak easily identified.

With the Impact-Echo measurements it was not possible to find the layer of 4 cm. Only the resonance frequency of the whole specimen could be clearly identified.

With the Impact-Echo measurements it was not possible to find the layer of 4 cm. Only the resonance frequency of the whole specimen could be identified. The detected 13.5 kHz resonance frequency of the concrete cube measured as well as with Impact-Echo and the modal analysis is a pure geometric effect. This resonance occurs only due to the cubic geometry (highly symmetric) and the isotropic material behaviour of the concrete. This effect cannot be used in general for any thickness determination.

If the layer thickness is unknown, it is not easy to find it with the methods described here. But if we know a reference thickness it should be possible to develop a technique to find out if the layer has the expected thickness.

ACKNOWLEDGEMENTS

The authors thank Gerhard Bahr and Markus Schmidt for their technical support during the measurements.

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COMPARATIVE EVALUATION OF CEMENTITIOUS MATERIALS ON EARLY AGE WITH ULTRASONIC WAVE TRANSMISSION, WAVE REFLECTION AND IMPACT-ECHO MEASUREMENTS

VERGLEICHENDE UNTERSUCHUNG DES ERSTARRUNGS- UND ERHÄRTUNGSVERLAUFS VON ZEMENTGEBUNDENEN MATERIA-LIEN MIT DER ULTRASCHALL TRANSMISSIONS METHODE, DER ULTRASCHALLREFLEXIONSTECHNIK UND DEM IMPACT-ECHO-VERFAHREN

INVESTIGATION COMPARATIVE DE MATERIAUX DU CEMENT AVEC LES METHODES ULTRASONIC A TRANSMISSION, ULTRASONIC A REFLEXION ET IMPACT-ECHO

Ralf Beutel, Turgay Öztürk, Christian U. Grosse

SUMMARY
This paper summarizes results of investigations dealing with the setting and hardening of mortar and concrete. Data of three different ultrasonic methods are compared and set into relation to the temperature measured inside the tested elements.

ZUSAMMENFASSUNG
Der Artikel beschreibt die Ergebnisse einer Untersuchung über das Erstar-rungsverhalten von Mörtel und Beton mit verschiedenen Ultraschallverfahren und stellt einen Vergleich zu dem im Bauteil gemessenen Temperaturverlauf bei der Erhärtung her.

RESUME
Cet article présente les résultats d’une étude effectuée sur la processus de durcissement des matériaux cimentaires avec les methodes ultrasonic á transmission, ultrasonic á reflexion et impact-echo. De plus il compare les résultats avec les températures mesurée par l’ élément testé.

KEYWORDS: Non-destructive testing, early age concrete, ultrasonic transmission, WRF, impact-echo.
1 INTRODUCTION

Numerous applications for non-destructive testing methods to investigate concrete structures are reported; they usually focus on the detection of flaws and the determination of concrete thickness. These methods are applied to constructions which are already in service. If there is a demand for quality control at early ages of structures, e.g. for the quality of the used concrete mixture, different or modified methods have to be applied.

During the last years, several testing methods were developed to control the manufacturing process of cement-based materials during setting and hardening. At the Institute of Construction Materials, University of Stuttgart, a through-transmission technique based on ultrasound was developed [2, 4]. This technique correlates the travel time, the attenuation and the frequency content of ultrasound waves sent through the material with the elastic properties of concrete or mortar. These parameters are continuously monitored during the setting and hardening of the cementitious material giving a comprehensive picture instead of snapshots of material characteristics at specific times. The through-transmission technique requires access to both sides of the material to enable a wave travelling through the material. To avoid this disadvantage the impact-echo method was modified, to monitor the hydration process with only one-sided access. This procedure was first suggested by Pessiki et al. [7, 8].

The third method, used for this investigation, is the wave reflection method WRF [5, 6]. In order to increase the sensitivity of the ultrasonic measurement, Öztürk [6] improved the wave reflection test setup that he had developed together with J. S. Popovics at the ACBM-Center, Northwestern University, Evanston [5]. The WRF evaluates the reflected part of the wave, which was incident upon the boundary between a known material, e.g. acrylic glass, and the material to be investigated, e.g. concrete. At concrete the hydration process increases its acoustic impedance leading to a change of the WRF value with time.

This paper reports on results of tests accomplished within the research project FOR 384 [11]. Ultrasonic wave transmission, impact-echo and WRF methods were used to determine setting and hardening process of mortar and concrete. In addition a shear wave transducer was mounted at the surface of the specimen, to measure the shear wave parts of the impact generated waves along the surface. Supplemental to these experiments the in-situ temperature was measured inside the specimen. Depending on the chosen method, the setting and hardening process of tested materials is related in different ways.
2 EXPERIMENTAL METHODS

The wave transmission method used for the experiments allows monitoring the setting behaviour of fresh mortar and concrete. For this method a sample of the mixture under test is poured into a container with attached ultrasonic transducers. The elastic wave travelled through the material is recorded. Using a time of flight measurement technique the P-wave velocity can be calculated. In addition the relative energy of the signal and the frequency spectrum can be determined. Further details about this test setup and its application are given in [2, 4, 9].

The impact-echo method uses transient stress waves generated by an elastic impact (fig. 1) on the surface of concrete structures. As the stress waves propagate through the material, they are reflected by internal interfaces (e. g. voids and tendon ducts) and external boundaries of the structure. Multiple reflections between the impact surface, internal interfaces and the opposite surface cause transient resonances, which can be identified in the spectrum of the recorded signals. The emitted sound waves are obtained by a displacement or acceleration transducer which is placed near the impact point on the surface of the structure.

![Impact-echo principle](image)

The depth of interfaces can be determined by analysing the frequency spectrum of signals using following equation

\[ d = \frac{v_p}{2 \cdot f_R} \text{[m]} \]  

(1)
where \( d \) is the depth of the interface (thickness of the structure), \( v \) is the measured compressional wave velocity and \( f_R \) is the corresponding resonance frequency in the spectrum. To apply this technique to fresh concrete several assumptions and changes are necessary. As a first approach it can be considered that the tested material is changing only its rheologic properties and not its geometry. If the thickness is constant, Eq. 1 can be transformed to

\[
f_R(t) = \frac{v_p(t)}{2 \cdot d} \text{[kHz]}
\]

where \( v_p(t) \) is the compressional wave velocity subjected to changes (usually increasing) during the hardening of tested materials. Therefore, the changing resonance frequency \( f_r(t) \) gives an indirect information about the elastic properties of the hardening material.

The test setup for the WRF measurements is shown in Figure 2. Normal-incidence P-wave transducer type DEUTSCH S40HB0.1-0.3 with a centre frequency of 200 kHz was used. The transducer was coupled on the acrylic glass plate with a thin layer of VASELINE®. The WRF sensor was integrated into the formwork.

At an interface between two materials with differing acoustic impedances, a portion of the incident wave energy is transmitted through the boundary into the second material and the remaining is reflected back into the first one. When the propagating wave is normally-incident upon the boundary, the ratio of the amplitude of the reflected wave to the incident amplitude is given by

\[
R(t) = \frac{Z_2(t) - Z_1}{Z_2(t) + Z_1}; \quad Z_i = \rho_i \cdot v_{p,i}(t)
\]

Fig. 2: WRF principle
where $R$ is the reflection factor, $Z_1$ the acoustic impedance of first material, $Z_2$ the acoustic impedance of second material, $\rho$ the density of the material and $v_P$ the ultrasonic velocity of the primary wave in the material. As the first material acrylic glass was chosen and the second material was the cement-based material.

A time domain signal analysis was applied. The first received backwall echo was windowed and the absolute value of its amplitude was taken eliminating the phase of the signal. In order to eliminate the influence of the measuring device and the coupling condition of the transducer, the results obtained from the measurements at cement pastes were normalised on that at air and the wave reflection factor was obtained as

$$WRF(t) = \frac{\left| \frac{R_{\text{Concrete}}}{R_{\text{Air}}} \right| (t)}{R_{\text{Concrete}}}$$

where $R_{\text{Concrete}}$ is the reflection factor of the acrylic glass – concrete interface and $R_{\text{Air}}$ the reflection factor of the acrylic glass – air interface. For this purpose a wave reflection measurement was performed on the empty mould before filling the concrete in. Successively the measurements at the concrete were started and data collection and analysis were performed automatically. The interval for the data collection was set to 10 minutes.

The shear waves were recorded by an S-wave transducer consisting of a sensor array. After filling the mixture into formwork, the transducer was fixed on a small plexiglass plate which was mounted at the impact surface with a distance of about 30 cm to the impactor. So we were able to measure the shear wave part of the impact generated waves along the surface.

### 3 TEST PROGRAMME

The mix proportions of the mortar and concrete tested are given in Table 1. In total one mortar and two concrete mixtures were tested. After mixing the cement or concrete mixture, respectively, and pouring it into formwork the measurements were started (fig. 3, 4). The formwork had a dimension of 80 x 80 cm² and the slabs had a thickness of 15 cm.
To determine the temperature variations a thermocouple element was set in the tested specimens. The investigations were accomplished over a period of about 24 hours. Ultrasound signals were recorded every ten minutes.

**4 RESULTS**

An important indicator of the cement hydration is the development of the temperature, which is a result of the exothermic reaction between water and cement. Figure 5 shows the results of the in-situ temperature measurements in comparison to the P-wave velocity obtained with the transmission method.
The diagrams in figure 5 show that an increase of the P-Wave curve can be observed directly after the beginning of the measurement. In comparison the temperature rises not until the first two hours. The temperature starts to increase at approximately the same time when the P-wave velocity has the highest rate of change (inflection point). Furthermore, the temperature curve of the tested concrete mixtures reaches its maximum at a concrete age, when the P-wave velocity has reached a value of about 90 percent of the final value of the entire curve. Afterwards the P-wave velocity increases much slower and reaches the final value at the end of the measurement after 24 hours. It should be noted that for determining the inflection point on the experimental curves a curve fitting was applied.
Fig. 6: Comparison of Wave-Reflection-Factor (WRF) and in-situ temperature rise for mortar (RS01) and concretes (RS02, RS03); Times for initial set marked as black rectangles and final set marked as triangles on the WRF curves.

Figure 6 shows the development of the WRF and that of the temperature inside the specimen with time for mortar (RS01) and concrete (RS02, RS03). After a specific time elapse the WRF decreases rapidly, has an inflection point, reaches a minimum and increases again. Immediately after mixing, the acoustic impedance of the cement-based material is lower than that of acrylic glass, but increases in the course of the hydration process of the cement. At the minimum of the WRF curve, the acoustic impedance of the cement-based material is equal to that of the acrylic glass. Afterwards the acoustic impedance of the cement-based material exceeds that of the acrylic glass.

At the investigation performed on mortar as well as on concrete the inflection point occurs later than the rise of the temperature curve. This is reasonable for the beginning of the exothermal reaction is accompanied by the initial set
just in case of cement paste. At mortar and concrete the occurrence of the initial set takes place after a specific time elapse.

In Öztürk [6] it was shown that the inflection point does not primarily correlate to the beginning of the exothermal reaction, but to the specific mechanical state of the specimen. The inflection point was found to match with the initial set of cement paste, mortar and concrete. This point was considered as the percolation threshold, where the system changes from a suspension of cement particles in water into an interconnected solid phase. The final set time was found to coincide with the increase of the WRF curve after having reached its minimum.

The S-wave velocity of the two concrete mixtures is plotted in figure 7. In comparison to the P-wave velocity, where the curve increase immediately, a significant increase of the S-Wave curve begins later at about 2 hours after mixing. This is due to the fact that for the propagation of S-waves certain shear strength is necessary. The hydration of the cement has to be in progression so that the cement aggregates are rigidly connected. The S-wave velocity increases when the cement matrix changes from liquid to solid medium. It should be noted that at the beginning the cement mixture is not a pure liquid medium e.g. like water. In fact it is a kind of viscous matrix which is able to propagate longitudinal and transversal components of elastic waves. Thus the measured values of S-wave velocity are not zero. Comparing S-Wave velocity and temperatures an increase at the same time of both curves can be determined.

![Graph of S-wave velocity and in-situ temperature rise for concretes (RS02, RS03)](image)

*Fig. 7: Comparison of S-wave velocity and in-situ temperature rise for concretes (RS02, RS03)*
The maximum amplitudes of the frequency spectra from impact-echo measurements on mortar RS01 and concrete RS03 during setting and hardening are shown in figure 8. Like the developing of the P-wave curve, the curve of the measured resonance frequency starts to increase at the beginning. After about 6 hours there is only a little increase of the curve until a value of the thickness resonance of about 13 kHz can be obtained. Regarding the mixture RS01 the final value is lower than expected due to a loss of sensor coupling.

5 CONCLUSION

The presented investigation shows that the described methods – ultrasonic transmission, wave reflection and impact-echo – have the ability to monitor the setting and hardening of cement based materials. In case of the WRF the setting times can be detected with high accuracy. The data measured on mortar and concrete are related to the in-situ temperature. Regarding the S-wave velocity, it can be seen that an increase at the same time as the temperatures begins. This indicates that both parameters are governed by the same mechanism, that is the development of exothermic reaction between water and cement resulting in the solidification of the cement paste matrix. Furthermore during this phase, the P-wave velocity and the WRF have reached their inflection point. The accomplished investigations indicate that the parameters of these methods are directly influenced by the cement hydration process.
6 ACKNOLEDGEMENT

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7 REFERENCES


SUMMARY

The paper addresses the statistical analysis of the shear strength of structural glued laminated timber (glulam) based on full-size flexure tests. In the study conducted, a total of thirty combined glulam beams (outer three laminations: strength class C35, respectively, six core laminations: strength class C24) were tested. A subset of eighteen specimens failed in the targeted shear mode whereas the remainder failed in flexural mode. In the statistical framework of incomplete data, the random sample obtained was identified as one including Type I right-censored observations. Although such data sets occur under a variety of circumstances in material testing, adequate evaluation procedures are rarely applied. Hence, a major part of the paper is devoted to the appropriate parameter estimation of such random samples. Since statistical distributions of the location-scale family lead to convenient closed-form equations, this case is emphasized. A two-parameter Weibull distribution being of log-location-scale form was found to fit the data adequately.

Statistical inference was based on likelihood ratio procedures. The accuracy of the point estimate for the Weibull distribution was characterized by means of its joint confidence region as well as by means of its marginal confidence intervals. The characteristic shear value of the glulam tested was derived as a lower one-sided confidence interval for the 5%-quantile of the Weibull dis-
The characteristic shear value obtained was found to be slightly above the value given in the new German timber design code DIN 1052.

ZUSAMMENFASSUNG


RESUME

Cet article traite de l’analyse statistique de résultats d’essais de cisaillement en vraie grandeur effectués sur des poutres en bois lamellé-collé. Dans l’étude considérée, un échantillon de 30 poutres lamellé-collé panaché (3 lamelles extérieures de part et d’autre en C35, lamelles centrales en C24) a été testé. Une rupture en cisaillement a été observée pour 18 spécimens, les 12 restants ayant fait l’objet d’une rupture en flexion. Statistiquement, on considère un tel ensemble de données comme « incomplet », néanmoins, l’ensemble des résultats issus du mode de rupture en flexion peut être considéré comme observation censurée à droite de type I. Bien que cette typologie de données incomplètes soit observée dans une grande variété de circonstances dans les essais de matériaux, l’application de méthode d’analyse statistique adéquate est rare. L’estimation de paramètres de distribution appropriés pour de tels ensembles de données est de ce fait traitée de manière détaillée. La loi de distribution logistique (position, échelle) amenant des analyses particulièrement pratique sera mise en avant. Une distribution de Weibull à deux paramètres, s’apparentant à une forme log-logistique, représente l’ensemble des données de manière très adéquate.

La méthode du maximum de vraisemblance a été choisie pour analyser l’inference statistique. La précision de l’estimation des paramètres de la distribution de Weibull a été vérifiée aussi bien à l’aide de la région de confiance qu’au moyen des intervalles de confiance marginaux. La résistance caractéristique est alors déterminée comme l’intervalle de confiance unilatéral inférieur du quantile à 5% de la distribution de Weibull. La résistance caractéristique au cisaillement du bois lamellé-collé ainsi obtenue est très légèrement supérieure à celle publiée dans le nouveau code de dimensionnement allemand DIN 1052 :2004.

KEYWORDS: Glued laminated timber, incomplete observations, type I right-censoring mechanism, likelihood ratio procedures, characteristic shear value
1. INTRODUCTION

The shear strength of structural glued laminated timber (glulam) is frequently determined in full-size flexure tests. In a typical test setup, the glulam beams are supported at both ends and loaded by two single forces until ultimate load is reached. The dimensions are usually chosen so that the glulam beams exhibit a low span-to-depth ratio. Although such a test setup is likely to induce shear failure, it is a characteristic feature of such tests that only a certain proportion of the tested specimens fails in the targeted shear mode while the remainder of the specimens fails in flexure. The shear strength of the specimens which fail in shear is easily obtained. Those specimen, however, which fail in flexural mode only yield a shear stress containing the information of what the shear strength must be at least.

In the statistical analysis of such data sets including both, shear strengths and shear stresses as well, the question arises how the shear stresses can be incorporated adequately in the evaluation. A common engineering approach in such a situation is simply not to distinguish between shear strengths and shear stresses and to evaluate all data combined. An attractive alternative to that approach might be to drop the shear stresses and to analyze only the subset of the shear strength values. As the paper will reveal, both engineering approaches are inappropriate. While the first approach overestimates the information content the shear stresses provide by treating them as “strength values”, the second approach takes no advantage of the information the shear stresses can contribute to the analysis. Taking into account that often considerable costs are involved in material, manufacture and testing of glulam beams, the latter approach appears economically highly ineffective.

In the paper presented, the most important observation schemes of data are shortly discussed. It will turn out that observations for which only the lower bound of the failure strength is known (“strength equals at least a certain failure stress”) are referred to as Type I right-censored. The statistical analysis of data including Type I right censored observations is illustrated by the results obtained in a study where a total of thirty full-size structural glulam beams were tested.

As it is common in statistics, the parameters of an appropriate distribution will be estimated first. A thorough model assessment is essential before evaluating the data further. The model assessment will be done by graphical and by analytical means, as well. Statistical inference for the estimated parameters will
be performed by measuring their accuracy with a joint confidence region and marginal confidence intervals, respectively. In the framework of semi-probabilistic design of timber structures, the derivation of the characteristic shear value is of particular interest. The only meaningful way to establish characteristic strength values adequately is to define them as lower one-sided confidence intervals for a specified quantile of the chosen distribution. How this is accomplished in the presence of Type I right-censored observations concludes the paper.

2. OBSERVATION SCHEMES OF DATA

In practice, complete observations are most frequently encountered and their analysis is well described in most elementary textbooks on statistics.

In contrast hereto, incomplete observations are often encountered in the study of lifetime data. The most prevalent type of incomplete observations arises when the exact lifetime of a specimen put on test is not observed but is known to exceed a certain time. Such an observation, for which only a lower bound of the lifetime is known, is referred to as right-censored. Right-censored lifetimes might arise, for example, if some specimens put on a Duration-of-Load (DOL-) test are still “alive” at the end of the observation period. Two different cases of right-censoring mechanisms need to be distinguished. An observation is termed Type I right-censored, if \( n \) specimens are put on test and the experiment is terminated after some time before all specimens have failed. A Type II right-censoring mechanism is said to apply, when \( n \) units are put on test and the experiment is terminated as soon as \( r \) of \( n \) specimens have failed.

In a different scenario, a specimen put on a DOL-test is inspected for failure after some time. If the specimen has failed before the first inspection only an upper bound of the failure time is known. Such observations are referred to as left-censored data.

In some situations, specimens can be inspected only within certain time intervals. For example, it might be impossible to survey the specimens put on a DOL-test continuously. Instead, it might be more convenient to inspect these specimens daily. If failure occurred within such an inspection interval, the observations are termed interval-censored data.

Right-censored, left-censored or interval-censored observations represent the most prominent examples of incomplete data although a variety of other ob-
servation schemes exists which also lead to the analysis of incomplete data (e.g. truncated data). Lawless [1] gives a complete overview of possible censoring mechanisms.

The well-established procedures for the statistical analysis of censored lifetime data can be easily transferred to the statistical analysis of mechanical stresses. In the light of the glulam data, the shear strengths represent “observed lifetimes” whereas the shear stresses are equivalent to specimens whose observation of lifetime is terminated before the “actual” lifetime was reached. Hence, the shear stresses can be regarded as Type I right-censored observations. The remainder of this paper deals exclusively with this observation scheme.

3. MATERIAL, TEST SETUP AND TEST RESULTS

In the study conducted, a total of thirty full-size flexure tests on glulam were performed with the target to obtain shear failure. The dimensions of the glulam beams were width · depth · length = 140 mm · 456 mm · 3404 mm. The beams consisted of twelve laminations manufactured with spruce (Picea abies), each having a thickness of 38 mm. In order to prevent flexural failure, the lay-up of the laminations was combined: while the three outermost laminations in the tension and compression zone, respectively, consisted of machine graded timber of the strength class C35 according to the European standard EN 338 [2], the six core laminations were made of machine graded timber of the strength class C24. All specimens were manufactured using full-length laminations without any end joints. A melamin urea-formaldehyde adhesive approved for out-door applications was used for face bonding of the laminations.

The test setup was chosen according to a proposal in [3] and is schematically shown in Fig. 1. The glulam beams were simply supported at both ends. The two loads were applied by hydraulic cylinders at a loading rate so as to reach the ultimate load within 300 ± 60 seconds.

The shear strengths and shear stresses obtained in N/mm² were, sorted in increasing order, as follows:

\[
\begin{align*}
3.40^* & \quad 3.83 & \quad 3.87^* & \quad 4.11 & \quad 4.13 & \quad 4.20 & \quad 4.21^* & \quad 4.23^* & \quad 4.33^* & \quad 4.49^* \\
4.50 & \quad 4.53 & \quad 4.53^* & \quad 4.62^* & \quad 4.77 & \quad 4.78 & \quad 4.80^* & \quad 4.86^* & \quad 4.92 & \quad 4.97 \\
5.02 & \quad 5.02^* & \quad 5.09^* & \quad 5.18 & \quad 5.33 & \quad 5.44 & \quad 5.55 & \quad 5.55 & \quad 5.68 & \quad 5.94
\end{align*}
\]
Fig. 1  Schematic test setup of full-size flexure tests on structural combined glulam

The asterisks denote Type I right-censored observations (shear stresses at flexural failure). Among thirty specimens tested, only eighteen specimens failed in the targeted shear mode and twelve specimens failed in flexural mode.

4. STATISTICAL ANALYSIS

4.1 Type I Right-Censored Data and Maximum Likelihood Estimates

Suppose, that a random sample \( X \) comprises only complete observations which are independent and identically distributed with probability density function (p.d.f.) \( f(x; \theta) \). The maximum likelihood estimate (m.l.e.) \( \hat{\theta} \) is then obtained by maximizing the likelihood function

\[
L(\theta) = \prod_{i=1}^{n} f(x_i; \theta)
\]  

(1a)

In almost any case, it is more convenient to work with the log-likelihood function

\[
\ell(\theta) = \log L(\theta)
\]  

(1b)

A familiar approach to maximization of the log-likelihood function \( \ell(\theta) \) is the Newton-Raphson iteration; it uses the iteration scheme

\[
\theta_j = \theta_{j-1} - H(\theta_{j-1})^{-1} \cdot U(\theta_{j-1}), \quad j = 1,2,\ldots.
\]  

(2)

where \( U(\theta) = \frac{\partial \ell}{\partial \theta} \) denotes the first derivative (or score) vector and \( H(\theta) = \frac{\partial^2 \ell}{\partial \theta \partial \theta'} \) denotes the second derivative (or Hessian) matrix. As a re-
sult of the iteration, the m.l.e. $\hat{\theta}$ is obtained. Alternatively, numerical search
procedures that do not use any derivatives might be applied to find $\hat{\theta}$.

More generally, suppose that some of the observations in the random sample $X$ are Type I right-censored. Under such a censoring mechanism, the likelihood function takes the form

$$L(\theta) = \prod_{i=1}^{n} f(x_i; \theta)^{\delta_i} S(x_i; \theta)^{1-\delta_i}$$

where $\delta_i$ is called the censoring or status indicator. The censoring indicator is a binary random variable that equals 1 if the observation $x_i$ is uncensored (i.e. shear failure) and that equals 0 if the observation $x_i$ is Type I right-censored (i.e. flexural failure). The term $S(x; \theta)$ in eq. (3) denotes the survivor function (s.f.) of the p.d.f. $f(x; \theta)$ which is readily obtained by the equation

$$S(x; \theta) = Pr(X \geq x) = \int_{x}^{\infty} f(t; \theta) \, dt$$

Thus, for Type I right-censored observations the p.d.f. in the likelihood function $L(\theta)$ is merely replaced by its survivor function. It should be noted that under a right-censoring mechanism two random variables are involved: first, the random variable $X$ and second, the binary censoring indicator $\delta$.

4.2 Statistical Model and Parameter Estimation

Over the past decades, the two-parameter Weibull distribution with p.d.f.

$$f(x; \alpha, \beta) = \frac{\beta}{\alpha} \left(\frac{x}{\alpha}\right)^{\beta-1} \exp\left[-\left(\frac{x}{\alpha}\right)^{\beta}\right] \quad , \quad x \geq 0$$

has gained considerable importance in timber engineering. Hence, this distribution is chosen to model the frequency properties of the data including Type I right-censored observations reported at the end of chapter 3. Instead of working with the random variable $X$ directly, it often turns out to be more convenient to work with the transformed random variable $Y = \log X$. As will become evident, this transformation simplifies the estimation of m.l.e. $\hat{\theta}$ considerably. Performing the transformation $Y = \log X$ (for details see e.g. [4]) yields of what is known as Gumbel (or extreme value) distribution with p.d.f.
\[ f(y; u, b) = \frac{1}{b} e^{(y-u)/b} \exp[-e^{(y-u)/b}], \quad -\infty < y < \infty \quad (6) \]

where \( u = \log \alpha \) is the location parameter and \( b = 1/\beta \) is the scale parameter.

The Gumbel distribution has the advantage to be of location-scale form whereas the Weibull distribution is of log-location-scale form. Distributions of the location-scale family (e.g. normal distribution, logistic distribution) have the appealing feature that under a Type I right censoring mechanism simple closed-form equations of the score vector \( U(\theta) \) and the Hessian matrix \( H(\theta) \) exist.

For location-scale distributions, the likelihood function eq. (3) including Type I right-censored observations takes the form

\[ L(u, b) = \prod_{i=1}^{n} \left[ \frac{1}{b} f_0(z_i) \right]^{\delta_i} S_0(z_i)^{1-\delta_i} \quad (7a) \]

where \( z_i = (y_i - u)/b = (\log x_i - u)/b \). The functions \( f_0(z) \) and \( S_0(z) \) in eq. (7a) denote the standardized probability density and survivor function, respectively. The corresponding log-likelihood function \( \ell(\theta) = \log L(\theta) \) (eq. (1b)) is readily seen to be

\[ \ell(u, b) = -r \log b + \sum_{i=1}^{n} \left[ \delta_i \log f_0(z_i) + (1 - \delta_i) \log S_0(z_i) \right] \quad (7b) \]

where \( r = \sum_{i=1}^{n} \delta_i \).

The components of the score vector \( U(\theta) = \partial \ell / \partial \theta \) are found to be

\[ U_1 = \frac{\partial \ell}{\partial u} = -\frac{1}{b} \sum_{i=1}^{n} \left[ \delta_i \frac{\partial \log f_0(z_i)}{\partial z_i} + (1 - \delta_i) \frac{\partial \log S_0(z_i)}{\partial z_i} \right] \quad (8a) \]

\[ U_2 = \frac{\partial \ell}{\partial b} = -\frac{r}{b} - \frac{1}{b} \sum_{i=1}^{n} \left[ \delta_i z_i \frac{\partial \log f_0(z_i)}{\partial z_i} + (1 - \delta_i) z_i \frac{\partial \log S_0(z_i)}{\partial z_i} \right] \quad (8b) \]

while the components of the Hessian matrix \( H(\theta) = \partial^2 \ell / \partial \theta \partial \theta \) take the form

\[ H_{11} = \frac{\partial^2 \ell}{\partial u^2} = \frac{1}{b^2} \sum_{i=1}^{n} \left[ \delta_i \frac{\partial^2 \log f_0(z_i)}{\partial z_i^2} + (1 - \delta_i) \frac{\partial^2 \log S_0(z_i)}{\partial z_i^2} \right] \quad (9a) \]
The standardized probability density and survivor function of the Gumbel distribution can be expressed as

\[ f_0(z) = e^z \exp\left(-e^z\right), \quad S_0(z) = \exp\left(-e^z\right) \]  

The first and second derivatives of \( \log f_0(z) \) and \( \log S_0(z) \) needed for calculating the components of the score vector \( U(\theta) \) (eqs. (8a,b)) and the Hessian matrix \( H(\theta) \) (eqs. (9a-d)) follow from eqs. (10a,b) immediately

\[
\frac{\partial \log f_0(z)}{\partial z} = 1 - e^z, \quad \frac{\partial^2 \log f_0(z)}{\partial z^2} = -e^z
\]

\[
\frac{\partial \log S_0(z)}{\partial z} = -e^z, \quad \frac{\partial^2 \log S_0(z)}{\partial z^2} = -e^z
\]

With the components of the score vector \( U(\theta) \) and the Hessian matrix \( H(\theta) \) known, the Newton-Raphson iteration according to eq. (2) can be performed. For the data under consideration, the m.l.e. \( \hat{\theta} = \left(\hat{\alpha}, \hat{\beta}\right) = (1.666, 0.0919) \) of the Gumbel distribution is obtained.

The simple transformations \( \hat{\alpha} = \exp(\hat{\alpha}) = \exp(1.666) = 5.289 \) and \( \hat{\beta} = 1/\hat{\beta} = 1/0.0919 = 10.876 \) finally yield the m.l.e. of the Weibull distribution.
4.3 Model Assessment

Before performing statistical inference, a thorough assessment of the parametric model is essential. Descriptive plots are a common tool for model assessment. Although being subjective, they provide a useful impression of the appropriateness of the chosen parametric model. Usually, a formal goodness-of-fit test supplements such plots. Here, the latter is omitted in favour of an analysis of the shape parameter of the generalized log-gamma distribution which allows a discrimination between the Weibull distribution on the one hand and the log-normal distribution on the other hand. The latter distribution is also frequently applied in timber engineering.

4.3.1 Graphical Model Assessment

Nonparametric frequency estimates for complete random samples are well-known. In case of a random sample including Type I right-censored observations, however, these estimates are not applicable. Kaplan and Meier proposed in 1958 an approach which allows a nonparametric estimate of the cumulative distribution function (c.d.f.) for any right-censored random sample. For details of the calculation, see [1,5]. In Fig. 2, the Kaplan-Meier estimate is plotted as a step function with the Weibull c.d.f. overlaid. As can be seen, there is no graphical evidence against the two-parameter Weibull model.
4.3.2 Analytical Model Assessment

The generalized log-gamma distribution is a further representative of the location-scale distribution family with location parameter $u$ and scale parameter $b$. In addition, it includes a shape parameter $k$. The shape parameter has the interesting feature to allow a discrimination between the Weibull distribution and the log-normal distribution. Letting $Y = \log X$ and $Z = (Y - u)/b$, the p.d.f. and s.f. of the generalized log-gamma distribution are given in standardized form as

\[
f_0(z; k) = \frac{k^{k-1/2}}{\Gamma(k)} \exp\left[k^{1/2} z - k \exp\left(z k^{-1/2}\right)\right] \quad (12a)
\]

\[
S_0(z; k) = 1 - I\left(k, k \exp\left(z k^{-1/2}\right)\right) \quad (12b)
\]

where $\Gamma(k) = \int_0^\infty u^{k-1} e^{-u} du$ and $I(k, x) = \frac{1}{\Gamma(k)} \int_0^x u^{k-1} e^{-u} du$ denote the complete and incomplete gamma function, respectively.

In the limit as the shape parameter $k \to \infty$, $f_0(z; k)$ approaches the p.d.f. of the standard normal distribution and as $k \to 1$, $f_0(z; k)$ approaches the p.d.f. of the Gumbel distribution. Similarly, the same holds true for the survivor function. Recall, that the normal and the log-normal distribution as well as the Gumbel and the Weibull distribution are related to each other by the simple transformation rule $Y = \log X$, respectively. Therefore, the shape parameter $k$ allows a discrimination between the log-normal and Weibull distribution, too.

Performing the Newton-Raphson iteration according eq. (2) in association with the score vector and Hessian matrix for location-scale models presented in section 4.2, the maximum likelihood estimate of the generalized log-gamma distribution is found to be $\hat{\theta} = (u, b, k) = (1.664, 0.0929, 1.069)$. The shape parameter $k = 1.069$ is very close to 1 which indicates that the two-parameter Weibull model is appropriate. In contrast hereto, the log-normal model would be inappropriate for the Type I right-censored random sample under consideration.

4.4 Statistical Inference

Exact statistical inference procedures for random samples including Type I right-censored observations are mathematically intractable. There exists, however, a variety of approximate methods for the analysis of such data. Among the most important are score procedures, maximum likelihood based procedures and
likelihood ratio procedures. Wald-based inference procedures assuming approximately normally distributed pivotal quantities as well as bootstrap methods which apply simulation of the distributional properties of pivotal quantities provide convenient alternatives.

Subsequently, the likelihood ratio statistic defined by

\[
\Lambda(\theta) = -2 \log \left[ \frac{L(\theta)}{L(\hat{\theta})} \right] = 2 \ell(\hat{\theta}) - 2 \ell(\theta)
\]  

(13)

is favoured. In large samples, the maximum likelihood estimate \( \hat{\theta} \) is approximately multivariate normally distributed with \( N_p \left( \theta; I^{-1}(\theta) \right) \) where

\[
I(\theta) = E \left( - \frac{\partial^2 \ell}{\partial \theta \partial \theta} \right)
\]

denotes the Fisher (or expected) information matrix. The likelihood ratio statistic \( \Lambda(\theta) \) is then approximately \( \chi^2 \)-distributed.

4.4.1 Approximate joint confidence region for \( \hat{\theta} \)

The m.l.e. of a parameter vector is of little value unless it is known how accurate it is likely to be. Hence, measurement of the extent of this uncertainty is an important part of the statistical problem. First, let us obtain an approximate joint confidence region for the m.l.e. \( \hat{\theta} = (\hat{a}; \hat{b}) = (1.666, 0.0919) \) of the Gumbel distribution discussed in section 4.2. In terms of likelihood ratio procedures, an approximate joint confidence region is given as the contour of all points \((u, b)\) satisfying \( \Lambda(u, b) \leq \chi^2_{p:1-\alpha} \), where

\[
\Lambda(u, b) = 2 \ell(\hat{u}, \hat{b}) - 2 \ell(u, b)
\]

(14)

denotes the likelihood ratio statistic and \( \chi^2_{p:1-\alpha} \) is the quantile of the \( \chi^2 \)-distribution with \( p \) degrees of freedom and confidence level \( 1 - \alpha \). Figure 3 shows the contour of the approximate confidence region for the m.l.e. \( \hat{\theta} \) assuming a confidence level of \( 1 - \alpha = 0.95 \). As two parameters are involved, the \( \chi^2 \)-distribution has \( p = 2 \) degrees of freedom. Under these assumptions, the quantile becomes \( \chi^2_{2:0.95} = 5.991 \).
4.4.2 Approximate two-sided marginal confidence intervals for $\hat{\theta}$

An approximate two-sided marginal confidence interval concerning $u$ is obtained by the likelihood ratio statistic

$$\Lambda_1(u_0) = 2 \ell(\hat{u}, \hat{b}) - 2 \ell(u_0, \bar{b}(u_0))$$

(15a)

where $\bar{b}(u_0)$ is the maximum likelihood estimate for $b$ when $u = u_0$. This is obtained by maximizing $\ell(u_0, b)$ with respect to $b$. The marginal confidence interval concerning $u$ is then given as the set of points satisfying $\Lambda_1(u_0) \leq \chi^2_{p;1-\alpha}$. In a similar way, an approximate two-sided marginal confidence interval concerning $b$ is obtained by using

$$\Lambda_2(b_0) = 2 \ell(\hat{u}, \hat{b}) - 2 \ell(\tilde{u}(b_0), b_0)$$

(15b)

where $\tilde{u}(b_0)$ maximizes $\ell(u, b_0)$ when $b=b_0$. Similarly, the marginal confidence interval is given as the set of points satisfying $\Lambda_2(b_0) \leq \chi^2_{p;1-\alpha}$. In Figs. 4a,b, the likelihood ratio statistics $\Lambda_1(u_0)$ and $\Lambda_2(b_0)$ as well as their intersections with the limiting quantile $\chi^2_{p;1-\alpha}$ are shown graphically.
Again, the confidence level was assumed to be $1 - \alpha = 0.95$. As the parameters are now considered separately, the degree of freedom of the $\chi^2$-distribution reduces to $\nu = 1$ so that the quantile becomes $\chi^2_{1, 0.95} = 3.841$.

The approximate two-sided marginal 0.95-confidence intervals for $u$ and $b$ are found to be $1.623 \leq u \leq 1.713$ and $0.0677 \leq b \leq 0.132$. These marginal confidence intervals are shown in Fig. 3 with lines in dashed style along with the joint confidence region for the m.l.e. $\hat{\theta}$.

The transformations $\alpha = \exp(u)$ and $\beta = 1/b$ yield the approximate two-sided marginal 0.95-confidence intervals for the parameters of the Weibull distribution; they are obtained as $5.069 \leq \alpha \leq 5.548$ and $7.561 \leq \beta \leq 14.780$. It can be seen that the confidence interval of the location parameter $\alpha$ is quite narrow indicating its precise estimation. The confidence interval for the scale parameter $\beta$, however, is fairly wide thus emphasizing the need of a greater sample size in order to obtain a more precise estimate.

### 4.4.2 Approximate one-sided lower confidence interval for the 5%-quantile

For location-scale models, the $q$th quantile $\hat{y}_q$ for $Y = \log X$ is

$$\hat{y}_q = \hat{u} + w_q \cdot \hat{b}$$

(16a)
where \( \hat{u} \) and \( \hat{b} \) are the m.l.e. and \( w_q = F_0^{-1}(q) \) denotes the \( q \)th quantile of the standardized c.d.f. \( F_0(z) = 1 - S_0(z) \). For the Gumbel distribution, the \( q \)th quantile of the standardized c.d.f. takes the form

\[
w_q = \log \left[ - \log (1 - q) \right]
\]

The related \( q \)th quantile of the Weibull distribution is easily obtained by the transformation \( \hat{x}_q = \exp \left( \hat{y}_q \right) \).

Considering mechanical strength properties, the 0.05-quantile is usually of particular interest. Inserting the m.l.e. obtained in section 4.2 and eq.(16b) into eq. (16a) yields the quantile for \( Y \) (Gumbel distribution)

\[
\hat{y}_{0.05} = \hat{u} + \log \left[ - \log (1 - q) \right] \cdot \hat{b} = 1.666 + \left( -2.970 \right) \cdot 0.0919 = 1.393
\]

(17a)

The 0.05-quantile for \( X \) (Weibull distribution) is

\[
\hat{x}_{0.05} = \exp \left( \hat{y}_p \right) = \exp \left( 1.393 \right) = 4.03 \text{ N/mm}^2
\]

(17b)

For the calculation of the quantile according to eq. (16a) the m.l.e. \( \hat{\theta} = (\hat{u}, \hat{b}) \) is needed. The joint confidence region shown in Fig. 3 illustrates graphically where the m.l.e. \( \hat{\theta} \) can be expected to lie in the parameter plane in a repetition of the experiment. From the hatched contour plotted it can be easily seen that the 0.05-quantile derived so far is totally inappropriate to establish a characteristic shear value as it is obtained entirely random. The link between the observed sample quantile and the population is provided by a one-sided lower confidence interval.

An approximate one-sided lower confidence interval concerning \( \hat{y}_q \) is obtained by the likelihood ratio statistic

\[
\Lambda \left( y_{q0} \right) = 2 \ell \left( \hat{u}, \hat{b} \right) - 2 \ell \left( \tilde{u} \left( y_{q0} \right), \tilde{b} \right)
\]

(18a)

where \( \left( \tilde{u} \left( y_{q0} \right), \tilde{b} \right) \) maximizes \( \ell (u, b) \) when \( y_q = y_{q0} \). Since for location-scale models the relation \( y_q = u + w_q \cdot b \) holds, we merely need to maximize

\[
\ell_1 (b) = \ell \left( y_{q0} - w_q \cdot b, b \right)
\]

(18b)

with respect to \( b \) in order to get \( \tilde{b} \), and then \( \tilde{u} = y_{q0} - w_q \cdot \tilde{b} \).
The approximate one-sided lower confidence interval consists of all values \( y_{q0} \) satisfying
\[
I \left( y_{q0} < \hat{y}_{q0} \right) \Lambda \left( y_{q0} \right) \leq \chi^2_{1; 1-2\alpha}
\] (19)
where \( I \left( y_{q0} < \hat{y}_{q0} \right) \) is a binary indicator function that equals 1 if the inequality is true and 0 if it is not true.

In Fig. 5, the likelihood ratio statistic \( \Lambda \left( y_{q0} \right) \) according to eqs. (18a,b) is shown graphically for \( q = 0.05 \). In many European standards (e.g. EC 5, EN 1058, EN 14358), the confidence level \( 1-\alpha = 0.841 \) is proposed. Under this assumption, the quantile of the \( \chi^2 \)-distribution becomes \( \chi^2_{1; 0.683} = 1 \) which is shown as horizontal line in Fig. 5. The left intersection of this line with the likelihood ratio statistic \( \Lambda \left( y_{q0} \right) \) yields the lower limit of the one-sided confidence interval.

For the data under consideration, the approximate one-sided lower 0.841-confidence interval is found to be \( y_{0.05; 0.841} = 1.334 \). The transformation to the Weibull distribution yields \( x_{0.05; 0.841} = \exp(1.334) = 3.80 \text{ N/mm}^2 \). Alternatively, for the more conservative confidence level \( 1-\alpha = 0.95 \) the lower limit of the one-sided confidence interval becomes \( x_{0.05; 0.95} = 3.63 \text{ N/mm}^2 \). In Fig. 2, these confidence intervals are shown as detail in the inserted graphics.
5. CONCLUSIONS

In the paper presented, it was reported on a total of thirty full-size flexure tests conducted on structural combined glued laminated timber in order to determine its shear strength. Only 60% of the specimens were found to fail in the targeted shear mode whereas 40% of the specimens failed in flexural mode. The data set obtained, consisting of shear strengths and shear stresses at flexural failure as well, was identified as a random sample including Type I right censored observations.

Random samples including Type I right censored observations are a special case of the more general statistical theory of incomplete data. While statistical evaluation routines for incomplete data are an important topic in many disciplines such as medicine, social sciences as well as in mechanical and electrical engineering, they are rarely if ever applied in timber engineering. Hence, the paper laid particular emphasis on the adequate statistical analysis of the incomplete shear data obtained in the flexural tests conducted on combined glued laminated timber.

Both, graphical and analytical model assessment proved that a two-parameter Weibull distribution, being of log-location-scale form, fitted the data adequately. The parameter estimation and statistical inference, however, were for sake of simplicity performed applying the Gumbel distribution being of location-scale form. For this family of distributions, convenient closed-form equations exist. The results for the Weibull distribution were obtained by means of simple transformation rules, respectively.

The contour of the joint parameter region was plotted in order to illustrate where the estimated parameter vector might be located in a repetition of the experiment. The marginal confidence intervals revealed that the location parameter of the Weibull distribution was estimated rather precisely; the marginal confidence interval for the scale parameter, however, was comparatively wide thus emphasizing the necessity of a greater sample size.

The characteristic shear value was derived as one-sided lower confidence interval for the 5%-quantile of the Weibull distribution. For both confidence levels considered (84.1% and 95%), the characteristic shear values were found to be $f_{v,k} = 3.80 \, \text{N/mm}^2$ and $f_{v,k} = 3.63 \, \text{N/mm}^2$, respectively. Thus, both...
values were slightly above the characteristic value $f_{v,k} = 3.5 \text{ N/mm}^2$ specified in the new German timber design code DIN 1052 [6].

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REFERENCES


SCANNING IMPACT-ECHO TECHNIQUES FOR CRACK DEPTH DETERMINATION

MARKUS KRÜGER

SUMMARY

Concrete cracking often results in stiffness loss and also corrosion of the steel reinforcement. The corrosion rate strongly depends on the crack width and the crack depth so there is the need to measure and to characterize its dimensions. The impact-echo technique is a useful tool for the detection of different faults in concrete structures. Unfortunately, the existing instruments and analysis tools designed for measurements were lacking of several features in the past detaining the extensive use. Therefore a new concept for impact-echo testing systems is presented. A new device was developed, which is small, robust and easy to handle. The system utilizes advanced impact generation for fast scanning techniques and reproducible impacts. The data acquisition, filtering and visualization of data are optimized for the inspection of large structures obtaining data at many measurement points. A new option is the automatic estimation of crack parameters like crack depth. In this paper fundamental principles of the different measurement techniques as well as details of the measurement system are described and some test results are shown.

ZUSAMMENFASSUNG


**RESUME**

La fissuration du béton armé mène en général à une réduction de rigidité et à un risque accru de corrosion des armatures. Ce risque dépend en particulier de la largeur et de la profondeur des fissures, la détermination de ces dernières est donc d'importance particulière pour la durabilité. La méthode impact-écho est utilisée pour l'inspection non destructive des constructions en béton et la caractérisation de différents défauts. À cause de l'envergure de la mise en œuvre des essais et de leur évaluation pour les grandes surfaces, cette méthode n'a été appliquée qu'occasionnellement jusqu'à présent. De nouveaux concepts pour la méthode impact-écho sont présentés dans cet article. Un nouvel appareillage et de nouveaux logiciels ont été développés, permettant une mise en œuvre et une évaluation des essais à la fois plus objectives et largement automatisées. Tant l'utilisation de méthodes à balayage que l'implémentation et la combinaison de différentes méthodes d'analyse des signaux ont simplifié l'inspection de grands éléments de construction. Outre les applications connues de la méthode impact-écho dans l'inspection des constructions, de nouvelles approches pour la détection et la caractérisation automatiques des fissures verticales sont discutées.

**KEYWORDS:** Scanning Impact-Echo techniques, crack determination, non-destructive testing

1 **INTRODUCTION**

Due to the demands for quality control and sustainability of structures in civil engineering, a growing market for non-destructive testing has evolved. For concrete structures, several methods are well-introduced concerning defect char-
acterisation. Ultrasound, radar, thermography, electro-potential-field methods and others are currently being used to detect voids, cracks, corrosion, etc. – with varying success. Several years ago, the Impact-Echo (IE) method, that considerably improved the detection of voids and honeycombing, was introduced by Carino et al. [1]. The strength of this method is its ability to detect voids in structures and to measure the thickness of concrete parts with good accuracy. For that reason, Impact-Echo was chosen to be the standard technology for quality control of tunnels in Germany [2]. It seems that IE is the first non-destructive technology to be part of a regulating standard for quality control in civil engineering in Germany. However, this technology is still not widely accepted due to the poor handling and limited functionality of commercially available equipment based on this approach. Moreover it was shown that single-point measurements are somehow more difficult to be interpreted compared to measurements using a scanning technique [3]. The so-called scanning IE technique was developed from measurements that were carried out by Weiler [4] and later on by Grosse and Weiler [5] as well as Kretschmar [6]. A more sophisticated approach using a static scanning frame was described by Colla et al. [3] and Lausch et al. [7]. It is obvious that the potential of this technique is currently not being used to its full extent regarding handling as well as analyzing techniques, therefore reducing the economic value of this method.

Based on the development of a new hardware [8] some improvements regarding void as well as crack detection were made, which are described in the following.

IMPACT-ECHO SYSTEM

Measurement Hardware

Up to now the number of commercially available Impact-Echo systems is limited. The data acquisition and analysis capabilities of these systems are very similar. With such equipment, repetitive velocity measurements are neither practical nor cost-effective although a constant velocity cannot be expected to be present in large structures. In general, Impact-Echo testing with currently available equipment takes up to two minutes per measurement point for data acquisition and verification of results. Considering the poor ergonomics of such devices, two operators are often necessary to handle the equipment. As personnel costs are crucial for in-situ measurements, reducing the complexity of the testing
process is essential for further acceptance of this technique. Otherwise, only measurements at selected single points or with relatively wide grids are feasible.

![Figure 1. DAI-1 impactor and sensor (left) together with the tablet-PC control unit (right).](image)

For that purpose a test system has been developed (see Figure 1). On the hardware side, the system consists of a transducer and an automatic impactor as well as a data acquisition PC-Card. The equipment is light, mobile and controlled by a rugged sub-notebook or a tablet-PC. The device is optimized for rough environments and a fast and easy data acquisition. For the detection of voids and cracks, the impact should generate a short relatively high energy but nevertheless non-destructive pulse with broad frequency content. High impact energy is necessary to detect defects and boundary surfaces in greater depth. The developed impactor operates on the basis of high speed tubular solenoids. It is equipped with an electronic control unit interfacing to external devices that allows the operator to fully control the impact generation and also gives feedback on impact time and duration. As the unit is able to deliver the exact time of impact, a second transducer so far required for velocity measurements is now obsolete [9], [10]. With the unit it is also possible to develop new fast inspection methods like the proposed methods of vertical crack detection and characterisation.

**Measurement Software**

The Impact-Echo technique is in general a punctiform test method that means that one measurement only gives information about one point of the structure (A-Scan). To get a better idea of the structure it is more useful to use scanning techniques measuring at multiple points, e.g. one could look at several points in a line (B-Scan) or could measure a whole surface (C-Scan). If a
Scanning impact-echo techniques for crack depth determination

B-Scan or a C-Scan is considered, a lot of single measurements have to be made and also combined which is very complex. Nowadays this combination of the test results is often done manually that is very time consuming. The measurement software, which is presented in this paper, therefore accounts for the following aspects: easy handling for fast and competitive measurements; automatic measurement grid generation for surfaces with unlimited measurement points; flexible toolbox with mathematical functions for determining automatic and semiautomatic measurement and analysis procedures (for crack/flaw detection and characterization); implementation of tools for the flexible graphical representation of A-Scans, B-Scans and C-Scans that will allow an easy real-time analysis during the measurement procedure.

IMPACT-ECHO MEASUREMENT METHODS

Standard Measurement Technique

The Impact-Echo method uses transient stress waves generated on the surface of concrete or masonry structures by an elastic impact (Figure 2). As the stress waves propagate through the material being tested, they are reflected by internal interfaces (discontinuities in the material) and external boundaries of the structure. Examples of such interfaces are delaminations, voids, honeycombing and cracks, as well as rising mains or large steel bars. In order to detect such interfaces, the emitted waves are recorded by a displacement or acceleration transducer which is placed near the impact point on the surface of the structure.

![Figure 2. Principle of impact-echo measurements to detect boundaries or voids.](image)

The depth of any internal flaws or external interfaces can be determined by analyzing the recorded signal and its characteristic frequency spectrum (FFT) using the following simple equation

\[ d = \frac{v_p}{2 \cdot f_R} \]
where \(d\) is the depth of the interface or void, \(v_p\) is the measured compressional wave velocity and \(f_R\) is the resonance frequency in the spectrum corresponding to the period \(T\) of the wave. Usually the resonance frequency is the dominant frequency in the spectrum. Together with the previously measured compressional wave velocity of this structure, the depth of the void can be evaluated from equation (1).

**Principles of Crack Detection using Impact-Echo**

One principle of crack detection using the proposed test setup is similar to time of flight techniques [11], [12]. A signal emitted by the impactor will be detected after a certain travel time \(t\) and with a certain amplitude or energy, respectively (see Figure 3). If a surface crack with a tip depth \(d'\) is present between emitter and sensor, a time delay \(\Delta t\) occurs in the signal with the following relation to the original travel time \(t\):

\[
\Delta t = (t_1 + t_2) - t
\]

With knowledge of the wave speed of the compressional wave \(\Delta t\) will correlate with the crack tip depth. Unfortunately, the time delay depends very strong on the material filling the gap between the crack edges since there is usually not only air in between. Additional effects are caused by the reinforcement able to bridge the crack flanks.

![Figure 3. Principle of IE measurements for crack detection with the time of flight method.](image)

Therefore it is appropriate to use the energy of the emitted signal as recorded by the sensor ensuring that the emitter produces a highly reproducible constant signal. This is especially true for the used electronic impactor. Tests have shown that the cumulative energy (samplewise addition of the squared amplitude of the received signal) is a good discriminator between concrete surfaces with and without cracks. The peak amplitude of a time signal travelling across a crack is delayed and the overall energy is significantly lower compared to a
wave travelling along an undisturbed surface, because a part of the impact energy is reflected at the crack surface. Figure 4 shows the principle of this technique that can be used for automatic crack detection and in future developments for a determination of other crack parameters like crack depth and width.

**Figure 4. Principle of IE measurements for crack detection considering signal amplitude and signal energy as criteria.**

**Figure 5.** Measured signals obtained across a crack at a crack width of <0.1mm (upper curve) and at a crack width of approximately 0.7mm (lower curve).
An example of a recorded signal measured across a crack is given in Figure 5. It can be seen that the maximum signal amplitude and the onset time of the signal depends on the crack width and thus also crack depth.

Figure 6 shows the cumulated energy of two measured signals. The blue curve represents the cumulated energy of a measurement across an undisturbed surface and the red curve represents the cumulated energy of a measurement obtained across a crack. It can be clearly seen that the cumulated energy of the measured wave decreases with an increasing crack depth. However, the cumulated energy of the signal gives also a good hint for the quality of the concrete. For a characterization of a crack the partial derivative of the cumulated energy of the first part of the curves could be taken (see the slope in the area of interest in Figure 6). If the partial derivative is positive no crack is existent (descending slope in Fig. 10). If on the other hand the partial derivative becomes negative a crack is existent and if the partial derivative becomes more negative this implicates larger crack depth (ascending slope in Fig. 10).

![Figure 6. Cumulated energy (amplitude) calculated for the first 5000 samples of received signals obtained across cracks (lower curve) and across undisturbed surfaces (upper curve).](image)

An interesting aspect is that the new developed Impact-Echo measurement technique as shown above provides different values simultaneously, which could be used for the characterization of the observed structure. The value which is normally used for the detection of flaws is the resonance frequency in the spectrum. However, with the new developed measurement system it is possible to use the Impact-Echo technique to receive additional values e.g. onset time for the wave speed determination and the time of flight technique and in addition
the cumulated energy for a more precise characterization of the inspected structure. Horizontal and vertical cracks and flaws as well as the concrete quality could now be characterized in one single measurement.

LABORATORY TESTS

To prove the theories of crack detection and crack characterization it was needed to make some preliminary tests with a well known test set-up. Therefore, a test specimen was produced, in which some cracks could be initiated by inserting splints with a hammer (see Figure 7). This allows producing realistic cracks with different crack width and crack depth.

Figure 7. Test specimen for laboratory tests (Cracks initiated by splines)

First test were made with different crack width of approximately 0.1, 0.2, 0.4 and 0.7 mm. The crack width was controlled by inserting the splints differ-
ently. A measurement grid of 25mm was used (Figure 8) which is less than the distance of 75mm between the sensor and the impactor. The measurement unit was relocated 6 times starting at 120cm (see Figure 8, left sketch). The first five measurements are without a crack and the sixth measurement was across the initiated crack.

Figure 9 shows the test results of the measured signals in the time domain for different crack width in a B-Scan that could be generated by the software automatically. The results show that the peak energy of a time signal travelling across a crack (measurement No. 6) is delayed and of lower amplitude as well as the overall energy is significantly lower (for details see also Figure 5) compared to a wave travelling along an undisturbed surface, because a part of the impact energy is reflected at the crack surface (see Figure 4). With regard to different crack openings induced by the splines one could also observe a decrease of signal energy with increasing crack width that corresponds to the theory. This is also shown in Figure 10, in which the maximum signal amplitude is plotted against crack width.

![Figure 9. Test results from measurements at different crack width (B-scan of signals).](image-url)
A combination of the described methods can be used for automatic crack detection and in future for a determination of other crack parameters like crack depth and width. However, the signal amplitude and the signal energy could be used as criteria only if an internal horizontal interface or external boundary exists at the inspected structural part. As a conclusion it could be shown that the test method permits the detection of vertical cracks in general as well as crack depth.

OUTLOOK AND FUTURE WORK

Regarding the civil engineering industry an increasing demand for quality control of structures can be observed. Advanced Impact-Echo testing techniques that are easy to use for fast, repeatable and reproducible measurements can be used to improve the existing techniques or to replace visible inspections detecting voids or cracks. The benefits are already obvious to be the one-sided access and the easiness to conduct measurements saving time and money and bring the inspection on a more reliable and objective level.

It was shown that the IE technique has the potential to detect precisely large voids, honeycombs and inhomogeneities as well as the thickness of concrete structures. The new developed device reduces the time necessary for each measurement by a factor of ten. Some new methods are described concerning the detection of cracks. First promising results were shown using the cumulative
energy of the transmitted signal as a crack discriminator. Essential is that the new system provides a more reliable impact generation.

These newly developed methods including a new software front end IEDA 2.0 will be evaluated and calibrated during further tests. The tests will give more details about the reproducibility and reliability of this method. After that the next actions are the development, evaluation and implementation of proper algorithms for semiautomatic or automatic crack detection into software.

2 REFERENCES


INVESTIGATIONS ON IN-PLANE LOADED WOODEN ELEMENTS – INFLUENCE OF LOADING AND BOUNDARY CONDITIONS

UNTERSUCHUNGEN AN SCHEIBENBEANSPRUCHTEN HOLZ-WANDELEMENTEN – EINFLUSS DER BELASTUNG UND DER LAGERUNGSBEDINGUNGEN

ETUDE DES MURS EN BOIS CHARGES DANS LEUR PLAN – INFLUENCE DU CHARGEMENT ET DES CONDITIONS D'APPUI

Bruno Dujic, Simon Aicher, Rocco Zarnič

SUMMARY

Investigations on in-plane loaded wooden wall elements are part of a comprehensive research program performed in recent years at University of Ljubljana to enable better understanding of response of wooden buildings exposed to earthquake action. Recently, Division of Timber Construction of MPA University of Stuttgart joined the research program. This paper reports on some results emerging from the research cooperation.

Eurocode 5 contains two methods for determination of the racking strength of cantilever-type wall diaphragms: an analytical approach and an experimental approach using the test protocol according to EN 594. Both approaches are related only to timber frame walls having sheathing plates. The test procedure according to EN 594 predefines the partially anchored wall which does not necessarily represent the actual anchorage and loading conditions in the building and does not apply for cyclic horizontal loads to simulate earthquake loadings. It is reported on experimentally obtained responses of wall elements with different build-ups exposed both to the EN 594 protocol and to cycling loading. In detail three different cases of boundary conditions that may occur in real structures and the influence of additional vertical loads is regarded.
ZUSAMMENFASSUNG


RESUME

L'étude du comportement des murs en bois ou matériaux dérivés chargés dans leur plan fait partie d'un large projet de recherche réalisé à l'université de Ljubljana ces dernières années. Le but de ce projet est de mieux comprendre le comportement des constructions en bois soumises à des charges sismiques. Le département bois de la MPA de l'université de Stuttgart participe à ce projet depuis quelque temps dans le cadre d'une coopération. Cet article rend compte des premiers résultats des travaux de recherche communs.

L'Eurocode 5 contient deux approches pour déterminer la capacité portante en cisaillement des murs en porte-à-faux: une approche analytique et une méthode expérimentale utilisant le protocole de chargement selon EN 594. Ces deux méthodes sont limitées à l'application aux murs à ossature avec des panneaux latéraux. La méthode expérimentale selon EN 594 définit un ancrage par-
tiel des murs, ce qui ne représente pas nécessairement l'ancrage réel des murs et la sollicitation du bâtiment. Le protocole ne peut pas être appliqué à des charges horizontales cycliques simulant l'action de séismes. Cet article rend compte du comportement de murs de différentes configurations sollicités selon EN 594 et par un chargement cyclique. En détail, l'influence de trois différentes conditions d'appui pouvant apparaître dans un bâtiment et des charges verticales supplémentaires est analysé.

KEYWORDS: Wall elements, diaphragms, racking strength, boundary conditions, loading protocols, cyclic loading, element responses

1. INTRODUCTION

The post earthquake observations of damaged wooden houses and analysis of experimentally tested structural elements developed the worldwide knowledge about response of wooden buildings on earthquake and strong wind. One of the major problems of understanding is related to boundary conditions and influence of vertical loading on building elements. Learning from experimental and on-site observations researchers have developed different test protocols and test set-ups aiming at a simulation of the natural behaviour of buildings as realistic as possible. Some of those efforts are reflected in codes and standards.

Eurocode 5 introduces two methods for determination of the racking strength of cantilever-type wall diaphragms: i) an analytical approach and ii) an experimental approach using a test protocol according to EN 594. Both approaches are related exclusively to timber frame walls with sheathing plates. However, the current construction practice introduces many other types of wooden wall diaphragms. Amongst them, increasingly very popular are multi-layer board or perforated glued elements and braced walls with different diagonal strengthening.

The Eurocode 5 calculation procedure is based on the lower value of the plastic capacity of the fasteners which connect the sheathing plates to the timber frame. The approach however is exclusively applicable to the assessment of the racking strength of elements having wood based sheathing plates and frame studs which are fully restrained. In the cases of partially anchored studs and low magnitudes of vertical loading, the calculation may result in load capacity estimates that significantly overestimate the time load-bearing capacity [DUJIC
2001] and [DUJIC AND ZARNIC 2002]. The test procedure according to EN 594 requests a partially anchored wall that does not necessarily represent the actual wall diaphragm used for the construction of the wooden buildings. Further, the EN 594 load protocol does not use a cyclic horizontal load to simulate earthquake loading. It is obvious from the above, that both, analytical and experimental methods addressed in Eurocode 5 need to be upgraded.

In this paper experimentally obtained responses of wall elements exposed both to the EN 594 protocol and to cycling loading are presented. Three different cases of boundary conditions that may occur in real structures were applied and the magnitude of the constant vertical load was varied.

2. THE TEST APPROACH

2.1 Boundary conditions of shear wall elements

Basically, three major cases of boundary conditions are most likely to appear in reality:

- **shear cantilever mechanism**, where one edge of the panel is supported by the firm base while the other can freely translate and rotate (“Case A”, Fig. 1)

- **restricted rocking mechanism**, where one edge of the panel is supported by the firm base while the other can translate and rotate as much as allowed by the ballast that can translate only vertically without rotation (“Case B”, Fig. 1)

- **shear wall mechanism**, where one edge of the panel is supported by the firm base while the other can translate only in parallel with the lower edge and rotation is fully constrained (“Case C”, Fig. 1)

In “Cases A and B” the wall panel is exposed to a constant vertical load at every stage of the cycling excitation or horizontal deformation induced along the upper edge where the ballast is acting. In “Case C” the vertical load increases when the panel intends to uplift due to displacements along the upper horizontal edge. The advantage of the herein proposed testing procedures following the “Cases A and B” is avoiding the boundary conditions of the “Case C”. The main problem of the protocol proposed by ASTM E72 is that it follows “Case C” due
to the vertical constraining of the upper edge of the test specimen [GIRHAMMER ET AL. 2002]. In practice, “Case A” represents in general the behaviour of narrow elements and of elements loaded vertically only by flexible roof constructions. The “Case B” is typical for elements carrying floor constructions on top and “Case C” is the typical case of infill of a stiff surrounding frame.

**Case A**

**Case B**

**Case C**

*Figure 1: Three different boundary conditions of wall element tests which can be realised at UL FGG*
2.2 Test setup at University of Ljubljana

Following the experiences obtained from testing of masonry elements, a universal shear wall test set-up was developed and installed at Faculty for Civil and Geodetic Engineering of University of Ljubljana in 1999 [DUJIC 2001]. The main idea of the new device was to use a gravity load induced by ballast as a constant vertical load and a displacement controlled hydraulic actuator driving the cyclic horizontal load. The main challenge was to simulate realistic boundary conditions that may occur during the action of an earthquake. In reality, the boundary conditions may change during an earthquake excitation because of changes of the building characteristics due to development of damages. Therefore, the testing device should allow the altering of boundary conditions from one to another test run.

The realised test set-up, shown in Fig. 2, enables testing of elements hereby simulating the three above described cases of boundary conditions. The horizontal load is applied by successively inducing displacements along the free edge of the specimen. The specimens (4) are turned upside-down and supported along the upper edge by a steel frame structure to ease application of gravity load by ballast. The test set-up is composed of six major parts, marked in Figure 2 by numbers 1 to 6. The pair of lever beams (1) follows the vertical deformation of the specimen, while constant vertical load induced by counterbalance acting on the specimen. The horizontal displacement is applied along the lower horizontal edge of the specimen by a single displacement-controlled actuator (5) that moves the roller beam (3). The beam rolls along the supporting beam (2) that is hinged between the pair of lever beams. During the testing, the lower edge of the panel is supported by a hinged (2) and horizontally movable mechanism (3), which allows its free horizontal movement and rotation (boundary condition of the “Case A”). Rotation of the supporting beam (2) can be constrained by both vertical side and horizontal sliding supports (6) allowing exclusively its vertical translation. The sliding supports enable the simulation of the boundary conditions of the “Case B”. Further alternation of the setup by blocking of the movement of the supporting beam in one direction (7) gives the boundary conditions of the “Case C”.

The set-up is calibrated for vertical and horizontal load. Strains measurements at the upper flanges of the lever beams in the cross-section above the lever support enable the control of the vertical load acting on the tested specimen. The horizontal action of the hydraulic actuator is controlled by a data ac-
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quisition and actuator control system by Röell/Amsler. The capacity of the test set up is 500kN of constant vertical load and 250kN of horizontal load in a displacement range of ± 200mm.

Figure 2: Longitudinal cross-section of set-up for testing of wall elements and test set-up in laboratory of FGG at University of Ljubljana.

3. TESTING PROCEDURES AND WALL ELEMENT RESPONSES

Well known critics of ASTM E 72 e.g. by [GRIFFITHS 1984] show the importance of proper boundary conditions to be used for realistic testing of wall elements. The European test standard EN 594 represents a step forward in the improvement of the test procedure. However, it does not solve the problem of taking into account the above described realistic boundary conditions in a proper way. EN 594 prescribes a partial anchoring of the elements along the bottom rail what does not conform to many systems that are presently on the market. Further, bottom rail anchoring is not an appropriate solution in earthquake prone areas where anchoring of the studs results in higher earthquake resistance the entire building. Besides this, EN 594 does not cover loadings that may occur in earthquake prone areas because it takes into account only a monotonous load protocol.

The main goal of this paper is to demonstrate the variety of testing possibilities using the herein proposed test approach with adaptable boundary conditions and different loading protocols from simple monotonous [EN 594, ASTM 564-95] to more complex cyclic ones. Cyclic testing can be carried out following the EN 12512:2001 or ISO 16670:2003 protocols or any other protocol - for example the CUREE protocol [KRAWINKLER 1999].
Two different types of wooden walls were tested: timber-framed walls with OSB or gypsum fibre board sheathing and solid cross-laminated wooden walls. The test protocol included in both cases boundary conditions of Cases A and B, three levels of vertical load and two patterns of horizontal load: monotonous according to EN 594 and cyclic according to ATC-1994.

It is evident, that the response of the shear walls depend primarily on the configuration and mechanical properties of the constituent elements and the assembly as a whole. However, ignoring the influence of different boundary conditions and the level of vertical load may lead to misinterpretation of the observed response. In Figure 3 three different patterns of wall behaviour are presented: shear, rocking and combined shear – rocking response. All of them can develop under boundary conditions of shear cantilever mechanism (“Case A”). The behaviour depends on the shear stiffness of the wall diaphragm as a whole, the magnitude of vertical load and the layout and mechanical characteristics of the anchors.

The shear response develops either if the panel is flexible in shear or if the magnitude of the vertical force is relatively high. The rocking response is typical for weakly anchored stiff elements or a low level of vertical loading. Combined behaviour can be observed, in most real cases a combined shear-rocking response occurs depending on different combinations of panel stiffness, anchoring and vertical load.

The response of the elements tested with “Case A” boundary conditions represents the conservative behaviour. If the same panel is exposed to other boundary conditions (“Case B” or “Case C”) the response values of rocking and combined shear-rocking may be higher than the values observed using “Case A” conditions. The reason therefore is the decrease of the tensile forces developed in the vertical edges of the element, consequently lowering the tensile loading of the anchors. Testing under conditions of the “Case B” is justified only when the behaviour of the element in the real building is governed by an in and out of plane stiff floor diaphragm (composite wood-concrete or solid wood slab). Testing under conditions of “Case C” is suitable for elements designed to act as frame infill, elements with glued-in-rods or for highly vertically loaded walls in the lowest storey of multi-storey buildings.

Published results of testing under conditions of the “Case C” can not be considered applicable to most realistic cases and may lead to serious mistakes if
used in the design of structures. Due to underestimation of the importance of the boundary conditions the load bearing capacity of the elements is extremely overestimated especially when the elements are loaded with vertical loads of low intensity or when the elements are weakly anchored. However, at present the majority of known tests in Europe and hereon based expertise and technical approvals are based on the “Case C” conditions.

*Timber framed elements*

![Timber framed elements diagram](image)

*Solid wooden laminated elements*

![Solid wooden laminated elements diagram](image)

*Rocking response of walls*  
*Combined shear - rocking response of wall*  
*Shear response of walls*

*Figure 3: Typical responses of wooden wall elements exposed to combined vertical and horizontal load.*
4. INFLUENCE OF LOADING PROTOCOL AND OF VERTICAL LOAD

The complete information about the mechanical characteristics of wooden wall elements and their anchoring can be obtained from responses both to monotonous and cyclic loading with proper combination of vertical forces (Figure 5a). The protocol of EN 594 is sufficiently covering the monotonous loading. The protocol of EN 12512 covers cyclic testing of particular joints made with mechanical fasteners, what is an insufficient tool for evaluation of the behaviour factor “q” needed for design of earthquake resistant buildings. The ISO 16670 standard also addresses exclusively the joints but the proposed protocol can be used for testing of wooden wall diaphragms, too. The reason therefore is that ultimate joint displacement is used, instead of yield slip (EN 12512) which is difficult to define. Since the ISO protocol is based on ultimate displacement it can forward a behaviour factor “q” as addressed in Eurocode 8. It is obvious that there is a need for development of an integral European standard covering both monotonous and cyclic testing of wall diaphragms.

The comparison of the responses of different wooden elements (Figure 4) subjected to cyclic and monotonous loading well illustrates the importance of cycling testing. In case of the element presented in Fig. 4a, the load carrying capacity of the element exposed to cyclic loading was about 15% lower than the resistance of the element exposed to monotonous loading. The cyclic response shows higher initial stiffness due to hardening of the fasteners exposed to low-cycle fatigue and lower ductility down to 50% of the ductility reached in monotonous loading. Therefore, earthquake design of wooden buildings can not be properly performed without data obtained from cyclic testing of elements exposed to different intensities of vertical load.

In general, wall elements mostly exhibit higher load-bearing capacity and ductility when exposed to monotonous loading in comparison when exposed to cyclic loading. Low-cycle fatigue of the mechanical fasteners in the wooden elements loaded in-plane by cyclic loading leads to a reduction of the element load-bearing capacity from 10 to 20% in comparison to one observed during monotonous testing. On the other side, the behaviour of fasteners exposed to cyclic loading can also result in a slightly higher stiffness of the elements than in case of monotonous loading.

Further, the behaviour of the wall elements is strongly influenced by the density of the fasteners along the sheathing-to-wooden frame contact and by the
stiffness of the panel-to-foundation anchoring. When the fasteners are densely distributed and the anchoring is stiff, the cyclic response of the elements exhibits higher load-bearing capacity but ductility in comparison to the static response tends to decrease.

**Figure 4:** Configuration of one-side (a) and two-side (b) sheathed timber frame elements and solid wooden elements (c).

**Figure 5:** Comparison of monotonic and cyclic response of different timber framed wall elements of length of 2.44 m, built up according to Figs. 4a and 4b and loaded vertically with 20 kN/m and 30 kN/m, respectively.

In the case of shear wall elements acc. to Fig. 4 b with sheathing boards of gypsum fiber board a dense distribution of fasteners and the specific properties of the board material lead to non-ductile failure during monotonous loading, while the same type of specimen behaves ductile during cycling loading (Fig. 5 b) without losing much of its load-bearing capacity and stiffness.
The graphs in Figure 5 reveal the influence of vertical load intensity both on the load carrying capacity and the type of response mechanisms. In the case of timber frame elements (Figure 4a) the rocking mechanism was observed at the lowest magnitude of vertical load and the shear mechanism at the highest magnitude of vertical load. The boundary conditions were of the “Case A” at all vertical load intensities. In the case of low vertical load, the anchorage system increases the racking resistance of the wall. Fully anchored framed wall elements having tie-downs at the leading stud have higher lateral resistance and load carrying capacity than partially anchored walls. At magnitudes of total vertical load above 50 kN (20 kN per meter length of the wall) the anchorage system did not significantly influence the lateral resistance of the shear wall anymore. In this case the shear mechanism was fully developed. Contrary, in case of the much stiffer solid wooden elements (Figure 4c) the shear mechanism did not develop in spite of changing the boundary conditions from “Case A” to “Case B”. The shear mechanism was finally obtained when the boundary conditions were set to the “Case C”.

Figure 6: Influence of vertical load intensity on load carrying capacity of wooden wall elements 2.44 m long.

a) Timber framed elements – configuration acc. to Fig. 4a
b) Solid wooden elements – configuration acc. to Fig. 4c
5. CONCLUDING REMARKS

The importance of a proper taking into account of the boundary conditions and of the influence of vertical load and of the type of horizontal loading is evident from comparison of test results with different types of wooden shear wall elements. The clear differences between monotonic and cyclic response, strongly influenced by the type of element build-up reveal the need for further development of standard protocols for wooden wall diaphragms used for structures located in earthquake prone areas. The type of sheathing material is very important in this context. New standards should implement the concept of performance based earthquake engineering design to obtain experimental data needed for evaluation of the behaviour factor “q”.

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REFLECTION MEASUREMENTS AT TIMBER GLUE-LINES BY MEANS OF ULTRASOUND SHEAR WAVES

RELEXIONSMESSUNGEN AN HOLZVERKLEBUNGEN MITTELS ULTRASCHALL-SCHWERWELLEN

MESURES DES REFLEXIONS AUX PLANS DE COLLAGE DU BOIS AU MOYEN D'ONDES DE CISAILLEMENT

Gerhard Dill-Langer, Simon Aicher, Wolfgang Bernauer

SUMMARY

In the frame of an on-going research project the feasibility of ultrasound shear wave reflection measurements at glued laminated timber (glulam) has been studied. In the applied experimental set-up the propagation direction of the ultrasound waves was parallel to the smallest dimension of the glulam beam, being the width direction perpendicular to fiber and parallel to the board edges. In particular the presented work should answer two questions:

- Is it possible (within the frame of the used equipment and boundary conditions) to identify clear back wall reflection signals from a glued laminated beam in structural dimensions ?
- Can the reflection method be utilised for detection of boundary layers in defects of a secondary glue-line connecting two glulam blocks ?

In the paper the experimental conception and some preliminary results are presented.

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Im Rahmen eines laufenden Forschungsprojektes wurde die Machbarkeit von Reflexionsmessungen an Brettschichtholzbauten mittels Ultraschall-Scherwellen untersucht. Beim verwendeten Versuchsaufbau wurden die Ultraschallwellen parallel zur Richtung der kleinsten Trägerabmessungen eingeschichtrahlt, d.h. rechtwinklig zur Faserrichtung und parallel zu den Lamellenkanten.
Im Einzelnen sollte die vorgestellte Untersuchung folgende Fragen beantworten:

- Ist es (im Rahmen der zur Verfügung stehenden Geräteausstattung und der gegebenen Randbedingungen) möglich, ein klares Rückwand-Echo an Brettschichtholzbauteilen mit baupraktischen Abmessungen nachzuweisen?

- Kann die Reflexions-Methode zur Erkennung von Grenzflächen verwendet werden, wie sie im Bereich von Fehlverklebungen bei Verklebungsfugen von blockverklebten Brettschichtholz-Bauteilen auftreten?

In dem Aufsatz werden der experimentelle Ansatz und einige vorläufige Ergebnisse dargestellt.

RESUME

Dans le cadre d'un projet de recherche en cours, une étude de faisabilité sur les mesures des réflexions des ondes ultrasonores de cisaillement dans les éléments en bois lamellé-collé a été réalisée. Dans le dispositif expérimental utilisé, la direction de propagation des ultrasons était parallèle à la plus petite dimension de la poutre, c.-à-d. perpendiculaire au fil du bois et parallèle aux arêtes des lamelles.

En particulier, l'étude vise à répondre aux questions suivantes:

- Est-il possible (dans le cadre de l'équipement disponible et des conditions aux limites) d'identifier clairement un signal réfléchi par la face arrière d'une poutre en bois lamellé-collé de dimensions usuelles?

- La méthode des réflexions peut-elle être utilisée pour détecter les surfaces de séparation, comme celles qui apparaissent à l'aboutage de deux blocs de lamellé-collé?

Cet article présente l'approche expérimentale et les premiers résultats.

KEYWORDS: Non-destructive testing, ultrasound, glued laminated timber, wood, shear waves, reflection measurements
1. INTRODUCTION

One of the important factors for the serviceability and safety of structures made of glued timber products consists in the integrity and load bearing capacity of the incorporated glue-lines. In order to develop non-destructive test methods for characterisation of the glue-line performance an applied research project on "Non-destructive detection of glue-line failures in glued timber products" has been started at MPA Otto-Graf-Institute.

In the preparation and within the frame of the on-going project several test and evaluation methods based on transmission measurements by means of longitudinal waves have been applied successfully, see for example [AICHER ET AL. 2002, AICHER ET AL. 2004, DILL-LANGER ET AL. 2005]. The methods using longitudinal waves are especially successful in the cases of a some finite gap (> 0,1 mm) in the area of the glue-line defect, representing for example larger air bubbles within a glue-line performed by pressure free gluing with epoxy adhesives. In those cases the advantages of longitudinal waves (for example simple coupling conditions, high input energy) overbalance the shortcomings (for example the lower sensitivity to boundary layers).

However, the transmission method exhibits some drawbacks, some of them being of practical nature, for example the need of both-sided access to the structural element. Moreover, the contrast between sound glue-lines and defects achievable from the data of transmission measurements always consists in a (small) difference of (large) numbers like differences in amplitude or time-of-flight, thereby being highly susceptible for unfavourable scatter.

In the case of very thin glue-lines (<< 0,1mm) performed by means of usual gluing pressures of about 0,6 to 1,0 N/mm², there is virtually no gap between the two assembly components, even within defect areas with missing glue. Thus, the compression part of longitudinal waves is transferred equally for sound glue-lines and for glue line defects and the US contrast for longitudinal waves with low frequency and thus large wave length is nearly vanishing.

Therefore, in the presented study shear waves in combination with the reflection method have been used for detection of defects in a thin glue-line. As an example, the secondary glue-line of a "block glued" member consisting of two glulam beam segments has been inspected. The shear wave reflection method has already be used successfully for detection of wood defects such as decay,
see [HASENSTAB AND KRAUSE 2005]. However, to the knowledge of the authors, the application to glue-line defect detection has not yet been studied before.

2. METHODS AND TEST CONFIGURATION

Before utilizing the shear wave reflection method for the inspection of glue-line defects, it has to be proven, that clear back wall reflection signals can be measured in the frame of the given boundary conditions (typical structural dimensions and usual quality of the glued laminated timber, available equipment, etc.). For this purpose a pair (transmitter / receiver) of US shear wave transducers has been applied to the planed surface of a glulam beam with the dimensions height×width×length = 600mm×114mm×1180mm.

The broad band transducers (PANAMETRICS V150) with a central frequency of 250 kHz and a nominal element size of 25 mm have been driven by a high voltage pulse generator (PANAMETRICS 5058 PR).

The propagation direction of the shear waves was mainly perpendicular to the grain in width direction of the beam. The test configuration is sketched in Fig. 1. The receiver was shifted from the transmitter by 100 mm in length direction (fiber direction). It turned out, that the best results were achieved with polarisation direction of the shear waves parallel to length direction (fiber direction). No coupling agent has been used (dry coupling) and the transducers have been pressed to the timber surfaces with a given and controlled pressure force of 1000N. The signals have been amplified by a broad band amplifier and recorded by a transient recorder (12 bit amplitude resolution, 10 MHz sampling rate).

Fig. 2 shows a typical signal, whereby two different features are clearly separated: a first peak (within about 10 to 20 µs) resulting from a US wave propagating directly from the transmitter to the transducer near the surface of the timber beam. For sake of simplicity this part of the signal is named "surface wave", although from the measured US velocity it is assumed, that this wave component is a longitudinal wave propagating next to the surface but without the characteristics of the much slower surface wave. The occurrence of longitudinal waves in spite of nominally "pure" shear wave input results from the limited efficiency of the shear wave transducer on the one hand and from the mode conversion effects in the highly anisotropic material timber on the other hand.

Clearly separated from the first peak a much smaller second peak is observed throughout occurring within 160 to 200 µs travel time. From literature
shear wave velocity values in the range of 1320 to 1372 m/s [Bucur, 1995] and the thickness of 114 mm this peak is clearly identified as the back wall echo of the glulam beam. In the range of 360 µs also a very small second back wall echo resulting from double reflection can be found. Although Fig. 2 shows one of the signals with relatively large signal to noise ratio the feasibility of back wall identification could be shown with all performed measurements.

The test specimen for detection of glue-line defects has been manufactured by two stages of gluing:
1. First the timber lamellas are bonded together resulting in a usual glulam cross-section (primary gluing).

2. Two glulam cross-sections are then connected by a secondary gluing process, whereby the two components of the assembly are bonded flat-wise resulting in a so called "block glued" member. The resulting cross-section is depicted in Fig. 3.

In the following only the secondary or "block" glue-line will be inspected by means of non-destructive testing. The primary glue-lines have been produced by usual industrial means and exhibit virtually no defects. For sake of simplicity the term "glue-line" in the following always refers to the secondary glue-line.

The secondary gluing has been performed by means of a PRF adhesive at a gluing pressure of 0,8 N/mm$^2$ in a hydraulic veneer pressing device. The faces of the two glulam components have been planed directly before gluing. The resulting glue-line thickness was smaller than 0,1 mm. In the center of the interface no adhesive has been applied at an area of 400 mm length and 300 mm width. At the edge of the artificial glue-line defect a sealing inside a millcut of 1mm depth and 14 mm width prevented penetration of adhesive to the defined defect area.

The lay-up of the block-glued specimen resulted in a sound glue-line with a defined glue-line defect. Due to the surface quality and the applied gluing pressure there is virtually no gap between the two glulam components in the area of the defect.

Fig. 3: Lay-up and dimensions of the glulam specimen with secondary "block" glue-line
The test configuration for the reflection measurements are analogous to the echo measurements of the single glulam segment described above: A pair of shear wave transducers is pressed to one surface of the block-glued member. Ultrasound shear waves are applied by the transmitter and the resulting signal is measured by the receiver and recorded by a amplifier / transient recorder system. The coupling conditions, the orientation and distance of the two transducers, the polarisation direction and the data of the US equipment conform to those mentioned above. The whole specimen surface (apart from some centimeters at the edges) has been scanned by shifting the transmitter / receiver pair of transducers within a grid of 50 mm in fiber direction and 32 mm in direction perpendicular thereto. In total \( 21 \times 17 = 357 \) signals have been recorded. The test configuration is sketched schematically in Fig. 4.

\[ \text{Fig. 4: Test configuration for glue-line defect detection by means of reflection measurements} \]

First preliminary results and evaluations are given below.

3. PRELIMINARY RESULTS

Figure 5 shows a typical signal recorded at a location within the section of sound glue-line. The comparison with the signal recorded at the single glulam cross-section (Fig. 2) yields some similarities and some deviations:

- The maximum peak resulting from direct wave propagation between transmitter and receiver is found in the same time domain as for the signal obtained in case of the single glulam cross-section.

- In the time domain corresponding to the location of the glue-line the amplitudes are quite low compared to the case of a single glulam cross-section.
- Some small, but reproducible amplitudes are observed in the time domain which corresponds to the back wall reflection of the composite block-glued cross-section.

![Image of a typical US signal (A-scan) of a glulam block with one secondary glue-line perpendicular to US propagation direction (for test configuration see Figs. 3 and 4): signal recorded in a section with sound glue-line.]

**Fig. 5:** Typical US signal (A-scan) of a glulam block with one secondary glue-line perpendicular to US propagation direction (for test configuration see Figs. 3 and 4): signal recorded in a section with sound glue-line.

Figure 6 shows a typical signal recorded at a location within the section of the glue-line defect (missing adhesive). The following features can be stated:

- Analogously to the single glulam cross-section and to the compound cross-section with sound glue-line the maximum peak from direct "surface" wave propagation is visible at the beginning of the signal.

- In the time domain corresponding to the location of the (adhesive free) boundary between the two glulam blocks a clear echo signal with significantly higher amplitude as compared to the case of sound glue-line is observed.

- Some small amplitudes are visible in the time domain corresponding to the location of the back wall reflection of the composite block-glued cross-section. This feature is comparable to the case of sound glue-line.
Reflection measurements at timber glue-lines by means of ultrasound shear waves

Fig. 6: Typical US signal (A-scan) of a glulam block with one secondary glue-line perpendicular to US propagation direction (for test configuration see Figs. 3 and 4): signal recorded in a section with glue-line defect (missing glue)

From the presented two A-scans it can be assumed that one promising parameter for a C-scan evaluation is the maximum-minimum difference amplitude (MMD) in a time window corresponding to the location of the glue-line, i.e. MMD_{glue-line}. In order to reduce scatter due to the coupling influence or due to different US attenuation of different boards the MMD_{glue-line} is normalised with respect to the "surface" wave amplitude MMD_{surface}, in all cases being identical to the global MMD of the signal.

In Fig. 7b the results of the reflection measurements are given as a 3dimensional C-scan representation, i.e. the parameter normalised reflection amplitude = MMD_{glue-line} / MMD_{surface} is plotted vs. the location (co-ordinates parallel and perpendicular to fiber direction) of the measurements. For better comparison the location and the dimensions of the glue-line defects are depicted in Fig. 7a in a graphical similar representation, whereby the z-axis has only the meaning to differentiate between sound and defect glue-line sections.

The normalised reflection amplitudes show the maximum values within the area of the glue-line defect. The minimum values are found in the section of sound glue-line. Although there is a high amount of scatter within the defect
area, a clear contrast between the sections of sound and defect glue-lines can be observed.

Fig. 7: Results of reflection measurements at a secondary glulam glue-line in C-scan representation:
   a) Location and dimensions of the glue-line defect
   b) Normalised reflection amplitude ($MMD_{\text{reflec}} / MMD_{\text{surface}}$)
4. CONCLUSIONS

The reported study on reflection measurements at structural sized glue-laminated timber beams shows

- the feasibility of back wall reflection measurements on glulam with a thickness of 114 mm
- the potential of the reflection method to detect glue-line defects in block members with a depth of 228 mm and one secondary glue-line, even if the glue-lines are thin (< 0.1 mm) and high gluing pressure has been applied.

The presented results are part of an on-going research project and have to be verified with more specimens and different specimen configurations concerning dimensions and timber quality. One of the problems consists in the relatively high amount of scatter which makes it difficult to establish a clear threshold value to differentiate between sound and defect glue-lines. In order to further reduce scatter it may be necessary to simultaneously measure additional non-destructive parameters for advanced normalization of the reflection data.

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JOINTS IN TIMBER STRUCTURES LOADED PERPENDICULAR TO GRAIN – COMPARISON OF DESIGN APPROACHES

QUERANSCHLÜSSE IM HOLZBAU – EIN VERGLEICH VON BEMESESUNGSANSÄTZEN

ASSEMBLAGES DES STRUCTURES EN BOIS CHARGEES PERPENDICULAIREMENT AU FIL – UNE COMPARAISON DES APPROCHES DE DIMENSIONNEMENT

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SUMMARY

The paper reports on the design of joints in timber beams loaded perpendicular to grain. Such joints are highly prone to splitting and in most cases the load-carrying capacity of the joint depends on the tension resistance perpendicular to grain rather than on the lateral load capacity of the dowel type fastener. The models developed for design of such joints with respect to failure in tension perpendicular to grain can be grouped in strength of materials approaches and linear fracture mechanics models. A design based on tension strength perpendicular to grain is implemented in the new version of DIN 1052:2004 being very close to the quasi-standardised design procedure in the frame of the still valid “old” German timber design code based on permissible stresses. Contrary, Eurocode 5 contains a fracture mechanics based model.

In the paper presented, a specific joint type with a single dowel-fixed steel plate in a glulam beam is considered and the design capacities according to the different code rules are evaluated. As parameters, the numbers and rows of fasteners are varied. Configurations with large effective joint heights, i.e. with a large distance of the loaded beam edge to the innermost dowel, are of special interest. Despite rather good agreement for smaller effective heights some model dependent discrepancies occur for larger effective height to beam height ratios.

ZUSAMMENFASSUNG

In dem vorliegenden Aufsatz wird über die Bemessung von Queranschlüssen an Holzträger berichtet. Solche Verbindungen sind sehr spaltempfindlich und in den überwiegenden Fällen wird die Tragfähigkeit des Anschlusses durch die Zugbeanspruchbarkeit rechtwinklig zur Faserrichtung und nicht durch die


RESUME

Cet article rend compte du dimensionnement des assemblages des poutres en bois chargés perpendiculairement au fil du bois. De tels assemblages sont très susceptibles au fendage et la capacité portante est dans la plupart des cas déterminée par la résistance en traction perpendiculaire au fil, et non pas par la résistance au cisaillement des chevilles. Les méthodes de dimensionnement pour la sollicitation en cisaillement perpendiculaire au fil pour ces assemblages peuvent être classés en deux groupes: celui des procédures utilisant un critère de résistance des matériaux et celui des modèles basés sur la mécanique de la rupture. La démarche correspond largement à la procédure de dimensionnement "quasi normalisée" dans le cadre de l""ancienne" norme allemande sur les constructions en bois, laquelle est basée sur les contraintes admissibles. L'Eurocode 5, par contre, contient un modèle basé sur la mécanique de la rupture.

Dans cet article est présenté un type d'assemblage avec une plaque d'acier logée dans une rainure de la poutre en bois lamellé collé et fixée par des chevil-
les; les capacités de charge sont déterminées selon les différentes normes. Les paramètres variables sont le nombre et les rangées des éléments de fixation. Les configurations avec une grande hauteur utile, c.-à-d. un grand écart entre le bord sollicité de la poutre et la cheville la plus éloignée sont d’un intérêt particulier. Une bonne concordance est observée pour les petites hauteurs utiles. En revanche, pour un rapport élevé de la hauteur utile à la hauteur totale de la poutre, des divergences parfois importantes se manifestent en fonction du modèle de dimensionnement.

KEYWORDS: Perpendicular-to-grain-joints, comparison of timber design codes, effective joint height, failure mode transition, dowel type fasteners

1 INTRODUCTION

Joints transferring loads perpendicular to grain are employed in all cases where loads are hung at the bottom side of (glued laminated) timber beams and at connections of secondary beams to main beams. At so-called perpendicular-to-grain-joints (PGJ), tensile stresses perpendicular to grain direction occur in the main girder. Consequently, the load-carrying capacity of lateral joints depends not only on the resistance of the fasteners and the connection construction but also considerably on the transverse material resistance against splitting.

Types of PGJ are joist hangers, T-beam-connections, tenon type joints and modern front side connectors with two complementary attaching plates, e.g. fit connectors. Figure 1a shows a joist hanger. The T-beam-connections are hidden inside the wood (Fig. 1b). Their advantage are the good fire protection characteristics, as the steel is protected by the wood on all sides, and their good appearance. By use of doweled connections with a steel plate slit in the timber, loads can be hung on the bottom side of glulam beams (Fig. 4). This type of connection can also be used when fire protection is required.
Figure 1: Examples of perpendicular-to-grain-joints (see also Figs. 2 and 4)

a) Joist hanger
b) T-beam-connection

The design of PGJ has been subject to intensive research in the past. Amongst others the main topic is related to the question whether a classical strength of materials approach or a fracture mechanics based design are more appropriate for prediction of the joint load capacity. Increasingly a fracture mechanics related failure mechanism is considered to apply, however, in several of today’s design codes the strength of material approach is implemented.

The paper presents a comparison of three different joint design rules. In detail, the joint design according to the new German timber design code, based on ultimate states, the permissible stress design approach used in Germany so far (DIN 1052:1988) and the Eurocode 5 approach are regarded. The comparison of the design is performed exemplarily for a dowel type joint with a steel plate mounted in a central slit. Some consideration is given to configurations with high ratios of effective joint height, i.e. for large distances of the loaded beam edge to the innermost dowel and to several dowel rows.

2 JOINT DESIGN GIVEN IN DIN 1052:2004

The load-carrying capacity and hence the design depends pronouncedly on the ratio of effective joint height to beam height $a/h$ (Fig. 2). Below a ratio of $a/h = 0.2$, perpendicular-to-grain-joints shall only be used for short time loads, e.g. wind loads. Further, the standard specifies that the verification of the transverse tensile stresses can be omitted for effective joint heights $a > 0.7 \, h$. In this
case the joint can be designed on the basis of the lateral load capacity of the fas-
teners and, not explicitly stated, with respect to the beam shear force capacity. For \(0.2 \leq a/h \leq 0.7\), the design equation and the design resistance \(R_{90,d}\) of the PGJ is

\[
\frac{F_{90,d}}{R_{90,d}} \leq 1
\]

\[
R_{90,d} = k_s \cdot k_r \cdot \left(6.5 + \frac{18 \cdot a^2}{h^2}\right) \cdot (t_{ef} \cdot h)^{0.8} \cdot f_{t,90,d}
\]

where

\[
k_s = \max\left\{1; 0.7 + \frac{1.4 \cdot a_r}{h}\right\}
\]

and

\[
k_r = \frac{n}{\sum_{i=1}^{n} h_i^2}
\]

In above equations, the following definitions apply:

- \(n\) number of fasteners parallel to the applied load
- \(F_{90,d}\) design tension force acting on the joint (see Fig. 2)
- \(t_{ef}\) effective fastener depth depending on type of fastener and jointed material combination (e.g. wood-based boards jointed by nails or screws to main glulam beam)
- \(t_{ef} = \min\{b; t; 6d\}\) effective fastener depth for one sided dowel connection
- \(f_{t,90,d} = f_{t,90,k} \cdot k_{mod} / \gamma_M\) design value and characteristic value of tension strength perpendicular to grain
- \(\gamma_M\) partial safety factor for the material property (\(\gamma_M = 1.3\))
- \(k_{mod}\) modification factor accounting for the climate service class and the duration of load
Figure 2: Example of a two-sided perpendicular-to-grain-joint and dimensions according to DIN 1052:2004

Adjacent joints (groups of fasteners) can be considered as independent joints, provided that the clear distance between the adjacent groups is larger than or equal to \(2h\). In case the clear distance is smaller than \(2h\) but larger/equal to \(0.5h\), the design load-carrying capacity has to be reduced by the coefficient

\[
k_g = \frac{l_g}{4h} + 0.5
\]  

(4)

where \(l_g\) is the clear distance between lateral joints. In the case of a clear distance \(l_g < 0.5h\), the adjacent joints have to be considered as a single joint.

3 DESIGN WITHIN THE FRAME OF DIN 1052:1988

The design of perpendicular-to-grain-joints is not regulated explicitly in the still valid “old” German timber design code DIN 1052:1988 based on the permissible stress concept. However, a design concept based on this standard is given in [2] and presently state of the art of such joints in Germany. The cited approach
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based on work by Ehlbeck et. al. [11] actually formed the basis for the above presented limit state design in DIN 1052:2004. The influence of the ratio of joint height to beam height $a/h$ is considered by the factor $f_1$ within the same $a/h$-boundaries as above ($0.2 \leq a/h \leq 0.7$):

$$f_1 = \frac{1}{1 - 3 \cdot \left(\frac{a}{h}\right)^2 + 2 \cdot \left(\frac{a}{h}\right)^3}$$

(5)

The influence of several fastener rows is taken into account by factor $f_2$ which fully corresponds to $k_r$ given in Eq. (3). The interaction of adjacent joints with a clear distance of less than $2h$ is considered by the factor

$$f_3 = 1 + \frac{W_m}{W_m + a}$$

(6)

where $W_m$ is the centroidal distance of adjacent joints. The type of fastener is taken into account by factor $f_4$ (e.g. 1.0 for dowel type fasteners). The effective joint width $ef \, W$ is

$$ef \, W = \sqrt{a_r + (Ch)^2}$$

for horizontal distance between adjacent fasteners less than $(0.8h - a)$

$$ef \, W = C \cdot h \cdot \left[1 + (m-1) \cdot \frac{a_r}{a_r - a}\right]$$

else

(7)

where

$$C = \frac{4}{3} \cdot \sqrt{\frac{a}{h} \cdot \left(1 - \frac{a}{h}\right)^3}$$

(8)

The effective joint area is calculated as

$$ef \, A = ef \, W \cdot t_{ef}$$

(9)

using the effective fastener depth $t_{ef}$ as before. The permissible tension stress perpendicular to grain depends on the area stressed perpendicular to the grain direction

$$zul \, \sigma_{t,90} = 0.333 \cdot (ef \, A \cdot 10^2)^{-0.2}$$

for glulam

$$zul \, \sigma_{t,90} = 0.200 \cdot (ef \, A \cdot 10^2)^{-0.2}$$

for solid wood

(10)

The allowable force of the perpendicular-to-grain-joint is finally evaluated as

$$zul \, F_{t,90} = zul \, \sigma_{t,90} \cdot ef \, A \cdot f_1 \cdot f_2 \cdot f_3 \cdot f_4$$

(11)
4 DESIGN ACCORDING TO EUROCODE 5

The Eurocode 5 (EN 1995-1-1:2004 [4]) design of (dowel) joints loaded perpendicular to grain is based on the linear fracture mechanics approaches introduced by van der Put [7], van der Put and Leijten [8], Leijten and Jorissen [9], and Larsen and Gustafsson [10, 14]. The principle load capacity equation closest to the below given Eurocode 5 design is (Leijten and Jorissen [9])

\[ V_u = 0.4b \sqrt{\frac{a}{G G c, I}} \sqrt{\frac{1}{1 - (a/h)}} \] \hspace{1cm} (12)

It can be seen that the joint load capacity, unlike the DIN approaches depends on the square root of \( a \) and \( a/h \), representing the fundamental size relationship of linear fracture mechanics. Further, the material resistance is no more defined by tension strength perpendicular to grain but by mode I fracture energy in tension perpendicular to grain \( G_{c,I} \) and shear stiffness \( G \) of the timber. The apparent fracture parameter \( \sqrt{G G_{c,I}} \) depends considerably on the type of the connection. Calibration to test results forwarded for the mean values a range of \( 9.3 \leq \sqrt{G G_{c,I}} \) \( [\text{N/mm}^{1.5}] \leq 34 \) (Leijten and Jorissen [9]). Based on Eq. (12) and the mentioned apparent fracture parameter fitting the characteristic load-carrying capacity perpendicular to grain given in Eurocode 5 is for a joint with several fasteners in one row (except for nail plates)

\[ F_{90,Rk} = 14b \sqrt{\frac{a}{1 - \frac{a}{h}}} \] \hspace{1cm} (13)

The design has to verify the condition

\[ \frac{F_{v,Ed}}{F_{90,Rd}} \leq 1 \] \hspace{1cm} (14)

where

\[ F_{v,Ed} = \max \left\{ F_{v,Ed,1}, F_{v,Ed,2} \right\} \]

\[ F_{90,Rd} = k_{mod} \cdot \frac{F_{90,Rk}}{\gamma_M} \]

design values of shear forces on either side of the connection resulting from \( F_{Ed} \cdot \sin \alpha \) (see Fig. 3)

\[ \text{design load-carrying capacity perp. to grain} \]
Contrary to the DIN approaches no explicit rules are given how to handle the design in case of several parallel rows of fasteners.

![Figure 3: Connection with force at an angle to the grain and dimensions acc. to Eurocode 5](image)

5 CASE STUDY

In the frame of the conducted parameter study, a dowel type joint with a steel plate mounted in a central slit of the timber beam is regarded exemplarily (Fig. 4). The distance of the fasteners closest to the bottom edge is kept constant: \( h - h_n = 140 \) mm. The parameter variations comprised the joint height to beam height ratio \( a/h \) and the number \( n \) of dowel rows. A range beginning with \( n = 2 \) and \( a/h = 0.22 \) and increasing to \( n = 10 \) and \( a/h = 0.86 \) was investigated. Further, the number of fastener columns is varied; hereby one and three parallel columns are regarded. The required minimum distances of the dowels according to DIN 1052:2004, chapter 12.3, table 8 were fulfilled. The cross section of the glulam beam is 240 mm × 1000 mm. The glulam strength class is GL28h (characteristic density: 410 kg/m³). The dowel diameter and the thickness of the steel plate were 20 mm and 16 mm, respectively. A view of the joint layout for the case of \( n = 4 \) is given in Fig. 4.
First, the results for the joint comprising one column of fasteners are discussed. The results are presented graphically in Figure 5 showing the sustainable characteristic loads depending on the ratio of effective joint height to beam height. With respect to determination of the given values, the following details are noteworthy. In case of DIN 1052:2004 the sustainable characteristic load \( R_{90,d}/\gamma_L = R_{90,k}/\gamma_G \) is given which equals in principle the permissible load of the “old” design concept. (Note: \( \gamma_L \) is the partial safety factor for actions, \( \gamma_G = \gamma_L \cdot \gamma_M / k_{\text{mod}} \) is global safety factor.) In the discussed example, the partial safety factors were chosen as \( \gamma_L = 1.425 \) and \( \gamma_M = 1.3 \) for tension perpendicular to grain resistance and shear load capacity and \( \gamma_M = 1.2 \) for fastener load capacity; \( k_{\text{mod}} \) was taken as 0.8 standing for medium duration of load in service classes 1 and 2. The characteristic tension strength perpendicular to grain is \( f_{t,90,k} = 0.5 \) N/mm².

For DIN 1052:1988 zur \( F_{t,90} \) as specified in chapter 3 is depicted.

In case of the Eurocode 5 design, specified by Eqs. (13) and (14) the maximum characteristic tension perpendicular to grain resistance is reached in the case of a design load \( F_{v,Ed} = F_{v,Ed,1} = F_{v,Ed,2} = F_{Ed} / 2 \), reflecting the situation where the load (for instance a secondary beam connected to the main beam) is applied at midspan. This yields a maximum design tension perpendicular to grain resistance of \( 2 F_{90,Rd} / \gamma_L \).
It can be seen from Fig. 5 that all three approaches deliver comparable sustainable characteristic or permissible loads. Despite that, the quantitative differences are in parts not negligible. For instance for a a/h-ratio of 0.3 the results for DIN 1052:1988 and Eurocode 5 are 122% and 141% of the solution acc. to DIN 1052:2004, respectively.

All design approaches deliver a steady load capacity evolution with increasing effective joint height. This is trivial in case of the Eurocode 5 solution where one equation for a single failure mode applies. In case of the DIN solutions the change of failure mode from tension perpendicular to grain capacity to lateral fastener capacity is astonishingly smooth. This however, is dependant on the specific joint configuration, as shown below.

Following, the results for the joint comprising three columns of fasteners are discussed. The results are presented graphically in Figure 6. As Eurocode 5 does not contain explicit rules on how to handle multiple fastener columns, exclusively the DIN solutions are regarded. At first, the DIN 1052:2004 results are discussed.
In the range of $0.2 \leq \frac{a}{h} \leq 0.7$ the load-carrying capacity, depending exclusively on the tension perpendicular to grain resistance, increases progressively in the form of a power type function. Beyond an $\frac{a}{h}$–ratio of 0.7, the tension perpendicular to grain resistance can be disregarded according to the DIN design concepts in [1] and [2]; then the load capacity is determined either by the lateral load-carrying capacity of the dowel type fasteners or by the bulk shear force capacity of the beam. Resulting from this, a point of discontinuity is located at $\frac{a}{h} = 0.7$. The sustainable characteristic load capacity curve for DIN 1052:2004 reveals a step increase by a factor of 2.7. This means that a small increase of the $\frac{a}{h}$–ratio by a minor shift of the fasteners towards a larger effective joint height can raise the design load-carrying capacity significantly. Above $\frac{a}{h} = 0.7$ the joint resistance is initially limited by the fastener resistance and then by the shear load capacity ($2V_{Rd}/\gamma_G = 2 \cdot (2/3) \cdot h \cdot b \cdot f_{v,k} \cdot k_{mod}/\gamma_G$) and $f_{v,k} = 3.5$ N/mm²).

A load-carrying capacity characterised by a step function with an extreme discontinuity of a factor of 2.7 does not seem physically reasonable. Even in
tests [5] with a very high $a/h$–ratio of 0,75 a transverse tension crack occurred at the upper fastener row. Thus, irrespective of the exact $a/h$–ratio it is necessary to account for the transverse tensile stress also in the very high $a/h$–range. By introducing a simple interaction formula for transverse tensile stress and shear stress, it is possible to smooth the step function considerably.

In the permissible stress design concept [2], the curve of $zul F_{t,90}$ depending on the $a/h$–ratio resembles the load capacity evolution according to DIN 1052:2004. However, quantitatively significant differences can be stated, both below and beyond $a/h = 0,7$. For small and medium effective joint height ratios up to about $a/h = 0,55$ the permissible values are about 30% higher as compared to DIN 1052:2004. In the $a/h$–range of 0,6 to 0,7 the differences increase up to a factor of circa 1,6.

The step of the DIN 1052:1988 approach at $a/h = 0,7$ is considerably smaller as in case of DIN 1052:2004 being due to two reasons: i) the capacity based on tension strength perpendicular to grain ($a/h < 0,7$) is higher and ii) $2 \cdot zul V = 358$ kN is noticeably (20%) smaller than $2 \cdot V_{Rd}/\gamma_G = 452$ kN. Beyond $a/h = 0,7$ the load-carrying capacity is limited by the permissible shear force capacity. The continued power function and the horizontal line of the shear force resistance intersect shortly after the discontinuity. Thus, the unbalance of the tension perpendicular to grain and the shear capacities are restricted to a very small $a/h$–range and the design method according to [2] can be considered reasonable.

6 CONCLUSIONS

In some cases perpendicular-to-grain-joints with very high effective joint height ratios ($a/h$) can be demanded by the specific structural circumstances. In the design concept of DIN 1052:2004, a sharp transition in the failure mode is implemented going from the transverse tension failure to the load-carrying capacity of the fastener or to the bulk shear force capacity of the beam. This failure mode change at an $a/h$–ratio of 0,7 is associated with a discontinuous increase of the load-carrying capacity ranging up to a factor of about 3 depending on the joint configuration. Such a pronounced discontinuity in the load-carrying capacity is physically not reasonable. It is proposed to smooth the step function by an interaction equation for shear stress and tensile stress perpendicular to grain. A sec-
ond and simple alternative is to abolish the rather arbitrary failure mode transition at $a/h = 0.7$ and to determine the sustainable characteristic load of the joint as the smallest value resulting either from tension perpendicular to grain capacity, lateral fastener capacity or shear capacity. The fracture mechanics based Eurocode 5 approach for the joint load capacity is more appealing from view of the applied mechanical concept. The absolute load capacities are rather similar to those of the DIN strength of materials approach in case of the regarded joint with one fastener column. However, for multiple columns a plausible rule has to be implemented.

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