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This issue contains a large spectrum of topics. There are four papers on non-destructive testing of materials of various kinds. One paper deals with near-field effects in signal based acoustic emission analysis. Another tackles the problem of signal conditioning of acoustic emissions and ultrasound signals. A third paper describes the signal denoising by means of fuzzy logic methods for non-destructive testing of timber. A new development concerns structural health monitoring with wireless sensor networks which will get much attention in the future.

Europe has received new standards on concrete and concrete construction. One paper is a summary of the latest development in this area. Self-compacting concrete (SCC) has started in Japan about 20 years ago and it receives much attention in Germany now. One of the open questions is related to the fire behaviour of SCC. Another property of SCC which has not yet been investigated in detail is the leaching behaviour. Two papers deal with these subjects. Chemical attack on concrete is a problem which occurs regularly. Therefore a paper on sulfate resistance of concrete has been incorporated in this issue. Statistics is important when the strength values of wood have to be characterized. One paper describes the statistical determination of characteristic strength values for structural timber. Finally, material characterization and damage behaviour of cellulosis-fiber gypsum composites is discussed.

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-Schüle Foundation. The support and cooperation by these organizations and those companies and organizations not mentioned are gratefully acknowledged.
The list of contents of previous issues can be found in the internet:
http://www.mpa.uni-stuttgar.de/
DEVELOPMENT OF CONCRETE CONSTRUCTION IN EUROPE

EUROPÄISCHE ENTWICKLUNG DER BETONBAUWEISE

DEVELOPPEMENT EUROPEEN DE CONSTRUIRE EN BETON

Hans W. Reinhardt

KEYWORDS: concrete, construction, Europe, standardization, SCC, textile concrete, UHPC, environment

SUMMARY

Some aspects of the European development in concrete construction are highlighted. First, a standardization of concrete as a material in EN 206 is emphasized as a milestone. It took 25 years to finish such a standard covering all types of concrete, strength classes and durability classes. A great focus was laid on the durability in the sense of exposure classes. These are compared with a resistance in the same way as structural design is performed. Some developments are shown such as self-compacting concrete, ultra-high performance concrete and textile reinforced concrete. Finally, the environmental consciousness which has a long tradition in Europe is exemplified with blended cements and co-combustion for flyash production. The results are encouraging.

ZUSAMMENFASSUNG


RESUMÉ

Quelques aspects du développement Européen de construire en béton sont examinés. D’abord, la normalisation du béton comme matériau en EN 206 est soulignée comme une étape importante. Il a demandé 25 ans pour finir cette norme qui contient tout les types de béton, les classes de résistance et les classes de durabilité. La durabilité a reçu grande attention avec définir les classes d’exposition. Elles sont comparées avec la résistance en la même manier que la résistance mecanique est calculée. Quelques développements sont illustrés tell que le béton autocompactant, le béton à très haute performance et le béton renforcé avec des textiles. Finalement, le respect de l’environnement qui a une longe tradition en Europe est illustré avec des ciments composites et la co-combustion pour la fabrication des cendres volantes. Les résultats sont encourageants.

INTRODUCTION

Europe has an old tradition in concrete construction. The first attempts go back to the 40’s of the Nineteenth Century when Joseph Monier made his flower pots of a kind of ferrocement. Joseph Louis Lambot built small ships, one of them still being available.

Fig. 1. Lambot’s ship of 1848 made of concrete [1]
They started with ferrocement which was and is still a very versatile material which can be shaped by hand. Since then, reinforced concrete developed steadily and in the early Twentieth Century prestressed concrete was developed. It is well known that concrete is the most popular construction material which is used everywhere. The total consumption in Europe per year amounts to 800 mill. tons.

Europe is growing together. This is also true for the technical goals. The European law enforces everybody not to build barriers to trade. It took more than 20 years to finalize a European Standard on concrete. The EN 206 part 1 is now ready and is introduced in all European countries. On the other hand, there is a continuous development of concrete construction. Three types of concrete are being developed and used more frequently, i.e. self-compacting concrete (SCC), ultra-high performance concrete (UHPC) and textile reinforced concrete (TRC).

In Europe there is a strong environmental movement. This means that the industry in Europe tries to reduce the impact on the environment. This is achieved through various channels. A few examples are recycling of concrete, co-combustion of coal in power plants, re-use of fresh concrete and waiste water. Sustainability of concrete construction is an important aspect and has a rather long tradition. All these aspects will get due attention in the following.

**STANDARDIZATION**

The new standard EN 206-1 [2] deals with the specification, performance, production and conformity. It is the materials standard which is needed in conjunction with the Eurocode. The Eurocode is the design code of concrete structures. EN 206-1 deals with the specification of the material. There the concrete classes are specified as strength, workability and density is concerned. The code is valid for all types of concrete, i.e. normal weight concrete, lightweight concrete and heavy weight concrete. The density classes are lightweight between 800 and 2000 kg/m$^3$, normal weight between 2000 and 2600 kg/m$^3$ and heavy weight more than 2600 kg/m$^3$. The strength classes range from C8/10 until C 100/115 and for lightweight from LC 8/9 until LC 80/88. The whole range of normal strength and high strength concrete is covered. Concrete is specified in three categories: there is a *standard concrete* for low strength until C 16/20 with rather fixed composition. The next one is designed concrete which is the most used. There, the properties are specified such as strength, workability and others.
There is a third category which is called the prescribed concrete. There, the composition is prescribed and the concrete producer mixes the concrete according to the specifications which are received from the specifier.

There are several rules for the conformity of the concrete. The rules are given in detail for the concrete manufacturer and also for the concrete consumer. All requirements are based on statistics, the so-called characteristic strength depends on a 5% quantile of the normal distribution. A new aspect of the conformity control is the concrete family. A family consists of similar concrete mixes, i.e. they should be of the same type of cement, aggregates from the same geological origin, not more than five strength classes. Also lightweight aggregate concrete may fall into a concrete family. The aim of this procedure is to reduce the number of specimens which are necessary for the conformity control of concrete.

Together with the EN 206 there is also a standard for the execution of concrete works (EN 13670). There is an important chapter on the curing of concrete. Curing measures depend on the type of cement, the temperature and the environment during hardening of the concrete.

Table 1. Minimum curing time of concrete in exposure classes according to EN 206-1 except X0 and XC1

<table>
<thead>
<tr>
<th>Surface temperature $T$ [°C]</th>
<th>Minimum curing time [days]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Strength development of concrete</td>
</tr>
<tr>
<td>1 $T \geq 25$</td>
<td>1</td>
</tr>
<tr>
<td>2 $25 &gt; T \geq 15$</td>
<td>1</td>
</tr>
<tr>
<td>3 $15 &gt; T \geq 10$</td>
<td>2</td>
</tr>
<tr>
<td>4 $10 &gt; T \geq 5$</td>
<td>3</td>
</tr>
</tbody>
</table>

It can be seen that the duration of curing is rather long if there is a low temperature and if a slow cement is used. On the other hand, at higher temperature and in the case of a fast cement the curing time is rather short.
There are three classes of supervision. The first class with the lowest requirements are valid for strength classes $\leq \text{C}25/30$. The next higher supervision class is applicable for $\text{C} 30/37$ to $\text{C} 50/60$. The highest supervision class is reserved for high strength concrete $\geq \text{C}55/67$. Before the new standards were made there were national guidelines for high strength concrete. These are now obsolete.

The standard contains the famous tables F2.1 and F2.2. These tables specify the composition of the concrete with respect to environmental classes. Environmental classes have been introduced for the detailed description of environment. Former standards were rather crude with respect to durability. We have now 18 durability classes ranging from X0 until XA3. The main parameter is the kind of attack of the environment. There are two main causes: the first one is the corrosion of the reinforcement. There is the aspect of carbonation of concrete and second is the ingress of chlorides. Chlorides either stem from the deicing salts or from seawater. Then, there is the corrosion of concrete due to frost action and frost with deicing salts. Secondly, there is aggression of chemicals in water or soil. In Germany there is a third type of aggressive environment, i.e. mechanical abrasion. The table shows the exposure classes as specified in EN 206-1 with an informative description of actions.

### Table 2. Exposure classes acc. to EN 206-1

<table>
<thead>
<tr>
<th>Class designation</th>
<th>Description of the environment</th>
<th>Informative examples where exposure classes may occur</th>
</tr>
</thead>
<tbody>
<tr>
<td>1  No risk of corrosion or attack</td>
<td></td>
<td></td>
</tr>
<tr>
<td>X0</td>
<td>For concrete without reinforce- ment or embedded metal: All exposures except where there is freeze/thaw, abrasion or chemical attack</td>
<td>Concrete inside buildings with very low air humidity</td>
</tr>
<tr>
<td></td>
<td>For concrete with reinforcement or embedded metal: Very dry</td>
<td></td>
</tr>
<tr>
<td>2  Corrosion induced by carbonation</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Where concrete containing reinforcement or other embedded metal is exposed to air and moisture, the exposure class shall be classified as follows:

NOTE: The moisture condition relates to that in the concrete cover to reinforcement or other embedded metal but, in many cases conditions in the concrete cover can be taken as reflecting that in the surrounding environment. In these cases classification of the surrounding environment may be adequate. This may not be the case if there is a barrier between the concrete and its environment.

| XC1       | Dry or permanently wet | Concrete inside buildings with low air humidity  
|           |                        | Concrete permanently submerged in water |
| XC2       | Wet, rarely dry        | Concrete surfaces subject to long-term water contact  
|           |                        | Many foundations |
| XC3       | Moderate humidity      | Concrete inside buildings with moderate or high air humidity  
|           |                        | External concrete sheltered from rain |
| XC4       | Cyclic wet and dry     | Concrete surfaces subject to water contact, not within exposure class XC2 |

3 Corrosion induced by chlorides other than from sea water

Where concrete containing reinforcement or other embedded metal is subject to contact with water containing chlorides, including de-icing salts, from sources other than from sea water, the exposure shall be classified as follows:

| XD1       | Moderate humidity      | Concrete surfaces exposed to airborne chlorides |
|XD2| Wet, rarely dry| Swimming pools  
Concrete exposed to industrial waters containing chlorides|
|---|---|---|
|XD3| Cyclic wet and dry| Parts of bridges exposed to spray containing chlorides  
Pavements  
Car park slabs|

4 Corrosion induced by chlorides from sea water

Where concrete containing reinforcement or other embedded metal is subject to contact with chlorides from sea water or air carrying salt originating from sea water, the exposure shall be classified as follows:

<table>
<thead>
<tr>
<th>XS1</th>
<th>Exposed to airborne salt but not in direct contact with sea water</th>
<th>Structures near to or on the coast</th>
</tr>
</thead>
<tbody>
<tr>
<td>XS2</td>
<td>Permanently submerged</td>
<td>Parts of marine structures</td>
</tr>
<tr>
<td>XS3</td>
<td>Tidal, splash and spray zones</td>
<td>Parts of marine structures</td>
</tr>
</tbody>
</table>

5 Freeze / thaw attack

Where concrete is exposed to significant attack by freeze/thaw cycles whilst wet, the exposure shall be classified as follows:

<table>
<thead>
<tr>
<th>XF1</th>
<th>Moderate water saturation, without deicing agent</th>
<th>Vertical concrete surfaces exposed to rain and freezing</th>
</tr>
</thead>
<tbody>
<tr>
<td>XF2</td>
<td>Moderate water saturation, with deicing agent</td>
<td>Vertical concrete surfaces of road structures exposed to freezing and airborne deicing agents</td>
</tr>
<tr>
<td>XF3</td>
<td>High water saturation, without deicing agent</td>
<td>Horizontal concrete surfaces exposed to rain and freezing</td>
</tr>
</tbody>
</table>
| XF4 | High water saturation, with de-icing agent or sea water | Road and bridge decks exposed to deicing agents  
Concrete surfaces exposed to direct spray containing deicing agents and freezing  
Splash zone of marine structures exposed to freezing |

6 Chemical attack

Where concrete is exposed to chemical attack which occurs in natural soils and ground water as given in table 2, the exposure shall be classified as given below. The classification of sea water depends on the geographical location; the classification valid in the place of use of the concrete applies.

Note:
A special study may be needed to establish the relevant exposure condition where there is:
- limits outside of table 2;
- other aggressive chemicals;
- chemically polluted ground or water;
- high water velocity in combination with the chemicals in table 2.

| XA1 | Slightly aggressive chemical environment acc. to table 2 |
| XA2 | Moderately aggressive chemical environment acc. to table 2 |
| XA3 | Highly aggressive chemical environment acc. to table 2 |

The famous tables F2.1 and F2.2 specify the composition of the concrete. However, these tables could not be agreed upon in the technical committee. The differences of climate between Southern Europe, which is subtropical, and Northern Europe, which is almost arctic, were too large. Also the national experiences with concrete compositions deem to satisfy the environmental actions.
So the tables are informative and national rules have been edited with respect to the concrete composition. As an example the German specifications are given in Table 3.

Strictly speaking, the tables are only valid for CEM I cement (Portland cement). So, in addition a rather large table has been issued with respect to the use of the other 26 cements specified in EN 197-1. Due to space constraints this table is not shown. For the Europeans it was a milestone to have one common standard in Europe.

Table 3. National specifications for the composition of concrete (DIN 1045-2)

<table>
<thead>
<tr>
<th>Concrete attack</th>
<th>Frost without/with deicing salt or seawater</th>
<th>Chemical attack</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exposure class</td>
<td>XF1</td>
<td>XF2 1)</td>
</tr>
<tr>
<td>max. w/c ratio</td>
<td>0.60</td>
<td>0.55</td>
</tr>
<tr>
<td>min. cement content, kg/m³</td>
<td>280</td>
<td>300</td>
</tr>
<tr>
<td>min. cement content with addition, kg/m³</td>
<td>270</td>
<td>270</td>
</tr>
<tr>
<td>min. air content, % by vol.</td>
<td>4.0</td>
<td>4.0</td>
</tr>
<tr>
<td>Other requirements</td>
<td>Aggregate with additional requirement against frost and deicing salt</td>
<td>F4</td>
</tr>
</tbody>
</table>

1) Additions may not be taken into account for the calculation of water-cement ratio and min. cement content

SELF-COMPACTING CONCRETE (SCC)

SCC is a development which started in Japan in the 80’s of the last century. However, it is forgotten that the underwater concrete which has been used in Europe long before had similar properties with similar composition. However, it was never used in normal, say dry, environment. There are specifications for SCC in various countries such as The Netherlands or Germany. For the measurement of workability we use the spread table, the funnel and the J-ring. For the
design of SCC a relation has been established between the funnel time and the slump flow as Fig. 2 shows.

![Image of Fig. 2: Window for the design of SCC](image)

*Fig. 2. „Window“ for the design of SCC [3]*

The figure shows the relation between the two properties. It has been demonstrated that all concretes which lie in the window in the middle of the figure, fulfill the requirements of self compacting concrete. The left hand concretes are not workable since they are blocking. The right hand concretes outside the window segregate. However, the window is not valid for all types of concretes since the composition can be adjusted to the amount of fines or the amount of stabilising agents. So, a concrete producer will always make his own window and will check the outcome of the conformity control against the window. It should be mentioned that the use of SCC is still very restricted in the European countries. An inquiry has shown that the consumption is about 15 % in Denmark, 1 % in Germany, 5 % in Sweden, 4 % in Switzerland.

The rather low consumption of SCC is due to restricted experiences with this type of concrete. The higher price, the sensitivity to composition variation and also the fact that labour cost is rather low in many European countries due to a surplus of labourers, are responsible.
ULTRA-HIGH PERFORMANCE CONCRETE (UHPC)

UHPC has started in France when the company Bouygues developed this fine grain high-strength concrete. The composition of the concrete is about the following:

RPC Premix with: 700 kg CEM, 225 kg silicafume, 990 kg sand, 210 kg quarz flour, 45 kg plasticiser, + 195 kg water + 200 kg steel fibers.

This mix with a high cement content and with very low water cement ratio, with the use of silica fume and small grain aggregates lead to strength classes of 150 MPa. Usually fibres are added in order to reduce the brittleness of the material. If heat treatment is added to normal curing then the strength can even be extended to 230 MPa. Executed examples show that rather elegant structures can be made of this material.

Meanwhile, the material spread over the whole world. Examples are available from Japan, Korea and Canada. What is still missing is the intrinsic use of this type of materials. All structures so far follow more or less the traditional design of a concrete structure. However, the material should allow for much more diversity and shapes. Since the material is fluent in the beginning and hardens in a form. One can think upon the manufacture of cast iron especially as prefabricated elements are concerned.

Fig. 3. Bridge in Quebec, Canada [4]
TEXTILE REINFORCED CONCRETE (TRC)

Textile reinforced concrete is a new generation of fibre reinforced concrete. The reinforcement consists of continuous fibres which are made of polymer material. Polymers are aramid and carbon. Also the mineral material glass can be used. Table 4 shows three types of fibres which are currently being used.

Table 4. Examples of textile reinforcement [6]

<table>
<thead>
<tr>
<th>Property, unit</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C</td>
</tr>
<tr>
<td>Material</td>
<td>Carbon</td>
</tr>
<tr>
<td>Structure of fabric</td>
<td>Biaxial</td>
</tr>
<tr>
<td></td>
<td>0°/90°</td>
</tr>
<tr>
<td>Weight, g/m²</td>
<td>320</td>
</tr>
<tr>
<td>Roving, tex</td>
<td>1700</td>
</tr>
<tr>
<td>Area of one roving, mm²</td>
<td>0.9</td>
</tr>
<tr>
<td>Mesh size, square, mm</td>
<td>10</td>
</tr>
<tr>
<td>Maximum tensile force per roving, N</td>
<td>1100</td>
</tr>
<tr>
<td>Tensile strength of roving, MPa</td>
<td>1220</td>
</tr>
</tbody>
</table>
The reinforcements are rovings of these materials. Rovings are composed of hundreds of filaments with a diameter of some millions. The tensile strength is very high ranging between 1500 and 3000 MPa. All these types of material are elastic almost upon fracture. Despite this elasticity and non yielding behaviour one can make ductile structural elements. Ductility is due to delamination and bond failure rather due to brittle fracture of the material. Fig. 6 shows the load deflection behaviour of strips in the bending test.
Fig. 6 shows clearly the linear elastic beginning of the load deflection curve. It shows the cracking load and it shows also the deflection hardening behaviour of the material. Finally, after reaching the maximum load one gets failure but not a very brittle one but a decay of the load with increasing deflection. This behaviour can be improved if the textile is prestressed.

Due to prestressing the first elongation of the textile is eliminated which is due to the stretching of the material. All woven fabrics show a certain undulation of the filaments. When the material is prestressed this undulation is suppressed. Furtheron, we have also an increase of the load due to the fictitious increase of the tensile strength of the matrix so both facts yield an increase of the cracking load and also an increase of the strain hardening of the material.
ENVIRONMENT

There is a rather long tradition in Europe with respect to the sustainability of materials and structures. The Kyoto agreement has been signed by the European countries and this enforces the countries to reduce CO₂ emission and strive for more environmental consciousness. To reduce CO₂ emission by cement production is the use of blended cements (CO₂ production per t Portland cement is 0.93 t), the same is true for energy consumption.

Energy

The use of composite cements is spreading in Europe. Two main factors enforce the cement producing companies to make composite cements instead of pure Portland cement: first the consumption of energy and second, the use of by-products of other fabrication processes. Burning of Portland cement klinker is a very energy consuming process. Per ton of klinker one needs 3500 MJ of energy. If a part of cement can be replaced by other materials which is ready one can save a great deal of energy. This has been used for decades for the blast furnace slag cement which has a very long tradition in Europe. The first application goes back to 1872. Since that time blast furnace slag cement is used in all countries of Europe. For instance, the Netherlands use more slag cement than Portland cement which has various reasons.

![Fig. 7. Energy demand for cement production [7]](image-url)
The first reason is that there is a very good slag available from the Dutch steel works in Ijmuiden. Second, it has been shown that blast furnace slag cement is superior to Portland cement in marine environment. Third, it produces less heat of hydration which makes it very suitable for thick-walled structures which are occurring in hydraulic structures very frequently. Nowadays, all other types of secondary material is used for cement production. These can be either natural products such as limestone or trass, these can be modified natural products such as phonolite, or these can be artificial products such as flyash and silica fume. These products make that the energy consumption is lower for the blended cement production than for the pure Portland cement.

**Co-combustion**

There is a great pressure on the ecological reuse of waste materials. One can think about the reuse of the material as such, as the replacement of other material in the building and construction process or one can think about the thermal recovery of the energy which is accumulated in the material. Looking to the materials involved, namely petcoke, sewage sludge, paper sludge, bone meal, straw, biomass and even banknotes (when the old currencies in Europe were replaced by the Euro), one realizes immediately that an ecological and economical reuse of these materials is impossible. However, a thermal recovery of the material is feasible. Power plants use it as a welcome replacement of coal.

Of course, the consequences on fly ash have to be checked with respect to the influence on the concrete like workability, air content, hydration of cement, freeze-thaw and de-icing salt resistance and maybe other properties. The aim should be to determine the limit of secondary fuel which can be used without changing the fly ash properties significantly. Some countries have carried out extensive test programs in order to establish such limits. It turned out that the limits depend on the type of material which is co-combusted, ranging between 5 and 15%. In view of the large qualities which are fired in the power plants it is a considerable amount.

The result of co-combustion results in an important ecological and, at the same time, economical profit. As long as the concrete properties do not change significantly due to this type of fly ash there are no arguments against the use of secondary fuel. The standardization organization has to make sure the limits of co-combustion.
In many countries in Europe experience with co-combustion has been gained. In a CEN report [8] there are requirements about the composition of the material used as co-combustion material in different countries as Table 5 shows. Table 6 shows the technical requirements for flyash from co-combustion according to EN 450.
Table 5. Requirements for materials allowed to be used as co-combustion material in different European countries

<table>
<thead>
<tr>
<th>Country</th>
<th>property/parameter</th>
<th>Unit</th>
<th>DE</th>
<th>Be</th>
<th>DK</th>
<th>I</th>
</tr>
</thead>
<tbody>
<tr>
<td>Co-combustion material</td>
<td>Sewage sludge</td>
<td>Sewage sludge</td>
<td>Straw</td>
<td>Residues from coal and coke from steel industry</td>
<td>Municipal waste</td>
<td>Sewage sludge</td>
</tr>
<tr>
<td>General properties</td>
<td>Calorific value (LL)</td>
<td>kJ/kg</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>16000</td>
</tr>
<tr>
<td></td>
<td>Humidity (UL)</td>
<td>%</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>8/6</td>
</tr>
<tr>
<td></td>
<td>Ash content (UL)</td>
<td>%</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Element content (UL)</td>
<td>Chloride</td>
<td>%</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Cl (organic)</td>
<td>mg/kg</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Sulfur</td>
<td>%</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2/1.5</td>
</tr>
<tr>
<td></td>
<td>P₂O₅</td>
<td>%</td>
<td>-</td>
<td>25</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Heavy metal content (UL)</td>
<td>mg/kg dry matter</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-------------------------</td>
<td>------------------</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Arsenic</td>
<td>- 250</td>
<td>-</td>
<td>9</td>
<td>9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cadmium</td>
<td>10 10</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Chromium</td>
<td>900 1250</td>
<td>-</td>
<td>100 100 50</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Copper</td>
<td>800 375</td>
<td>-</td>
<td>-</td>
<td>- 300</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Copper (soluble)</td>
<td>-</td>
<td>-</td>
<td>300 300 -</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mercury</td>
<td>8 5</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Manganese</td>
<td>-</td>
<td>-</td>
<td>400 400 150</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nickel</td>
<td>200 250</td>
<td>-</td>
<td>40 40 20</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lead</td>
<td>900 1250</td>
<td>-</td>
<td>-</td>
<td>- 200</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lead (volatile)</td>
<td>-</td>
<td>-</td>
<td>200 200 -</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Zink</td>
<td>- 1250</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cd+Hg</td>
<td>-</td>
<td>-</td>
<td>7 7 7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vanadium</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1) In the Netherlands the requirements are based on the framework of emission limits and limits of hazardous wastes and part of the permits for co-combustion. the Dutch power plants analyse the co-combustion streams and reject the policy for the choice of co-combustion materials on the quality of the byproducts.

2) related to the ash of the sewage sludge

3) during the technical process of steel production coal and coke is combusted; from this combustion residues occur

LL: lower limit
UL: upper limit
Table 6. Technical requirements for fly ash from co-combustion according to EN 450

<table>
<thead>
<tr>
<th></th>
<th>Reference</th>
<th>EN 450</th>
</tr>
</thead>
<tbody>
<tr>
<td>LOI</td>
<td>% by mass</td>
<td>≤ 5</td>
</tr>
<tr>
<td>Cl</td>
<td>% by mass</td>
<td>≤ 0.10</td>
</tr>
<tr>
<td>SO₃</td>
<td>% by mass</td>
<td>≤ 3.0</td>
</tr>
<tr>
<td>Free CaO</td>
<td>% by mass</td>
<td>≤ 1.0</td>
</tr>
<tr>
<td>Fineness</td>
<td>% by mass</td>
<td>≤ 40</td>
</tr>
<tr>
<td>Activity index</td>
<td>%</td>
<td>≥ 75 at 28 d</td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 85 at 91 d</td>
</tr>
<tr>
<td>Soundness</td>
<td>mm</td>
<td>≤ 10</td>
</tr>
</tbody>
</table>

One can see that there are requirements with respect to loss of ignition, chloride, sulphate, free calcium oxide and fineness and the activity and the saltiness. An extensive investigation has been made in the Netherlands.

Many co-combustion materials have been used. An ecological concern is that heavy metals and other elements can be leached from concrete which contains flyash with co-combustion material. Tests in Germany have shown that leachate from the co-combustion flyash is within the range of the concentration in the leachate obtained from coal flyash.

Comparing the results with the requirements one can state that all co-combustion materials have fulfilled the requirements when they are added as small percentage of the coal.

Tests have also been carried out on the durability of concrete. An example is the ingress of chloride. Fig. 8 shows that there are no large differences between the reference concrete and the concrete made of co-combustion flyash.
Also the freeze-thaw behaviour has been investigated. The conclusion was that there were no significant differences between concrete samples with flyash from co-combustion and reference flyashes. Summarizing it must be noted that it is an economical and ecological advantage to use waste material as co-combustion material. If the co-combustion material is limited with respect to the coal which is incinerated one can always stay within the limits of concentration of heavy metals and other elements. In Germany work was focussed on flyash obtained from co-combustion and sewage sludge up to maximum of 5% by mass as replacement of coal. The results show that 5% had no significant influence on the EN 450 properties and the leaching behaviour of the flyash. In Germany, a co-combustion of sewage sludge up to an amount of 5% by mass is allowed. Trials have been performed with petcoke within the framework of technical approvals with petcoke contents up to 50% by mass replacement of coal. No significant influence was reported.

Recycling

Recycling of building material is encouraged in Europe. There are several guidelines available in various European countries. For example, the German DIN 1045-2 together with [9] allows 45% recycled material for exposure class X0 and XC1 to XC4, 35% for XF1 and XF3, 25% for XA1.
Table 7. Allowable percentage of recycled aggregate > 2 mm with respect to total aggregate. 
Type 1 aggregate contains ≥90% crushed concrete, type 2 ≥70% [9]

<table>
<thead>
<tr>
<th>Arrange of application</th>
<th>Aggregate type 1 acc. to DIN 4226-100</th>
<th>Aggregate type 2 acc. to DIN 4226-100</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASR guideline</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WO (dry)</td>
<td>Carbonation XC1</td>
<td></td>
</tr>
<tr>
<td>1)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WF (moist)</td>
<td>No risk of corrosion X0</td>
<td>≤ 45</td>
</tr>
<tr>
<td>1)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WF (moist)</td>
<td>Carbonation XC1 to XC4</td>
<td>≤ 35</td>
</tr>
<tr>
<td>1)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WF (moist)</td>
<td>Frost without de-icing salt XF1</td>
<td>≤ 35</td>
</tr>
<tr>
<td>1)</td>
<td>and XF3 [1)</td>
<td></td>
</tr>
<tr>
<td>WF (moist)</td>
<td>and in concrete with high water</td>
<td>≤ 25</td>
</tr>
<tr>
<td>WF (moist)</td>
<td>penetration resistance</td>
<td></td>
</tr>
<tr>
<td>WF (moist)</td>
<td>chemical resistance (XA1)</td>
<td>≤ 25</td>
</tr>
<tr>
<td>1)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

[1) additional requirements see § 1, (3) and (4)]

It is also allowed to use recycled water and fresh concrete [10]. When a recycling aid is used [1] the fresh concrete can be saved for several days and reactivated together with new fresh concrete.

CONCLUSION

Concrete exists in Europe since the middle of the 19th Century. The development is continuing. The emphasis shifted from strength to durability and environmental issues. This aspect has been demonstrated at the new European standard for concrete and also some concrete examples. On the other hand, new materials evolve which have superior properties.
REFERENCES


FIRE BEHAVIOUR OF PLAIN SELF-COMPACTING CONCRETE (SCC)

BRANDVERHALTEN VON UNBEWEHRTEM SELBSTVERDICHTEN- DEN BETON (SVB)

COMPORTEMENT DU BETON PLEIN ET AUTOCOMPACTANT DANS L’ INCENDIE

Michael Stegmaier, Hans-Wolf Reinhardt

SUMMARY

In this research Self-Compacting concretes designed as Powder Type, Viscosity-Agent Type and Combination Type were investigated with respect to their behaviour at fire conditions. Therefore cubes with an edge length of 300 mm without reinforcement were cast and subjected to a fire with a duration of 2 h according to ISO 4102 T. 2 after storage of 180 d. The spalling of the concretes depended on the \((\text{w/c})_{\text{eq}}\)-ratio and the cement-powder ratio \((\text{c/p})\). The concretes showed a relatively high residual compressive strength which depended also on the \((\text{w/c})_{\text{eq}}\)-ratio and the cement-powder ratio \((\text{c/p})\) of the mix.

ZUSAMMENFASSUNG

RESUMÉ

Dans ces recherches les bétons autocompactants sont explorés avec l’effort de l’incendie. Les bétons sort au type de poudre, au type de stabilisation et au type de combination. Pour cela des cubes avec une longueur de 300 mm ont été faits sans armature. Après 180 jours de stockage ils ont été exposés a une incendie de deux heures selon ISO DIN 4102 part 2. Le dommage de la surface des bétons a dependu d’équivalent du rapport eau/ciment et du rapport ciment/poudre. De plus par comparaison les bétons autocompactants ont en la disposition de la haute resistance residuelle qui a été influencée par l’équivalent du rapport eau/ciment et du rapport ciment/poudre.

KEYWORDS: SCC, fire, spalling, residual compressive strength

The results presented in this paper are a brief summary of a research that was supported by the DAfStb. Details of the investigations and an overview of research related to fire behaviour of SCC can be found in [1].

1. RANGE OF INVESTIGATION AND MIX COMPOSITION

The compressive strength of the individual concretes was determined on separately cast cubes of an edge length of 150 mm at an age of 28 days. These cubes were stored for 7 days at 20 °C and 100 % RH and for 21 days at 20 °C and 65 % RH. The specimens for the fire tests were stored in the same manner for the first 28 days. After this time they were placed in a room with a temperature of 22 °C and 40 % RH till the age of 180 days. The loss of weight due to drying of these specimens was measured every month. The loss of mass of the cubes can be calculated as follows:

\[
\Delta m_r = (\frac{m_i}{m_0} - 1) \times 100 \% \quad [\% \text{ by mass}] 
\]

with:
- \(\Delta m_r\) = loss of mass of the specimen due to drying [% by mass]
- \(m_i\) = mass of the specimen at time i [kg]
- \(m_0\) = mass of specimen after 1 day [kg]
After the fire test the damage caused by the fire was documented and the specimens were cleaned by brushing off the loose parts. The loss of mass of the specimens due to spalling was calculated with the following equation:

$$\Delta m = (m_{nach} - m_{vor}) \cdot 100\% \quad [\text{by mass}] \quad (2)$$

with

- $\Delta m$ = loss of mass of the specimen due to fire [% by mass]
- $m_{vor}$ = mass of the specimen before the fire test [kg]
- $m_{nach}$ = mass of the specimen after the fire test [kg]

Additionally to the spalling of the concretes the residual compressive strength of the concretes was measured. For this purpose a core was drilled from one specimen of each concrete mix.

The composition of the different concretes is presented in table 1. The designed compressive strength of the concretes varies between 25 MPa and 75 Mpa. The nomenclature of the mixes is as follows: the initial letter represents the type of SCC. K stands for Combination Type, M for Powder Type and S for Viscosity Agent Type. The following number describes the designed compressive strength of the SCC. That means the mix M55/67 is a Powder Type SCC with a strength class of C55/67.

**Table 1: mix composition of the investigated SCC.**

<table>
<thead>
<tr>
<th></th>
<th>Combination Type</th>
<th>Powder Type</th>
<th>Viscosity Agent Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C20/25</td>
<td>C25/45</td>
<td>C55/67</td>
</tr>
<tr>
<td></td>
<td>C20/25</td>
<td>C35/45</td>
<td>C55/67</td>
</tr>
<tr>
<td></td>
<td>C20/25</td>
<td>C55/67</td>
<td>C60/75</td>
</tr>
<tr>
<td></td>
<td>C25/30</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>General:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cement</td>
<td>CEM III A-LL</td>
<td>CEM III A-LL</td>
<td>CEM III A-LL</td>
</tr>
<tr>
<td></td>
<td>32,5 R</td>
<td>32,5 R</td>
<td>42,5 R</td>
</tr>
<tr>
<td>Cement content [kg/m³]</td>
<td>240</td>
<td>300</td>
<td>350</td>
</tr>
<tr>
<td>Water content [kg/m³]</td>
<td>160</td>
<td>160</td>
<td>160</td>
</tr>
<tr>
<td>Volume of aggregates [dm³/m³]</td>
<td>624</td>
<td>644</td>
<td>619</td>
</tr>
<tr>
<td>Weight of aggregates [kg/m³]</td>
<td>1624</td>
<td>1675</td>
<td>1611</td>
</tr>
<tr>
<td><strong>Fillers:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fly ash [kg/m³]</td>
<td>0.0</td>
<td>99.0</td>
<td>118.8</td>
</tr>
<tr>
<td>Limestone powder [kg/m³]</td>
<td>315.8</td>
<td>104.1</td>
<td>76.9</td>
</tr>
<tr>
<td><strong>Admixtures:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>Wörmann FM/BV 375</td>
<td>Wörmann FM/BV 375</td>
<td>Wörmann FM/BV 375</td>
</tr>
<tr>
<td></td>
<td>1.25</td>
<td>1.35</td>
<td>1.35</td>
</tr>
<tr>
<td>Content of superplasticizer [% v.C.]</td>
<td>1.25</td>
<td>1.25</td>
<td>1.25</td>
</tr>
<tr>
<td>Viscosity agent</td>
<td>Wörmann UW - Compound</td>
<td>Wörmann UW - Compound</td>
<td>Wörmann UW - Compound</td>
</tr>
<tr>
<td></td>
<td>0.0</td>
<td>0.10</td>
<td>0.10</td>
</tr>
<tr>
<td>Viscosity agent content [% v. C.]</td>
<td>0.20</td>
<td>0.10</td>
<td>0.10</td>
</tr>
<tr>
<td>Powder content [kg/m³]</td>
<td>569</td>
<td>516</td>
<td>560</td>
</tr>
<tr>
<td>(w/c)eq - ratio [1]</td>
<td>0.71</td>
<td>0.49</td>
<td>0.43</td>
</tr>
</tbody>
</table>

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The \((w/c)_{eq}\)-ratio used in table 1 is calculated as follows:

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

(3)

with:

\(w\) = amount of water in the mix [kg/m³]
\(c\) = amount of cement [kg/m³]
\(f\) = amount of fly ash \((f \text{ countable} \leq 0.33c)\) [kg/m³]

2. FRESH CONCRETE PROPERTIES

Before the specimens for the fire tests were cast, the usual fresh concrete tests to characterize the rheological properties and the tendency of blocking of the mixes were performed. Bleeding and segregation was estimated directly at the fresh concrete. Additionally the distribution of the coarse aggregates within the cubes with an edge length of 150 mm was investigated after the compressive tests were performed. The results of the fresh concrete tests are shown in table 2.

\begin{table}[h]
\centering
\begin{tabular}{|l|c|c|c|c|c|c|}
\hline
\hline
K20/25 & 10.5 & 750 & 750 & 3.8 & 2.24 & 23.9 \hline
K35/45 & 13.0 & 720 & 725 & 2.9 & 2.30 & 24.1 \hline
K55/67 & 18.0 & 690 & 690 & 1.5 & 2.36 & 24.2 \hline
M20/25 & 11.0 & 780 & 785 & 4.8 & 2.19 & 23.4 \hline
M35/45 & 12.0 & 740 & 730 & 0.8 & 2.35 & 23.6 \hline
M55/67 & 15.5 & 745 & 750 & 0.8 & 2.33 & 24.9 \hline
M60/75 & 12.0 & 770 & 730 & 1.3 & 2.36 & 26.5 \hline
S25/30 & 8.0 & 680 & 635 & 0.9 & 2.34 & 22.8 \hline
\end{tabular}
\caption{results of the fresh concrete tests.}
\end{table}

The fact that some slump flow results with J-Ring are larger than those without J-Ring can be explained by the inaccuracy of this test and the inherent distribution. This is possible to occur when single measurements are performed. All the used concretes showed no tendency to blocking and no segregation of coarse aggregates [1].

3. RESULTS

3.1 Compressive strength

The results of the compressive strength tests with the cubes with an edge length of 150 mm are shown in figure 1. The compressive strengths of the concretes with comparable \((w/c)_{eq}\)-ratios are all within the same magnitude.
The difference between M55/67 and M60/75 however is relative small compared to the difference in mix composition. This can be explained by the different air contents of the mixes (see table 2). The different air contents of concrete mixes can be taken into account with the following equation [2]:

\[ f_c = f_{c0} \cdot 10^{-0.035p} \]  \hspace{1cm} (4)

with:

- \( f_c \) = compressive strength with air pores [Mpa]
- \( f_{c0} \) = compressive strength of the concrete without air [Mpa]
- \( p \) = air content [% by vol.]

When this formula is used the difference between those two mixes increases from 4.4 MPa to 8.0 MPa. A difference in this magnitude seems reasonable considering the difference in the \((w/c)_{eq}\)-ratio so the relatively high air content of M60/75 is the reason for the low compressive strength of this mix.

3.2. Residual compressive strength after fire test

Figure 2 shows the residual compressive strengths of the used concretes related to the compressive strength of the concretes at the age of 28 days.
Figure 2: residual relative compressive strength after the fire related to the compressive strength after 28 days.

The residual compressive strength of the tested SCC’s is relatively high compared to normal concretes [3, 4, 5]. There is a clear correlation between the (w/c)_eq-ratio of the concretes and the residual strength (see figure 3).

Figure 3: residual compressive strength vs. the (w/c)_eq-ratio
An increasing \((w/c)_{eq}\)-ratio results in an increasing residual compressive strength of the concrete. Additionally the residual compressive strength is related to the cement-powder ratio \(c/p\) of the individual mix. An increasing \(c/p\)-ratio leads to a reduction of the residual compressive strength (see figure 4).

![Figure 4: Residual compressive strength vs. the cement-powder ratio.](image)

### 3.3 Loss of mass in fire test

Those high residual compressive strengths of the SCC mixes are quite astonishing as the spalling of the specimens was relatively strong. The loss of weight reached values between 44.8 % and 62.8 % of the initial mass of the specimen (see figure 5).
It is important to note that after the fire almost all specimens had still the shape of a cube (see figure 6). But the outer parts of the concrete could easily be removed by brushing (see figure 7). Though these loose parts of the concrete do not have any strength left they still can act as thermal insulation for existing reinforcement in the structure. So the spalling is possibly not as bad as it seems. But to reconfirm this assumption further research with reinforced specimens is necessary.

Figure 5: spalling of the concretes due to fire (mean of 3 cubes).

Figure 6: cube 3 of mix K55/67 after fire

Figure 7: cube 3 of mix K55/67 after brushing off the loose parts.
Analogous to the residual strength there is a correlation between the spalling of the concretes and the (w/c)_{eq}-ratio (see figure 8). This correlation is not as good as it is for the residual strength and more tests are necessary to confirm this statement.

Figure 8: spalling vs. the (w/c)_{eq}-ratio.

It seems that the spalling decreases with increasing (w/c)_{eq}-ratio. There is also a relationship between the spalling of the concrete and the cement-powder ratio (see figure 9). But here also more data is necessary to confirm this correlation.

Figure 9: spalling vs. the cement-powder ratio.
CONCLUSIONS

Fire tests with eight Self-Compacting Concretes (SCC) with different compositions have been carried out. The tested concretes showed a relatively high residual strength after the fire ranging from 47 % to 67 %. A strong correlation between the \((w/c)_{eq}\)-ratio and the residual compressive strength of the concretes was found. With increasing \((w/c)_{eq}\)-ratio the residual strength of the mixes increased too. There was also a good correlation between the cement-powder ratio \(c/p\) and the residual compressive strength. Here a increasing \(c/p\)-ratio leads to a decreasing residual strength.

During the fire test only little spalling occurred. The specimens showed numerous cracks while testing and after cooling down to room temperature but still had the shape of a cube. The outer parts of the specimens however could easily be removed by brushing. Concerning the condition of the specimens after brushing, spalling between 45 % and 63 % was measured. There is also a correlation between spalling and the \((w/c)_{eq}\)-ratio and also between spalling and the \(c/p\)-ratio but more data are necessary to confirm these findings.

REFERENCES


DETERMINATION OF CHARACTERISTIC VALUES FOR STRUCTURAL TIMBER

BESTIMMUNG CHARAKTERISTISCHER RECHENWERTE VON BAUHOLZ FÜR TRAGENDE ZWECKE

DÉTERMINATION DES VALEURS CARACTÉRISTIQUES DE DIMENSIONNEMENT DU BOIS DE STRUCTURE

Wolfgang Klöck

SUMMARY
This paper addresses the determination of characteristic values for structural timber as stated in the European standard EN 14358. For the user of this standard, however, the specific statistical result of the calculation is not evident, as the terminology applied is misleading. In addition, the fundamentals of the subject are rarely presented in literature. Hence, a comprehensive derivation of the statistical factors underlying the evaluation procedure proposed in EN 14358 is given. Statistical assumptions and the mathematical background are illustrated in full detail. Characteristic values are shown to be lower one-sided confidence intervals for the 5%-quantile; not the 5%-quantile as stated by the current standard.

The tables which present the statistical factors in EN 14358 are rather incomplete and specify numerical values only for a few sample sizes. In order to make the application of EN 14358 more convenient and numerically more precise, complete tables with statistical factors ranging up to a sample size of 100 are given for both confidence levels applied in the standard.

ZUSAMMENFASSUNG
Der Aufsatz beschäftigt sich mit der Bestimmung charakteristischer Rechenwerte von Bauholz für tragende Zwecke. Dabei wird auf das Auswerteverfahren Bezug genommen, welches in der europäischen Norm EN 14358 angegeben ist. Für den Anwender der Norm geht aus dem Normentext jedoch nicht zweifelsfrei hervor, was das Ergebnis der Rechnung im statistischen Sinne ist, da die Terminologie der Norm nicht korrekt ist. Da zudem die Grundlagen des
Auswerteverfahrens in der Fachliteratur, wenn überhaupt, nur bruchstückhaft behandelt werden, wird hier eine vollständige Herleitung der statistischen Faktoren, die dem Verfahren zugrunde liegen, angegeben. Dabei werden sowohl die statistischen Annahmen als auch der mathematische Hintergrund dargestellt. Aus der Herleitung der statistischen Faktoren geht hervor, daß nicht, wie in der Norm angegeben, die 5%-Fraktile, sondern vielmehr ein einseitiges unteres Vertrauensintervall der 5%-Fraktile als charakteristischer Rechenwert erhalten wird.

Die in EN 14358 angegebenen Tabellen sind lückenhaft und weisen nur für einige wenige Stichprobenumfänge statistische Faktoren aus. Um die Anwendung der Norm zu erleichtern, werden vollständige Tabellen bis hin zu einem Stichprobenumfang von 100 Proben für die beiden in EN 14358 angewandten Vertrauensniveaus angegeben.

RÉSUMÉ

Cet article porte sur la détermination des valeurs caractéristiques de dimensionnement pour le bois de structure, telles qu’elles sont définies dans la norme européenne EN 14358. Pour l’utilisateur de cette norme cependant, il n’est pas évident d’appréhender le résultat de ce calcul, au sens statistique, dans la mesure où la terminologie appliquée n’est pas explicite. De plus, les bases théoriques du sujet sont rarement traitées dans la littérature. Cet article propose donc de préciser, de manière compréhensible, les paramètres statistiques qui sous-tendent la procédure d’évaluation de la norme EN 14358. Ainsi, les hypothèses statistiques et les fondements mathématiques sont illustrés de manière détaillée. L’analyse montre clairement que ce n’est pas le fractile à 5%, mais une borne inférieure de l’intervalle de confiance de ce fractile, que l’on obtient comme valeur caractéristique.

Les tableaux présentant les paramètres statistiques dans la norme EN 14358 sont plutôt incomplets, et ne spécifient des valeurs numériques que pour quelques tailles d’échantillonnage. Afin de rendre l’application de la norme EN 14358 plus simple et plus précise, des tableaux complets comportant des paramètres statistiques jusqu’à une taille d’échantillons de 100 sont proposés, pour les deux niveaux de confiance appliqués dans la norme.

KEYWORDS: structural timber, characteristic values, transformation of random variables, noncentral t-distribution, quantiles, tolerance intervals
1. INTRODUCTION

In semi-probabilistic design of structures, the proper determination of characteristic values plays a crucial role. For structural timber, the European standard EN 14358 [5] describes a proper evaluation procedure for the determination of characteristic values. However, the terminology applied in the standard is ambiguous as the term “5%-quantile” is used in a misleading manner. Although a footnote provides some explanation, it remains difficult for the user to understand the assumptions on which the statistical factors presented are based, as well as their final effect.

Of course, it is imperative to understand the meaning of such statistical factors before applying them in an evaluation. Hence, the first aim of this paper is to present a comprehensive derivation of the statistical factors as given in EN 14358. Since the tables of this standard provide only a few numerical values, the second aim is to make complete tables available for sample sizes ranging up to 100 specimens and for the two confidence levels proposed in the standard.

2. GENERAL STATISTICAL BACKGROUND

Suppose that X is a normally distributed random variable. In the context here, the aim is to estimate a lower or an upper limit value L so that at least a proportion \( \gamma \) of the population is greater or smaller than L. For example, an upper limit value L with a specified proportion of \( 0.95 \gamma = \gamma \) describes a level at which 95% of the population lie below L. In statistics, such limit values L are often referred to as lower one-sided or upper one-sided tolerance intervals.

In the calculation of such tolerance intervals, three cases generally need to be distinguished [3]:

1. The random variable X is normally distributed with known mean \( \mu \) and known standard deviation \( \sigma \) (\( X \sim N(\mu, \sigma) \), where the symbol “\( \sim \)” is read as “is distributed as”)

2. The random variable X is normally distributed with unknown mean \( \mu \) but known standard deviation \( \sigma \) (\( X \sim N(\bar{x}, \sigma) \))

3. The random variable X is normally distributed with unknown mean \( \mu \) and unknown standard deviation \( \sigma \) (\( X \sim N(\bar{x}, s) \))

The first case is trivial as it leads directly to the determination of the lower or upper quantile of the entire known population, characterized by population
mean $\mu$ and population standard deviation $\sigma$. Nevertheless, it is used in this paper as an introductive example into the subject and to illustrate some basic statistical concepts. The second case which is of remarkable interest in many applications (e.g., quality control) was not accounted for in the European standardization and, hence, is omitted in this paper. In practice, the third case is by far the most interesting one as it reflects the situation most frequently encountered in empirical sciences: from a limited random sample $x_1, \ldots, x_n$, sample mean $\bar{x}$ and sample standard deviation $s$ are obtained and the aim is to estimate a limit value $l$ above or below which a specified proportion $\gamma$ of the population lies with confidence $1 - \alpha$. The third case forms the basis for the determination of the statistical factors presented in EN 14385 and therefore will be discussed in full detail.

3. CALCULATION OF TOLERANCE INTERVALS

The European standard EN 14385 instructs the user to apply the lognormal distribution in order to model the frequency properties of the random variable $X$. However, the following results are independent of the type of normal distribution (normal or lognormal distribution). For sake of simplicity, subsequently only the normal distribution will be considered. In order to avoid any confusion with positive and negative signs, the calculation of an upper tolerance interval will be discussed. Of course, the resulting statistical factors can be used in the same manner to calculate lower tolerance intervals which are of interest when strength properties are concerned.

3.1 Normally distributed random variable $X$ with known $\mu$ and $\sigma$

As an introductive example, a normally distributed population with known mean $\mu$ and known standard deviation $\sigma$ is considered. The upper one-sided tolerance interval $L$, below which the proportion $\gamma$ of the distribution lies, is then determined by the equation

$$L = \mu + K_\gamma \cdot \sigma$$

In order to obtain the appropriate factor $K_\gamma$, the probability

$$Pr \left( L \leq \mu + K_\gamma \cdot \sigma \right) = \gamma$$

needs to be calculated. This equation is equivalent to
\[ Pr \left( \frac{L - \mu}{\sigma} \leq K_{\gamma} \right) = \gamma \]  \hspace{1cm} (2b)

Since the quotient in eq. (2b) is a standardized normal random variable \( ((L - \mu) / \sigma) \sim N(0,1) \), the factor \( K_{\gamma} \) is obtained as the solution of the equation

\[ N \left( 0,1; \gamma \right) := \int_{-\infty}^{x} \frac{1}{\sqrt{2\pi}} e^{-\frac{x^2}{2}} \, dx = \gamma \]  \hspace{1cm} (2c)

The determination of the factor \( K_{\gamma} \) is equivalent to the calculation of the upper quantile \( N \left( 0,1; \gamma \right) \) of the standardized normal distribution. For example, setting in eq. (2c) the proportion \( \gamma = 0.95 \) and solving for the upper integration limit yields the well-known factor \( K_{0.95} = 1.645 \).

### 3.2 Normally distributed random variable \( X \) with unknown \( \mu \) and \( \sigma \)

In practice, the entire population characterized by mean \( \mu \) and standard deviation \( \sigma \) is rarely known. Rather, a limited random sample \( x_1, \ldots, x_n \) with sample mean \( \bar{x} \) and standard deviation \( s \) is given. An upper limit value \( l \), below which a specified proportion \( \gamma \) of the random sample lies, may be obtained in an entirely analogous manner as described in section 3.1

\[ l = \bar{x} + K_{\gamma} \cdot s \]  \hspace{1cm} (3a)

Contrary hereto, the aim is now to perform the statistical inference from the random sample to a specified proportion of the population. In other words, we seek an estimate \( \ell_{1-\alpha} \) for the unknown limit value \( L \) of the population below which the proportion \( \gamma \) lies with confidence \( 1 - \alpha \)

\[ \ell_{1-\alpha} = \bar{x} + k \cdot s \]  \hspace{1cm} (3b)

The estimate \( \ell_{1-\alpha} \) shall be at least equal \( L \) with confidence \( 1 - \alpha \). As probability, this is expressed in the following equation \([1,2]\)

\[ Pr \left( \ell_{1-\alpha} \geq L \right) = Pr \left( \bar{x} + k \cdot s \geq \mu + K_{\gamma} \cdot \sigma \right) = 1 - \alpha \]  \hspace{1cm} (4)

which allows the determination of the appropriate factor \( k \). Equation (4) can be rearranged to read
\[ Pr \left( \frac{-\bar{x} + \mu + K \gamma \cdot \sigma}{s} \leq k \right) = 1 - \alpha \quad (5) \]

In order to perform the calculation of this probability, it is necessary to know how the quotient on the left hand side of the inequality in eq. (5) is statistically distributed. To make that evident, the expression in brackets can be algebraically expanded in an appropriate manner by \( \sigma, \sqrt{n} \) and \( \sqrt{n-1} \) \[ Pr \left( U \leq k \sqrt{n} \right) = Pr \left( \frac{-\bar{x} - \mu}{\sigma} \sqrt{n} + K \gamma \sqrt{n} \sqrt{n-1} \leq k \sqrt{n} \right) = 1 - \alpha \quad (6) \]

Applying the theory of functions of multivariate random variables \([3,4]\) and introducing degrees of freedom \( f = n - 1 \), the quotient \( U \) in eq. (6) can be re-written

\[ U = \frac{-X + \delta}{Y} \sqrt{f} \quad (7) \]

It can be verified by the transformation of a univariate function of one random variable \([3,4]\), that the random variable \( X \) in the numerator of eq. (7) is distributed as a standardized normal distribution \( N(0,1) \) with the probability density

\[ X = \frac{-\bar{x} - \mu}{\sigma} \sqrt{n} \sim f_X(x) = \frac{1}{\sqrt{2\pi}} e^{-\frac{x^2}{2}} \quad (7a) \]

The random variable \( Y \) in the denominator of eq. (7) has a \( \chi \)-distribution with \( f = n - 1 \) degrees of freedom and with probability density

\[ Y = \sqrt{f} \frac{s}{\sigma} \sim f_Y(y) = \frac{1}{\frac{f}{2}^{\frac{f-1}{2}}} \frac{f}{2}^{-1} \Gamma \left( \frac{f}{2} \right) e^{-\frac{y^2}{2}} \quad (7b) \]

where \( \Gamma(x) = \int_0^{\infty} e^{-t} t^{x-1} dt \) denotes the complete Euler Gamma function.

The constant \( \delta = K \gamma \sqrt{n} \) in eq. (7) is termed noncentrality parameter, where \( K_{\gamma} \) is determined as outlined in section 3.1.
According to a famous proof of statistics [4], the sampling functions \( X \) and \( s \) are stochastically independent. Consequently, the transformed random variables \( X \) and \( Y \) are stochastically independent, too. In this case, the joint probability density \( f_{X,Y}(x, y) \) is therefore simply given as the product of the probability densities of \( X \) and \( Y \)

\[
f_{X,Y}(x, y) = f_X(x) \cdot f_Y(y) = \frac{1}{\sqrt{2^{f-1} \pi \Gamma\left(\frac{f}{2}\right)}} e^{-\frac{1}{2} \left(\frac{x^2 + y^2}{f}\right)}
\]

Assuming a sample size of \( n = 10 \), in Fig. 1 the marginal probability densities \( f_X(x) \) and \( f_Y(y) \) (according to eqs. (7a) and (7b)) are plotted qualitatively along the \( x \)- and \( y \)-axes while the joint probability density \( f_{X,Y}(x, y) \) (according to eq. (8)) is shown as contour plot; all these probability densities are displayed in dashed style. Orientating, the both expectation values of the respective marginal probability densities of \( X \) and \( Y \) as well as their point of intersection are plotted.

Now, the probability density of the random variable \( U \) according to eq. (7) is obtained by means of the theory of transforming a multivariate function with two random variables [3,4]. The multivariate transformation rule is given by the equation system (see eq. (7))

\[
u = -\frac{x + \delta}{\sqrt{f}} \quad v = y
\]

The inverse of this transformation rule is found to be

\[
x = -\frac{u}{\sqrt{f}} + \delta = h_1(u, v) \quad y = v = h_2(u, v)
\]

The transformed joint probability density \( f_{U,V}(u, v) \) is then defined by the equation

\[
f_{U,V}(u, v) = f_{X,Y}(h_1(u, v), h_2(u, v)) \cdot |\det J|
\]
where $| \det \mathbf{J} |$ is the determinant of the Jacobi matrix. For the given case, the determinant of the Jacobi matrix results in

$$
| \det \mathbf{J} | = \left| \begin{array}{cc} \frac{\partial h_1(u,v)}{\partial u} & \frac{\partial h_1(u,v)}{\partial v} \\ \frac{\partial h_2(u,v)}{\partial u} & \frac{\partial h_2(u,v)}{\partial v} \end{array} \right| = \left| \begin{array}{cc} \frac{\partial}{\partial u} & \frac{\partial}{\partial v} \\ \frac{\partial}{\partial u} & \frac{\partial}{\partial v} \end{array} \right| = \sqrt{\frac{v}{f}} \sqrt{\frac{u}{f}} = \frac{v}{\sqrt{f}} \quad (11)
$$

Inserting eqs. (9b) and (11) in eq. (10) yields the transformed joint probability density

\begin{align*}
\text{Fig. 1} & : \text{ Marginal probability densities } f_X(x) \text{ and } f_Y(y), \text{ expectation values and joint probability density } f_{X,Y}(x,y) \text{ (dashed style). Joint probability density } f_{U,V}(u,v), \text{ marginal probability densities } f_U(u) = f_X(x) f_Y(y) \text{ and } f_V(v) = f_Y(y) \text{ and expectation values (solid style)}
\end{align*}
\[
f_{U,V}(u, v) = \frac{1}{\sqrt{2^{f-1} \pi f}} \frac{v^{f-1} e^{-\frac{u v}{\sqrt{f}} + \delta}}{\Gamma\left(\frac{f}{2}\right)} v^{f-2} \left(1 + \frac{v^2}{f}\right)^{\frac{f}{2}} \left(1 + \frac{v^2}{2f}\right)^{-\frac{f}{2}}
\]

Assuming again a sample size of \( n = 10 \) and \( K_\gamma = 1.645 \) corresponding to a proportion \( \gamma = 0.95 \) (noncentrality parameter \( \delta = K_\gamma \sqrt{n} = 1.645 \cdot \sqrt{10} = 5.202 \)), the transformed joint probability density \( f_{U,V}(u,v) \) is shown as contour plot with solid lines in Fig. 1, as well.

The probability density of the random variable \( U \) according to eq. (7) is now obtained as the marginal probability density \( f_U(u) \) of \( f_{U,V}(u,v) \)

\[
f_U(u) = \int_0^\infty f_{U,V}(u,v) \, dv
\]

Performing this integration yields the probability density of what is referred to as noncentral \( t \)-distribution. The probability density exists in analytic form and is found to be [3]

\[
f_U(u) = f_\text{t}(t; f, \delta) = \sqrt{\frac{\delta^2}{2}} \frac{e^{-\frac{\delta^2}{2}}}{\Gamma\left(\frac{f}{2}\right)} \left(1 + \frac{u^2}{f} + \delta^2\right)^{\frac{f}{2}} \left(1 + \frac{v^2}{2f}\right)^{-\frac{f}{2}}
\]

with parameters \( f = n - 1 \) (degrees of freedom) and \( \delta = K_\gamma \sqrt{n} \) (noncentrality parameter). Equation (7) in connection with eqs. (7a, b) are called the constructive definition, since these equations define the noncentral \( t \)-distribution in an unique manner.

Under the assumptions given above (\( n = 10, \delta = 5.202 \)), the marginal probability density \( f_U(u) = f_\text{t}(t; f, \delta) = t(f, \delta) \) is plotted qualitatively in Fig. 1 along the u-axis in fat solid style. The second marginal probability density \( f_V(v) \) is plotted qualitatively along the v-axis. Note, that \( f_V(v) = f_Y(y) \) since the transformation rule \( v = y \) (see eq. (9a)) for this coordinate is just an identity and involves no transformation. Orientating, the both expectation values of the respective marginal probability densities of \( U \) and \( V \) as well as their point of intersection are plotted in Fig. 1, again.
Fig. 2: Probability density \( f_U(u) = f_t(t; f, \delta) \), expectation value (dashed vertical line) and confidence level \( 1 - \alpha = 0.75 \) (hatched area)

So far it is proven that the expression \( k\sqrt{n} \) according to eq. (6) is distributed as a noncentral \( t \)-distribution with probability density

\[
k\sqrt{n} \sim t(f, \delta)
\]

(15)

with \( f = n - 1 \) degrees of freedom and \( \delta = K_\gamma \sqrt{n} \) as noncentrality parameter. The quantity \( K_\gamma \) is obtained according to eq. (2c), whereas \( \gamma \) denotes the proportion of the population specified to be at least below the upper limit value \( \ell_{1-\alpha} \) with confidence \( 1 - \alpha \). Again assuming \( n = 10 \) and \( \delta = 5.202 \), the probability density of \( k\sqrt{n} \) is plotted in Fig. 2.

Now, it is easy to evaluate the probability \( Pr(U \leq k\sqrt{n}) = 1 - \alpha \) as given in eq. (6) numerically in dependence of the sample size \( n \) and of the proportion \( \gamma \). In mathematical terms, this is accomplished by solving the probability integral

\[
t(f, \delta; 1 - \alpha) := \int_{-\infty}^{k\sqrt{n}} f_t(t; f, \delta) \, dt = 1 - \alpha
\]

(16)
for the upper integration limit \( k \sqrt{n} \). The calculation performed in eq. (16) is equivalent to the determination of an upper one-sided confidence interval for the quantity \( k \sqrt{n} \) with confidence \( 1 - \alpha \). Figure 2 shows graphically the meaning of this confidence interval where the confidence level is displayed as hatched area (assumed confidence level : \( 1 - \alpha = 0.75 \)). Orientating, again the expectation value of the probability density is plotted as a dashed vertical line.

From eq. (16) immediately follows that the factor \( k \) is obtained as [1]

\[
k_{\gamma, n, 1-\alpha} = \frac{\sqrt{n}}{\sqrt{n}} = \frac{t(f, \delta; 1 - \alpha)}{\sqrt{n}} = \frac{t(f, K_{\gamma} \sqrt{n}; 1 - \alpha)}{\sqrt{n}}
\]

(17)

It can be seen that in the case of unknown mean \( \mu \) and unknown standard deviation \( \sigma \) the factor \( k \) according to eq. (3b) becomes a function of the proportion \( \gamma \), the sample size \( n \) and the confidence level \( 1 - \alpha \). Inserting eq. (17) into eq. (3b) yields the final equation for an upper one-sided tolerance interval

\[
\ell_{1-\alpha} = \bar{x} + k_{\gamma, n, 1-\alpha} \cdot s = \bar{x} + t(f, \delta; 1 - \alpha) \cdot \frac{s}{\sqrt{n}}
\]

(18)

A lower one-sided tolerance interval \( \ell_{\alpha} \), as it is of interest for strength properties, is obtained by inserting the quantile \( t(f, -\delta; \alpha) \) into eq. (18) instead of \( t(f, \delta; 1 - \alpha) \) which results in negative but numerically identical values.

It is interesting to note that, if the noncentrality parameter is set \( \delta = 0 \), eq. (18) becomes identical to the equation for the determination of an upper one-sided confidence interval for the (unknown) population mean \( \mu \) (50%-quantile). In this case, the noncentral \( t \)-distribution simplifies to the more familiar central (Student) \( t \)-distribution \( t(f, 0) = t(f) \) with the upper one-sided quantile \( t(f, 1 - \alpha) \). Therefore, tolerance intervals as discussed in this section must be regarded as one-sided confidence intervals for the quantiles of a normal distribution \( N(\bar{x}, s) \).

This result is in contradiction with the terminology of EN 14358 where it is stated that the “5%-quantile” is the result of the evaluation yet, more specifically speaking, one-sided confidence intervals for the quantiles of a normal distribution with unknown mean \( \mu \) and unknown standard deviation \( \sigma \) are obtained.
4. EVALUATION OF THE STATISTICAL FACTORS

As stated in the introduction, the tables given in EN 14358 are rather incomplete. Only a few statistical factors are given in dependence of sample size. As the gaps between the different sample sizes are rather large, the interpolated statistical factors become correspondingly inaccurate. It is hence desirable, to have tables with numerically precise figures at hand. These tables are the subject of this chapter.

In EN 14358, the proportion as understood in the previous chapters is set \( \gamma = 0.95 \) while two confidence levels are proposed for the application: \( 1 - \alpha = 0.75 \) and \( 1 - \alpha = 0.841 \). The latter confidence level corresponds to the integral of the standardized normal probability density running from negative infinity to one.

Having the proportion \( \gamma \) and the confidence levels \( 1 - \alpha \) defined it is a simple task to evaluate the statistical factors \( k_{\gamma, n, 1-\alpha} \) according to eqs. (16) and (17). For the confidence level \( 1 - \alpha = 0.75 \) the factors \( k_{\gamma, n, 1-\alpha} \) are listed in Table 1 and for the confidence level \( 1 - \alpha = 0.841 \) in Table 2.

5. CONCLUSIONS

In the presented paper, the determination of characteristic values for structural timber based on the European standard EN 14358 was considered. A comprehensive derivation of the statistical factors given in this standard was performed by means of mathematical statistics in order to illustrate the assumptions and the background of these figures. It was found that the terminology of EN 14358 is imprecise in so far as the “5%-quantile” is termed to be the result whereas actually a lower one-sided confidence interval for the 5%-quantile is obtained as result. For a more convenient application of EN 14358, complete tables with statistical factors were provided based on the two confidence levels proposed in the code.
Table 1

Statistical factors $k_{\gamma,n,1-\alpha}$ for the calculation of one-sided tolerance intervals

(Population mean $\mu$ and standard deviation $\sigma$ unknown)

Proportion : $\gamma = 0.95$, confidence level : $1 - \alpha = 0.75$

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Table 2
Statistical factors $k_{\gamma,n,1-\alpha}$ for the calculation of one-sided tolerance intervals
(Population mean $\mu$ and standard deviation $\sigma$ unknown)
Proportion : $\gamma = 0.95$, confidence level : $1 - \alpha = 0.841$

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Of course, the outlined evaluation procedure is not limited to the determination of characteristic values for structural timber. Rather, the procedure is of general validity. As statistics is an exact science, there can be one and only one way to determine one-sided confidence intervals for the quantiles of a normally distributed random variable. The more surprising it is that other European standards (e.g., EN 1058 for wood based panels [6]) propose evaluation procedures which totally deviate from the one discussed here. The validity of these deviating procedures will be investigated in following papers.

ACKNOWLEDGEMENT

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REFERENCES

SUMMARY

Acoustic emissions and ultrasound signals do not always have a high signal to noise ratio. Furthermore, signal and noise are often in the same frequency range. Due to the application of filters, the signal to noise ratio can be improved. As an introduction example of a causal filter the envelope determination of an acoustic emission signal is discussed. Signal conditioning using a FIR filters with a linear phase shift and an anti-causal, zero-phase IIR filters is then discussed. The discrete wavelet-transform and the continuous wavelet-transform are introduced as a further possibility for signal conditioning. The different FIR and IIR filters are compared. This is verified for several applications. The results of the wavelet denoising are also compared to the ones gained by classical filtering. It can be stated that FIR and IIR filters are a stable and reliable tool for signal conditioning of acoustic emissions and ultrasound signals. Wavelet denoising can be of equal quality as classical filters and offers a variety of further applications. However, wavelets should be applied carefully due to the fact that significant artefacts can be created during denoising.

ZUSAMMENFASSUNG

Schallemissionen und Ultraschall Signale haben nicht immer ein hohes Signal-Rausch Verhältnis. Darüber hinaus liegen Signal und Rauschen häufig im selben Frequenzbereich. Mit Hilfe von Filtern ist eine Verbesserung des Signal-Rausch Verhältnisses möglich. Als Einführungsbeispiel zu kausalen Filterfunktionen wird die Berechnung der Envelope eines Schallemissionssignals vorge-

RESUME

Des emissions acoustiques et des signals ultra-son n'ont pas toujours une relation haute entre le signal et le bruit. Signal et bruit sont recurrents en même gamme de fréquence. A cause d'utiliser des filtres de fréquences on peut améliorer la relation entre le signal et le bruit. Pour introduire le principe des filtres de fréquences l'enveloppe d'un émission acoustique est calculée. Après, des filtres de fréquences FIR avec déphasage linéaire et des filtres de fréquences anti-causal sans déphasage ont présente. La transformation wavelet, discrete et continue et aussi présente. Les filtres de fréquences et la transformation wavelet ont comparé aux résultats d'application à des signaux. On peut dire que les filtres FIR et IIR sont stable et digne de confiance en application à des émissions acoustiques et des signals ultra-sons. Filtrar avec la transformation wavelet peut gagner les mêmes résultats que filtrer avec des filtres de fréquences classique et offre des applications supplémentaires. Mais, on doit user la transformation wavelet avec prudence parce qu'on peut générer des artefacts très vite.

KEYWORDS: Acoustic emission, ultrasound, filter function, wavelet-transform
1. INTRODUCTION

Acoustic emissions are defined as the spontaneous release of localized strain energy in stressed material. Due to micro cracking in the material this energy release can be recorded by transducers on the material's surface [Grosse, 2002]. Acoustic emission analysis is capable of revealing damage processes in materials during the entire load history.

![Acoustic Emission Examples](image)

*Fig. 1: Top left: acoustic emission example with a high signal to noise ratio and a clear onset. Top right: acoustic emission example with a medium signal to noise ratio and disturbances. Bottom left: acoustic emission example with a low signal to noise ratio. The high frequency noise hides the onset. Bottom right: acoustic emission example disturbed by a low frequent sinusoidal signal.*

One severe problem in acoustic emission analysis is that huge data sets (often more than 1000 acoustic emission events) with a low signal to noise ratio are detected. Acoustic emission data, e. g. from concrete, normally contains a lot of high frequency noise, mainly caused by the measurement equipment (preamplifier etc.) and the surrounding. Due to the testing process itself a low frequent signal, caused by the testing device (loading machine), may often superimpose the acoustic emission signal additionally. Fig. 1 shows four examples of possible acoustic emission of concrete including different kinds of noise. The signals also differ in frequency content from each other.
Ultrasound signals emitted by an actuator and recorded by a transducer have generally a better signal to noise ratio than acoustic emissions. However, in certain cases, e.g. at the beginning of setting and hardening tests of concrete where concrete is like a fluid paste with solid particles [Reinhardt and Grosse, 2004], the signal to noise ratio of ultrasound signals is low. I.e. if the ultrasound signal is damped and scattered the signal to noise ratio decreases. Fig. 2 shows two ultrasound signals of a setting and hardening test of concrete, one from the beginning with a low signal to noise ratio and one from a later stage where the concrete is already hardened.

![Ultrasound signals](image)

Fig. 2: Top: ultrasound signal transmitted through hardened concrete with a high signal to noise ratio. Bottom: ultrasound signal transmitted through fresh concrete. Notice the low amplitude due to damping of the material.

Concerning further analysis steps, the better the signal to noise ratio the better the results. Therefore, to enhance the signal to noise ratio, filters in a general sense are used. A variety of tools exist. The ones used most frequently are IIR (Infinite Impulse Response) and FIR (Finite Impulse Response) filters [Mathworks, 2000], the wavelet-transform [Percival and Walden, 2002; Misiti et al., 2000] or methods of statistical signal processing [Buttkus, 1991; Mathworks, 2000]. Filters are used in different fields of application where transient signals similar to acoustic emissions and ultrasound signals occur e.g. seismology,
acoustics and optics. However, concerning acoustic emissions signal and noise are often in the same frequency range [Grosse, 1996]. This has to be taken into consideration if filters are applied to such signals.

Furthermore, filters can easily modify a signal in a way which may lead to wrong results, e.g. due to phase shifts and amplitude distortions. The experiences gained about the application of filters on acoustic emission data and on ultrasound signals will be discussed in the following. The influences of the different signal processing procedures will be shown on several examples.

2. APPLICATION OF FILTER FUNCTIONS TO ACOUSTIC EMISSIONS AND ULTRASOUND SIGNALS

Filters are, in the most general sense, devices or algorithms which act on some input signal to produce a output signal [Scherbaum, 1996]. The mathematical foundation of filtering is convolution. I.e. concerning digital signal processing, a filter's output is related to its input by convolution with its impulse response. Applications of this principle in different forms will be shown in the following.

2.1 The envelope of the signal

A relative simple form of signal conditioning is the calculation of the signal's envelope by the Hilbert transform.

The Hilbert transform \( \hat{R} \) of a real time dependent function \( R(t) \) is defined as [Buttkus, 1991]:

\[
\hat{R}(t) = \frac{1}{\pi} \int_{-\infty}^{\infty} \frac{R(t)}{\omega - u} \, du = \mathcal{H}\{R(t)\} 
\]

The Hilbert transform is represented by a convolution integral, i.e. the Hilbert transform is a causal transfer function which behaves like a filter. Transforming a time series by the Hilbert transform, a phase shift of \( \pi/2 \) is generated.

Thus, the envelope time function \( E(t) \) can be calculated [Buttkus, 1991]:

\[
E(t) = \sqrt{R(t)^2 + \hat{R}(t)^2} 
\]

Squaring and norming of the envelope of the signal leads to a suppression of noise of lower amplitude and to an increase of the signal content of higher amplitude (Fig. 3). The high frequency noise is suppressed and the signal is ac-
cented. Then, the envelope can be used to estimate the onset of the signal or for signal detection in general. Calculating the signal's envelope is a fast and simple way and therefore often applied to signal conditioning.

![Graph showing an acoustic emission signal and its envelope](image)

**Fig. 3**: Top: Acoustic emission signal from Fig. 1 (bottom left). Bottom: Squared and normed envelope of the signal calculated by the Hilbert transform.

However, if there are further disturbances on the signal other signal conditioning approaches should be used. Calculating the envelope the way described above is normally free of any errors of mistreating the signal. This cannot be guaranteed for the methods described in the following.

### 2.2 Application of IIR and FIR filters

IIR and FIR filters can be used to erase high frequent noise and/or low frequent disturbances in form of a low-, a high-, a bandpass or a bandstop filter. Such filter functions are the main computational workhorses for classical digital signal processing [MathWorks, 2000]. Before digital signal processing became possible, such filters were already used as analog filters.

The mathematical tool to calculate the discrete transfer function of a discrete system is the z-transform. The z-transform $Y(z)$ of a digital filter's output $y(n)$ is related to the z-transform $X(z)$ of the input by [MathWorks, 2000]:

$$
Y(z) = H(z)X(z)
$$
\[ Y(z) = H(z)X(z) = \frac{b(1) + b(2)z^{-1} + \cdots + b(nb+1)z^{-nb}}{a(1) + a(2)z^{-1} + \cdots + a(na+1)z^{-na}} X(z) \quad (3) \]

where \( H(z) \) is the filter transfer function and \( z \) is a continuous complex variable. The constants \( b(i) \) and \( a(i) \) are the filter coefficients. The order of the filter is the maximum of \( na \) and \( nb \). Even if there are several exceptions as a rough guideline it can be said:

- \( nb=0 \) means, that the filter is an IIR, all-pole, recursive filter
- \( na=0 \) means, that the filter is a FIR, all-zero, nonrecursive filter
- \( na > 0 \) and \( nb > 0 \) means, that the filter is an IIR, pole-zero, recursive filter

Scherbaum [1996] summarized the main characteristics of FIR and IIR filters as follows:

- FIR filters are always stable. However concerning steep filters many coefficients are needed. Filters with given specifications such as linear phase or even zero phase can easy be implemented.
- Steep IIR filters can easiliy be implemented with a few coefficients. Therefore, filtering is very fast. However IIR filters are potentially unstable and given specifications such as zero phase are difficult to implement.

Concerning digital signal processing a variety of tools e.g. in Matlab [MathWorks, 2000], LabVIEW [National Instruments, 2004] or for free in the world wide web [Mathtools, 2004; MathWorks, 2004] are available. Furthermore, the Numerical Recipes [Numerical Recipes, 2004] for different programming languages and the DSP group [DSP group, 2004] provide free filter algorithms. The corresponding adresses in the world wide web can be found in the references. This list is not exhaustive. This is only a compilation of resources which I frequently use for solving problems in digital signal processing of acoustic emissions and ultrasound signals. Several of these sources of information about filter functions also contain descriptions and algorithms of the wavelet-transform which will be discussed in section 2.3.

The signals shown in Fig. 1 were used for demonstrating the capabilities of the different filter functions. The software tool used here was Matlab. FIR filters with a linear phase shift and anti-causal, zero-phase IIR filters (in the following called IIR filter) were used an compared.
The FIR filter is a causal filter and produces therefore a linear phase shift of \( n/2 \) where \( n \) denotes the filter order, e.g. \( n=100 \) means a shift of 50 samples. The anti-causal, zero-phase IIR filter produces no phase shift. However, in general the amplitude is smaller than the one of an FIR filtered signal. Fig. 4 (middle) shows the already phase corrected FIR filtered section of the signal around the onset (dotted line) of Fig. 1 (top right) and the the same section of the original section (solid line). The non-filtered signal is shifted 0.1 V upwards to allow a better comparison of the two waveforms. The FIR filter does not distort the original waveform. The shape and the amplitude of the signal are kept. The curve is smoothed according to the filter characteristics.

Fig. 4: Top: acoustic emission signal from Fig. 1 (top right). Middle: Kaiser window FIR bandpass filtered signal (corner frequencies: 5 kHz and 300 kHz) Bottom: Comparison of onset region filtered by the FIR bandpass filter (dotted line) and an anti-causal, zero-phase IIR bandpass filter with the same corner frequencies (solid line). The IIR filtered signal was artificially shifted by 0.1 V to allow a better comparison of the filtered signals.

The comparison of the FIR and the IIR filtered signal (Fig. 4, bottom) shows that both filters are equal in quality. I.e. none of them produces a non-linear phase shift and the amplitude damping of the IIR is low compared to the FIR.
The capabilities of the FIR and the IIR bandpass filter are confirmed by results of the FIR and IIR lowpass filter (Fig. 5). The amplitude differences between the FIR filtered signal and the IIR filtered one are very small (Fig. 5, middle and Fig. 5, bottom). Again non-linear phase shift is not observable.

![Graphs showing signal processing](image)

*Fig. 5: Top: Acoustic emission signal (see also Fig. 1, bottom left) with a low signal to noise ratio. Note the low amplitude. Middle: lowpass (corner frequency: 150 kHz) filtered signal using a Kaiser window FIR filter. Bottom: lowpass (corner frequency: 150 kHz) filtered signal using a Butterworth IIR filter.*

Filtering the Signal shown in Fig. 1 (bottom right) with a FIR and an IIR highpass filter shows differences in the amplitudes of the FIR and the IIR filtered signal (Fig. 6). This highpass filtered signal is an example for possible effects of an IIR filter compared to a FIR filter. Especially the coda of the IIR filtered waveform in Fig. 6 (bottom) shows significant lower amplitudes than the coda of the FIR filtered waveform (Fig. 6, middle). A detailed look at the main part of the signal also shows this effect. However, again no nonlinear phase shift could be verified.
Fig. 6: Top: Acoustic emission signal disturbed by relative low frequent noise (see also Fig. 1, bottom right). Middle: highpass (corner frequency: 15 kHz) filtered signal using a Kaiser window FIR filter. Bottom: highpass (corner frequency: 15 kHz) filtered signal using a Butterworth IIR filter.

2.3 The Discrete wavelet-transform

Wavelets are mathematical functions that have to be well localized (other requirements are of technical matter). The mathematical steps used for the wavelet transform can be summarized as follows:

i. a fully scalable modulated window solves the signal cutting problem

ii. this window is shifted along the signal and for every position the spectrum is calculated

iii. the process is repeated many times with a slightly shorter or longer window for every new cycle

iv. this results in a collection of time-scale representations (scale is proportional to frequency) of the signal, all with a different solution

These four points represent the principle of the wavelet transform. General concepts known from Fourier analysis form the application base. Therefore, the basic concepts of convolution and filtering of finite sequences and some basics
about orthonormal functions are needed. The mathematical concept presented in the following is adopted from Percival and Walden [2002].

The DWT is an orthonormal transform of the form:

\[ W = \mathcal{W}X \]

The first two points of the above enumeration show that a periodized filter is needed which is indeed the wavelet. The \( N \times N \) matrix \( W \) consists of these periodized filters. The third point indicates that the filter is rescaled for every new cycle. That means the matrix \( W \) consists of the wavelet and the scaling filter. The wavelet filter is a high pass filter with a nominal pass-band while the scaling filter is a low pass filter with a different nominal pass-band. Applying them on the time series \( X \), the Wavelet coefficients \( W \) are gained which are a collection of time-scale representations of the signal, all with a different solution. In other words: the wavelet is scaled and shifted and then moved along the time series. Therefore, the wavelet-transform is essentially a bandpass filter of uniform shape and varying location and width [Torrence and Compo, 1998].

The main difference between the discrete and the continuous wavelet transform (CWT) is that due to redundancies in the CWT the DWT can be thought of as a subsampling of the wavelet coefficients with dyadic scales. That means each vector of \( W \) contains \( 2^j \) elements, \( j = 1, \ldots, J \). Each element is one wavelet coefficient. The rows of \( W \) that produce the wavelet coefficients (the wavelet filter) for a particular scale are circularly shifted versions of each other. The amount of the shift between adjacent rows is \( 2^j \) for \( j = 1, \ldots, J \). Furthermore, changes concerning the wavelet scale (the scaling filter) are also of dyadic order in the same range as for the wavelet filter. The DWT is computed using the Mallat algorithm that is faster than the fast Fourier transform. Nevertheless, a time series can be perfectly reconstructed from its DWT coefficients. This is an advantage of the DWT compared to the CWT.

A detailed description of the wavelet theory can be found in Percival and Walden [2002]. A broad compilation of articles, tutorials and links can be found in the world wide web at the homepage of Amara Graps [2004]. Beside the commercial wavelet tools of Matlab and LabVIEW there is also a free wavelet toolbox available: Wavelab 802 [2004].

Using the wavelet-transform for denoising requires an adopted model for the current noise of the dataset. Concerning the example shown in Fig. 7 a bior-
thogonal wavelet of the order 4.4 was wavelet-transformed using a level 8 DWT. The applied denoising algorithm was Stein's Unbiased Risk Estimator with hard thresholding for an unscaled white noise model. Furthermore, the approximation coefficient of level 8 was set to zero to erase the offset of the signal (Fig. 7, middle). The wavelet denoised signal in Fig. 7 (bottom) was artificially shifted by 0.2 V to allow a better comparison. The direct comparison of the wavelet denoised signal and the FIR filtered signal (Fig. 7, bottom) shows only slight differences in amplitude smoothing. I.e. the FIR filter produces a smoother curve. Furthermore, the comparison shows that no phase shift is observable. This is guaranteed due the use of biorthogonal wavelets.

Fig. 7: Top: acoustic emission signal from Fig. 1 (top right). Middle: wavelet denoised signal using a biorthogonal wavelet of the order 4.4. Bottom: Comparison of the wavelet denoised signal (solid line) to the FIR filtered signal already shown in Fig. 4. Only the region around the onset of the signal is shown. The wavelet denoised signal was artificially shifted by 0.2 V to allow a better comparison of both signals.

Choosing a different noise model for wavelet denoising can lead to significant different results. Fig. 8 shows a comparison of the bandpass FIR filtered signal (dotted line) already used for comparison in Fig. 7 (bottom) and the corresponding wavelet denoised version of the signal differing only in the chosen noise model from the wavelet procedure described above. Instead of using un-
scaled white noise for thresholding non-white noise was used. The result differs significantly from the one shown in Fig. 7. The similarity between the FIR band-pass filtered signal and the wavelet denoised signal from Fig. 7 (bottom) could not be maintained with the non-white noise threshold. The amplitude of the wavelet denoised signal is damped and distorted. Due to the non-white noise thresholding, several artefacts also occurred in the shape of the signal. E.g. the area right in front of the onset of the signal changed completely its shape. Furthermore, the structure of the noise is completely different. This makes clear that if choosing the wrong model of noise for thresholding the important characteristics of the signal can be conditioned badly. Then, e.g. a wrong onset time of the signal will be determined.

![Fig. 8: Comparison of the FIR filtered signal already shown in Fig. 4 (dotted line) and the corresponding wavelet denoised signal. The area around the onset of the signal is displayed. The change compared to the denoising shown in Fig. 7 is that the noise was modelled as non white noise.](image)

2.4 The Continuous wavelet-transform

The principal ideas behind the DWT and CWT are identical. Therefore, the possibilities and the procedures concerning filtering and denoising of acoustic emissions and ultrasound signals do not differ from each other. The continuous
The continuous wavelet-transform also means continuously shifting a continuously scalable function $\psi_{\lambda,\nu}$ over the signal and calculating the correlation between the two. The discrete sequence $R(t)$ is decomposed into a set of basis functions $\psi_{\lambda,\nu}$, called the wavelets. Thus, $\lambda$ denotes the scale (scale is proportional to frequency) and $\nu$ the translation. The discrete sequence $R(t)$ is decomposed into a set of basis functions with the new dimensions $\lambda$ and $\nu$. Thus, all scales are accounted for the transform. The CWT gives an high resolution image of the frequency distribution over time. Therefore, it is much slower than the DWT or a short-time Fourier transform. However, if there is no idea about the frequency distribution over time of the important parts within the investigated signal the CWT is a helpful tool to make clear the frequency changes over time.

Examples of application concerning filtering and denoising of acoustic emissions and ultrasound signals with the CWT can be found in Ruck and Reinhardt [2002, 2003].

The CWT offers the possibility to apply a complex transform using complex wavelets. One possible application will be given in the following.

Since the complex continuous wavelet transform is a complex valued orthonormal transform represented by a convolution integral, the modulus of one scale of the complex continuous wavelet transform represents the envelope of a signal at one certain frequency.

$$|W(\lambda, \nu)| = \sqrt{x^2 + y^2} \quad \text{where}, \quad W(\lambda, \nu) = x + iy$$

The envelope calculated by the complex continuous wavelet-transform (Fig. 9) could also be calculated by filtering the original signal with a bandpass filter and then calculating the envelope using the Hilbert transform. However, the advantage of the complex continuous wavelet transform is that the filter bonds do not need to be known in advance. The corresponding scale can be chosen instead.
Fig. 9: Top: Acoustic emission signal from Fig. 1 (bottom left). Bottom: Squared and normed envelope of the signal calculated from one scale of the complex continuous wavelet-transform.

3. CONCLUSION

Signal conditioning is a crucial part during data analysis, due to the fact that mistakes there have got a severe impact on the results gained during the further analysis. Furthermore, concerning acoustic emissions or ultrasound signals signal and noise are often in the same frequency range [Grosse, 1996]. These facts require stable and reliable algorithms for signal conditioning.

Calculating the envelope using the Hilbert transform is a descriptive form of filtering due to the fact that the Hilbert transform is a causal transfer function which behaves like a filter and produces a phase shift of $\pi/2$. Squaring and norming of the envelope suppresses low amplitude noise. Using the complex continuous wavelet-transform leads also to the envelope, however only of a certain frequency.

The applied FIR filters are also causal filters and produce a linear phase shift of half the filter order, i.e. if the filter order is 100 the signal is shifted 50 samples. The filtered sequence has to be corrected by this number of samples.
FIR filters from the used Matlab package [MathWorks, 2000] do not cause any non-linear phase shift or amplitude distortion. Therefore, they are a reliable tool for signal conditioning. IIR filters normally cause highly nonlinear phase distortions. However, using an anti-causal, zero-phase filter implementation of an IIR filter the nonlinear phase distortions are corrected. This cannot be guaranteed for all cases. Regarding the examples shown in Fig. 4 to 6 no nonlinear behaviour of the anti-causal, zero-phase IIR filter could be verified. A further requirement for the successful use of such IIR filters is that the filtered signal has a length of at least three times the filter order and tapers to zero on both edges [MathWorks, 2000]. Nevertheless, the anti-causal, zero-phase IIR filter can cause amplitude damping. This could only be verified for the highpass filter (Fig. 6).

The wavelet-transform is essentially a bandpass filter of uniform shape and varying location width [Torrence and Compo, 1998]. Denoising by thresholding can produce as good results as a bandpass filter (Fig. 7) if the correct noise model was chosen. Furthermore, the denoising technique has the advantage over traditional filtering in that it removes noise at all frequencies and can be used to isolate single events that have a broad power spectrum or multiple events that have varying frequency [Torrence and Compo, 1998]. However, if the noise was classified wrong, i.e. if the the wrong noise model was chosen the denoising procedure can produce significant artefact. E.g. artificial signal onsets are created. The Stein's Unbiased Risk Estimator algorithm is able to chose the thresholds for every level adaptively. The selection rules are more conservative and are more convenient when small details of signal lie in the noise range. However, if the wrong noise classification was chosen wavelet denoising will produce significant artefacts which will lead to wrong results.

Finally it has to be stated that signal conditioning is not able to improve the signal to noise ratio of all signals. Fig. 2 (bottom) is an example for the case where signal conditioning would not be successful. It is not not possible to valuate the results of signal conditionig if the signal has got such a low amplitude and is heavily distorted in a way that makes the separation of signal and noise impossible. In such cases an improved recording procedure is recommended.

ACKNOWLEDGEMENTS

This work was carried out in the collaborative research centre SFB 381 at the University Stuttgart whose financial support by the Deutsche Forschungsgemeinschaft is gratefully acknowledged.
REFERENCES


SUMMARY

Nowadays sustainability for both old and new buildings becomes more and more important. Therefore non-destructive test methods are of high interest. Non-destructive test methods are used for quality control of construction materials, inspection and analysis of materials or structures and are used for continuous monitoring of large constructions. A multitude of characteristic values of building structures is measured with conventional wired sensor systems nowadays, e.g. moisture, temperature and stress, strain or displacement under static or dynamic load. Modal analysis or acoustic emission analysis is used to characterize the condition of the structure. In the future non-destructive test methods in combination with structural health monitoring techniques will help to better understand the structural behaviour and to better predict the remaining lifetime.

The presented paper provides a short insight into the potentiality of structural health monitoring using wireless sensor networks, which is under development at the Institute of Construction Materials. Such monitoring systems are easy to install and inexpensive, that enlarges its application.

ZUSAMMENFASSUNG

Dem Bauwerksmonitoring, das heißt der fortlaufenden Überwachung von Bauwerken mittels geeigneter Technologien, kommt insbesondere im Zusammenhang mit einem zunehmenden Alter von Bauwerken wie auch ansteigender Anforderungen an Tragfähigkeit und Dauerhaftigkeit eine immer größere Bedeutung zu. Bisher wird an einzelnen ausgewählten Bauwerken bereits eine

Der vorliegende Beitrag gibt einen Einblick in neue Monitoringtechnologien mit drahtlosen Sensornetzwerken, die momentan am IWB entwickelt werden. Derartige Monitoringsysteme sind gegenüber konventionellen Messsystemen einfacher zu applizieren und deutlich kostengünstiger, was eine breitere Anwendung in der Praxis ermöglicht.

RESUMÉ

Une grande nombre des qualités characteristiques des constructions uniques sont enregistrée avec d’apparaillage conventionelle. Ce sont par example la temperature d’air, la temperature de la construction, l’humidité, l’élongation et la déplacement d’une structure et les vibrations. Une analyse modale ou une analyse d’émission acoustique peut être faite. Dans l’avenir les qualités qui sont enregistrés par des methodes non-destructives contribueront à prédir des changements de l’état d’ouvrage d’art et la durée de vie restante.

La contribution suivante donne un aperçu de la technologie de la systèmes sans fil. Ces systèmes sont faciles à appliquer et côutes moins que les systèmes conventionnels.

KEYWORDS: Structural health monitoring, wireless sensor networks, wireless communication, MEMS, low power sensors

INTRODUCTION

The demand for non-destructive test methods and monitoring techniques increases rapidly in Europe. For maintenance purposes it is more and more recommended not only to inspect building structures like bridges at certain time intervals. Moreover there is the demand for continuous monitoring techniques. This problem becomes very important at railway bridges. In this context a Euro-
Several recent damages on bridges have lead to the installation of wired monitoring systems to analyse the structural behaviour and the deterioration processes. However, such monitoring systems use standard sensor technologies and several other devices which are time consuming to install and expensive. For example one force balanced acceleration sensor for modal analysis costs about 3000 to 5000 Euros and data acquisition and cabling for each sensor about 1000 to 2000 Euros. That means that conventional monitoring systems consisting of a larger amount of sensors are high-priced expensive and therefore will be installed on a few bridges only.

A wireless monitoring system with MEMS sensors could reduce these costs dramatically. Microelectromechanical systems (MEMS) are small integrated devices or systems that combine electrical and mechanical components that could be produced at much less than 50 Euro each. A wireless sensor mote with such a MEMS sensor could then be fabricated at a price varying from 100 to about 400 Euro each. As a result the use of monitoring systems equipped with MEMS sensors and wireless communication could enormously decrease the costs to only a small percentage of a conventional monitoring system and therefore will increase its application not only in monitoring bridges.

Fig. 1: Diagram of wireless sensing of large structures using radio frequency transmission technique [2]
However, if a larger amount of bridges are equipped with a wireless monitoring system, the demand for its visual inspection decreases as well and due to the detailed information of the structural behaviour of bridges obtained from the monitoring system, maintenance costs could be reduced. Only if certain changes in the structural behaviour are recognised and damages will become obvious, a repair is necessary. This repair could be done right after the occurrence of the defect what reduces the risk of consequential damage. The analysis of the measured data and the knowledge of the continuous changes of structural behaviour will also improve the lifetime prognosis of the building structure. This especially could reduce the overall maintenance costs of buildings and transport networks.

LAYOUT OF A WIRELESS MONITORING SYSTEM

A wireless monitoring system consists of several different components that will be described in the following (see Fig. 2). A monitoring system should provide relevant data from the observed structure without the requirement to inspect it. So the data has to be transmitted in a sufficient way to the user, e.g. through the internet. The sensor motes have to be power and cost optimized so they can provide data only at small distances. For that reason there is the demand to install a central processing unit on site in addition to the installation of the sensor motes. This central unit has to collect, to store the data in a database and further to analyse the data from the sensor motes until this data is requested by the user or until a sudden event is detected which results in an alarm message. The central unit also should allow a calibration and a wireless reprogramming of the sensor nodes to keep the whole system flexible. The central unit could be a conventional personal computer equipped with a constant power supply and specific hard- and software.

![Fig 2: Schematic diagram of a wireless monitoring system](image-url)
THE SENSOR MOTE

The sensor motes are the main components of a wireless monitoring system. There are different tasks a sensor mote has to perform, which are to collect and digitize data from different sensors, to store sensor data, to analyse data with simple algorithms, to send and receive selective and relevant data to and from other nodes as well as the central unit and to work for an adequate time period without a wired power supply. Therefore a sensor mote consists of a CPU or DSP with sufficient memory, a low power radio, an aligned analog to digital conversion module (ADC), a power supply and one or more diverse sensors.

![Sensor Mote Diagram]

Fig. 3: Schematic layout of a sensor mote and prototype (©Smartmote)

Power consumption and power supply

A monitoring system is supposed to work for a longer period that means for several month or years. A detailed bridge inspection will take place at an interval of three or six years in Germany for example. It is therefore desired, that the lifetime of the monitoring system is also three years at least. Power consumption of the mote smaller than 2 mW is recommended and in order to achieve a maximum lifetime an effective power supply has to be chosen.

An important aspect in designing and in programming a sensor node is to minimize its overall power consumption [3]. As a first step it is recommended to look for an optimized hardware. There are a lot of power consuming components like the sensors, the A/D-conversion-module, the radio module, the sensor-CPU, and the memory which require energy to work properly (see table 1). If low power consumption is considered it is suggested to limit the voltage range of these components to a maximum value of 5 Volt or better of about 3 Volt or even lower.

In a next step it is recommended that the system components operate in sleep or power down mode as often as possible. These modes require only little
energy. Most of the used radio modules and processors support such power saving modes. Some of the devices could also be switched off if not needed. For this reason a monitoring system has to provide different event handling and wake up modes. Hard- and software has to be optimized in this way.

At this juncture the wake up time itself could be of importance. Especially if the measurement task is acoustic emission analysis a very fast wake up time of a few µs is required for some of the components. For example a localisation of an acoustic event in a concrete structure with an accuracy of about 0.25 m needs accuracy in the signal onset detection of about 60 µs [2] [4]. However, such a fast wake up time could be provided only by a few commercially available components up to now. That means that further developments are necessary.

Table 1: Average energy required for different components (at 3V)

<table>
<thead>
<tr>
<th>Component</th>
<th>Sleep mode</th>
<th>Full operation</th>
</tr>
</thead>
<tbody>
<tr>
<td>8bit-Processor @20MHz</td>
<td>24 µW</td>
<td>24 mW</td>
</tr>
<tr>
<td>Memory</td>
<td>6 µW</td>
<td>45 mW (writing)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>12 mW (reading)</td>
</tr>
<tr>
<td>Radio module (RF)</td>
<td>6 µW</td>
<td>24 mW (receiving)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>36 mW (transmitting)</td>
</tr>
<tr>
<td>Signal conditioning and A/D-conversion</td>
<td></td>
<td>0.6 to 2 mW</td>
</tr>
<tr>
<td>100kHz, 12 bit</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>MEMS-Sensors</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Acceleration 2kHz, 12 bit</td>
<td>15 µW</td>
<td>6 to 15 mW</td>
</tr>
<tr>
<td>Humidity &amp; Temperature</td>
<td>1 µW</td>
<td>1.5 mW</td>
</tr>
</tbody>
</table>

Another high power consuming part is represented by the sampling rate and the amount of handled data, e.g. high sampling rates and high amplitude resolution result in high power consumption. In fact power consumption of temperature or humidity measurements is not a problem. Moreover it becomes of interest to monitor dynamic behaviour or uses acoustic emission techniques with high sampling rates.

Nowadays battery powered systems, e.g. equipped with Lithium-batteries, are most appropriate. However, power supplies like solar cells, methanol powered fuel cells are alternatives. Ongoing research is made in the field of energy harvesting, for example for cellular phones. Therefore it is not the intention of this paper to review problems concerning power supplies.
Wireless communication

It is suggested that for most monitoring tasks a communication range of 10 to 30 m is sufficient. There are some communication standards like WLAN (IEEE802.11, wireless local area network) and Bluetooth™ that are well known and feature such ranges. WLAN describes a family of specifications for wireless local area networks and offers high data rates up to 54 Mbps at distances of up to a hundred meters and is low power consuming if one looks at the energy consumed per transferred bit (see table 2). However the IEEE802.11 standard does not feature power management. If only a few data bits have to be transmitted, IEEE802.11 need much more power and becomes inefficient. Bluetooth™ offers up to 1 Mbps to a maximum distance of about ten meter and is equipped with several power saving mechanisms. Bluetooth™ works with frequency hopping to eliminate interfering signals and uses a complex protocol stack that needs system memory of about 200 Kbytes [5]. However, Bluetooth™ is optimized for widespread applications and for a larger amount of data so a lot of different functions are embedded. There are some new developments like Zigbee™ (IEEE802.15.4), which work with fully handshaked protocols for transfer reliability and also permit power management to ensure low power consumption. Advantageous is the much smaller protocol stack of 4 to 32 Kbytes than that of Bluetooth™. The Zigbee™ standard is optimized for intermittent data transfer so the overall power consumption is low if only few data have to be transmitted in between a longer time period.

To further reduce the power consumption several working groups have developed their own proprietary RF communication devices using low power radio transceiver in the free ISM frequency bands. Such a proprietary RF solution is the broadband nanoNET radio communication which works in the 2.4 GHz range and uses CSS (Chirp Spread Spectrum) functionality [6]. The nanoNET technology has a small protocol stack and offers higher data transfer rates and higher reliability than the Zigbee™ standard but requires less energy [7]. However, this technique is still under development right now. Also other communication techniques are still under investigation like ultrasonic or optical methods, but these methods are described elsewhere [7].
Table 2: Average energy required per transmitted bit and maximum data transfer rate

<table>
<thead>
<tr>
<th>Communication standard</th>
<th>12 m distance</th>
<th>30 m distance</th>
<th>Max. Transfer rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>IEEE 802.11b (WLAN)</td>
<td>200 nJ/bit</td>
<td>300 nJ/bit</td>
<td>11 Mbps</td>
</tr>
<tr>
<td>Bluetooth™</td>
<td>2.5 µJ/bit</td>
<td>-</td>
<td>0.8 Mbps</td>
</tr>
<tr>
<td>Zigbee™</td>
<td>7 µJ/bit</td>
<td>7 µJ/bit</td>
<td>20 to 250 kbps</td>
</tr>
<tr>
<td>Home-RF (example)</td>
<td>1 µJ/bit</td>
<td>2 µJ/bit</td>
<td>0.8 to 2 Mbps</td>
</tr>
<tr>
<td>nanoNET (CSS)</td>
<td>60 nJ/bit</td>
<td>80 nJ/bit</td>
<td>2 Mbps</td>
</tr>
</tbody>
</table>

With regard to the power consumption of the different communication methods for saving energy it is suggested to send relevant data to other sensor nodes or the central unit only, because signal processing in the sensor node needs less power than sending it through the radio module.

Data acquisition and data aggregation

Before data processing start they must be converted from analogue to digital. This is done by an A/D-conversion-module. Due to the power consumption the sampling rate and the amplitude resolution has to be customized for the measuring task.

As shown above it is recommended to minimize the data transfer through the radio module which means that signal processing and data analysis should be done in the sensor mote as far as possible. The signal analysis could be made by software or by special hardware, e.g. Baas [8] has built a fast Fourier transform (FFT) implemented directly into hardware that can perform a 1024-point FFT at 3.1 µJ using a 1.1 V supply. Only relevant sensor data should be transmitted to other motes or the central unit, e.g. maximum or minimum values. This is also in general needed to minimize the data traffic that is of interest if the monitoring system consists of a high amount of motes.

Due to interference by other electro-magnetic fields the data transfer could be disturbed and data are lost. For reliability reasons data transfer redundancy is required under certain circumstances. One solution is the storage of a set of data in the sensor mote sending it consecutively through the radio module at specific time intervals, events or on request.
Robustness and Reliability

The whole monitoring system, which has to be installed on site, has to withstand rough climate and other conditions (see table 3). For example it has to be resistant against oil, fuel, salt and alkali. The sensors have to be robust and durable such that their measured data is reproducible and reliable over the monitoring period. Furthermore the system stability, which includes the wireless data transfer to and from the sensor nodes, should be high.

Table 3: System specifications

<table>
<thead>
<tr>
<th></th>
<th>Unit</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temperature</td>
<td>°C</td>
<td>-30 ÷ 80</td>
</tr>
<tr>
<td>Relative humidity</td>
<td>%</td>
<td>10 ÷ 100</td>
</tr>
<tr>
<td>Shock</td>
<td>G</td>
<td>1000</td>
</tr>
</tbody>
</table>

MEASUREMENT PROGRESS

Structural health monitoring implies a continuous measurement of several values over a long period of time. Thus continuous or rather discrete monitoring has to be more specified. For example it is not feasible to measure humidity or temperature every second or minute and a continuous monitoring of acoustic events at high sampling rates is not appropriate due to the large amount of data. It is therefore necessary to divide monitoring into discrete and event based monitoring, which is similar to data aggregation on request or on the other hand event based data aggregation.

Discrete monitoring

Discrete Monitoring is a question of the time interval between two measurements. A time signal triggers the measurement progress and is therefore some kind of an event. However, time based monitoring is different from event based monitoring because time is a global value and not specific data of the observed object. To reduce the amount of data it is recommended to choose time intervals for measurements like they are listed in table 4 for example.
Table 4: Measurement intervals with respect to discrete monitoring

<table>
<thead>
<tr>
<th>Measured Values</th>
<th>Unit</th>
<th>Time interval</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temperature</td>
<td>Hours</td>
<td>1 to 24</td>
</tr>
<tr>
<td>Relative humidity</td>
<td>Hours</td>
<td>1 to 24</td>
</tr>
<tr>
<td>Temperature induced strain, stress or displacement</td>
<td>Hours (Days)</td>
<td>1 to 24 (1 to 365)</td>
</tr>
<tr>
<td>Corrosion</td>
<td>Month</td>
<td>Any</td>
</tr>
<tr>
<td>Chemical Attack</td>
<td>Month</td>
<td>Any</td>
</tr>
<tr>
<td>pH-value</td>
<td>Month</td>
<td>Any</td>
</tr>
</tbody>
</table>

Event based monitoring

Event based monitoring is useful if temporary loads or other effects influence the structure, e.g. trains, trucks, wind, snow or rain, earthquakes or structural failure itself. That means that an object specific event triggers the measurement progress. Some of these special events are listed in table 5. If an event based monitoring is chosen, it has to be considered that every measured value has to get its time stamp, because in this connexion time is a value to be measured.

Table 5: Aspects of interest caused by special events

<table>
<thead>
<tr>
<th>Aspects of interest</th>
<th>Caused by (Event)</th>
<th>Measured Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temperature change</td>
<td>Climate</td>
<td>Temperature</td>
</tr>
<tr>
<td>Change in relative humidity</td>
<td>Climate</td>
<td>Humidity</td>
</tr>
<tr>
<td>Rupture of steel tendons</td>
<td>e.g. corrosion, temperature, static or dynamic loads</td>
<td>Acoustic events</td>
</tr>
<tr>
<td>Concrete cracking</td>
<td></td>
<td>Change in stress, strain or displacement</td>
</tr>
<tr>
<td>Fatigue of steel connections</td>
<td></td>
<td>Change of dynamic behaviour</td>
</tr>
<tr>
<td>Maximum/medium/minimum absolute strain, stress or displacement</td>
<td>e.g. climate, loads, wind</td>
<td>Stress, strain or displacement</td>
</tr>
<tr>
<td>Structural behaviour due to dynamic loads</td>
<td>e.g. special train, wind or earthquakes</td>
<td>Acceleration, Eigenmodes</td>
</tr>
<tr>
<td>Structural behaviour due to static loads</td>
<td>e.g. special load</td>
<td>Stress, strain or displacement</td>
</tr>
<tr>
<td>Structural failure</td>
<td>any (e.g. corrosion, earthquakes, accidents)</td>
<td>any</td>
</tr>
</tbody>
</table>
Sensors for wireless monitoring

To achieve a long working time of the sensing unit passive sensors seem to be the best choice. They do not require electric power, because they obtain their energy directly from the change of physical values. For example piezoelectric materials are such materials.

But also active sensors could be of interest. Active sensors are defined as sensors which necessitate electric power to work properly. MEMS sensors are active sensors for instance. Most of the MEMS sensors are featured with an integrated signal conditioning, sufficient A/D-Converter and/or power down features and therefore require certain electric power. It could be of advantage if an A/D-Converter is integrated into the sensor, because it is often optimized for the measuring task as well as for low power. Some MEMS have also integrated temperature compensation. Furthermore, most of the MEMS are equipped with an event handler so a fast wake up of the system is guaranteed if an ascertain trigger is reached and the MEMS are in idle or sleep mode. However, in some cases like for acoustic emission analysis the wake up time of the commercially available MEMS is so long that important signal data is lost.

The temperature and the moisture of the air and of the structure are general values, which are relevant for almost all measurement tasks. There are a lot of commercial standard and also MEMS-based sensors available which might fit the requirements. A MEMS sensor embedded into a concrete structure is shown in Fig. 4.

Concerning modal analysis the required specifications strongly depend on the type of monitored structure. Mainly bridges or towers are of interest. For very stiff structures like concrete bridges a bandwidth of 0.1 to 20 Hz is of im-

Fig. 4: Prototype of a wireless mote with connected moisture and temperature sensor
portance. If only structural parts like steel beams or steel cables are analyzed, the Eigenmodes are in a higher frequency range and the interesting bandwidth will be up to 500 Hz in some cases. Acceleration, velocity and the sensitivity depends on these frequencies so the use of one sensor for all the purposes is not appropriate.

If an acoustic emission analysis is considered one has to distinguish between laboratory tests and monitoring on site. Due to power considerations high sampling rates greater than 100 kilo samples per second are not recommended for wireless monitoring on site, so one has to look for a compromise. It is therefore not needed to use sensors with very high frequencies. Broadband transducers with a bandwidth of 3 to 100 kHz seem to be appropriate. However, the needed sensitivity strongly depends on the application and the energy content of the monitored events respectively. MEMS based capacitive sensors as well as conventional piezoelectric sensors could be used for wireless monitoring. Nevertheless the sampling rate and the amplitude resolution and not of the type of the used sensor is of main influence for the power consumption.

A high amount of strain gauges are in practical use to measure strain and also stress under static or dynamic loads. The current of strain gauges is small and therefore power consumption is only of a few mW, so that strain gauges could be used with wireless motes, too. In most applications stress is calculated indirectly from the measured strain. However, creep of the strain gauges and of the used adhesives to apply them to the structure is still a problem if one will measure for long time periods. Temperature compensation has to be done with all gauges. But due to available proper compensation methods this is not a real problem. Instead of strain gauges also LVDT could be used to measure displacement and thus indirectly strain. On the other hand a LVDT is much more expensive than a strain gauge.

For force measurement commercial pressure cells as well as low power MEMS pressure sensors could be installed.

CONCLUSIONS

Wireless sensor networks using MEMS technologies could enormously reduce the costs for structural health monitoring to just a few percentage of a conventional cabled monitoring system. This will increase its application and thus more detailed information could be obtained from the structural behavior as well as the actual condition of the building structure. This will enable engineers to
use more precisely information for the structure analysis and repair as well as life time prediction. For that reason first prototypes of wireless monitoring systems were developed. Still a lot of work has to be done. The next steps are to look for efficient data reduction methods as well as for sufficient algorithms for signal analysis, so that the great amount of measured data is reduced to only a few relevant data. Then structural health monitoring will be intrinsically efficient.

REFERENCES


SUMMARY

Cellulose fiber gypsum boards represent a short fiber composite material class which has gained considerable importance as a structural component in building industry in recent years. The non-combustible material is frequently used for bracing in timber frame construction. One of the most interesting features of the material consists in it’s pronounced strain softening behaviour at inplane tension loading where damage localizes at an early loading stage in a crack band.

It is reported on first results of uniaxial tension tests aiming at a direct determination of several fracture softening properties. The strains were measured localized by means of a laser extensometer. Results are given for the strain dependent localization onset, the width of the zone of distributed micro cracking (crack band width), the softening modulus and fracture energy. Further on, fracture energy determined by localized strain measurement is compared to the result obtained from evaluation of global strain measurement.

ZUSAMMENFASSUNG

Cellulosefaser-Gipsplatten repräsentieren einen Kurzfaser-Verbundwerkstoff, der in den vergangenen Jahren eine erhebliche Bedeutung als Material für tragende Zwecke im Bauwesen erfahren hat. Der nicht brennbare Werkstoff wird häufig zur Schubaussteifung im Holzrahmenbau eingesetzt. Eine der inte-
ressantesten Eigenschaften des Materials besteht in dem ausgeprägt dehnungs-
entfestigenden Verhalten, wobei sich die Schädigung frühzeitig in einem Schä-
digungsband lokalisiert.

Der Aufsatz berichtet über erste Ergebnisse von einachsigen Zugversuchen
zum Zweck der direkten Ermittlung mehrerer Bruch- sowie Entfestigungseigen-
schaften. Die Dehnungen wurden ortsaufgelöst mittels eines Laser-Extensome-
ters gemessen. Die vorgestellten Ergebnisse umfassen den dehnungsabhängigen
Lokalisierungsbegleit, die Breite der Zone der verteilten Mikrorissbildung
(Rissbandbreite), den Entfestigungsmodul und die Bruchenergie. Die mittels
ortsaufgelöster Dehnungsmessung bestimmte Bruchenergie wird mit dem Er-
gebnis zufolge globaler Dehnungsbestimmung verglichen.

RÉSUMÉ

Les panneaux à base de fibres cellulosiques et de plâtre constituent un
composite à renforts fibreux courts qui a pris ces dernières années une impor-
tance considérable comme composant structural en Génie civil. Ce matériau non
inflammable est fréquemment utilisé comme élément de chaînage dans les
constructions à ossature bois. Une des caractéristiques les plus intéressantes du
matériau est son comportement adoucissant très prononcé en traction dans le
plan, où l’endommagement se concentre à un stade précoce dans une bande fis-
surée.

Cet article présente les premiers résultats d’essais de traction uniaxiale vi-
sant à déterminer plusieurs caractéristiques d’endommagement adoucissant. Les
déformations ont été mesurées au moyen d’un extensomètre laser. Les résultats
présentés concernent l’amorce de la localisation des déformations, la largeur de
la zone micro-fissurée, le module adoucissant et l’énergie de rupture. Par la suite
on compare l’énergie de rupture mesurée par la déformation locale à celle quan-
tifiée par la déformation macroscopique.

KEYWORDS: Fiber gypsum board, short fiber composite material, strain softening, da-
mage zone, strain localization, fracture energy, crack band model
1. INTRODUCTION

Fiber gypsum boards represent a very specific type of short fiber composite materials. The plate type material consists of recycled paper fibers and a matrix material of gypsum. The fiber volume fraction is in the range of about 17 – 20%. The material is a technically superior substitute for the brittle gypsum wall board and excelled especially by two features, being incombustible and showing pronounced strain softening, i.e. damage tolerance [1-3]. At present, a comprehensive research project, funded by German Research Foundation (DFG) is conducted on this material at Material Testing Institute of Stuttgart University.

The aims of the project comprise the consistent experimental characterization and numerical modeling of the constitutive behavior of the material at static and quasi static cyclic loading. With regard to structures, the load carrying behavior of the board material, used as sheeting of seismically loaded timber frame shear walls, and its interaction with dowel type fasteners is investigated.

In the frame of the experimental material characterization and modelling, a profound understanding of the different stages of the softening mechanism is essential. So, for example present knowledge of the material behavior does not allow a judgement at what stage of the pre-peak loading path strain localization starts. Results from bending specimens did not reveal thus far whether the first nonlinearity stems from distributed micro-cracking localizing in the late stage into a crack band. However, it could also be that micro-cracking area is localizing from very early loading stages due to very high material inhomogenity at the mesolevel.

In order to identify the softening region directly in an advanced experimental approach, optical strain measurements were performed with a laser extensometer enabling a continuos locally resolved one-dimensional observation of the strain evolution within a globally relevant length scale. The employed experimental procedure allows a direct observation of the initially unknown locus of fracture localization and distributed micro-cracking within a zone of certain deliberate width. Complementary, the fracture appearance is also characterized by scanning electron microscopy aiming at a thorough understanding of the cohesion/decohesion features of the matrix fiber compound.

This article reports on first results obtained in uniaxial static tension tests with unnotched specimens.
2. SPECIMEN AND TEST SETUP

The material used was fiber gypsum board with a thickness of 12,5 mm conforming to product requirements specified in [4]. Figure 1a shows the geometry and the dimensions of the employed necked tension specimens.

The moisture content of the specimens at test time was 1,5% being typical for that material after conditioning at a climate of 65% relative humidity and 20°C.

For purpose of contact free optical strain measurement along the center part of the specimen with constant width of 30 mm, an optical grid was glued one-sided on the specimen as shown in Fig. 1b. The optical grid consists of black strips with a width of 1 mm arranged in a distance of 2 mm (center to center). During performance of the monotonic tension test, the transition of light (translucent) and black strips is scanned by a laser extensometer with a frequency of 50 Hz. The evaluation of strains between the individual strips of the grid is performed by differentiation of the relative changes of the grid distances. The tension tests were performed displacement controlled with a cross-head loading speed of 0,2 mm/min. The size of the specimen was chosen in such a manner that instable failure due to too high release of elastic energy from undamaged parts at unloading is avoided.

![Fig. 1a: Specimen and test set-up](image1)

![Fig. 1b: Optical grid](image2)

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3. TEST RESULTS

3.1 Stress vs. mean strain relation

All following statements refer to the same specimen chosen as being well representative for the five specimens tested so far. Figure 2 shows two curves of tension stress $\sigma$ vs. mean strain $\bar{\varepsilon}$ of the same specimen until complete fracture (note: mean strain $\bar{\varepsilon}$ is understood as the averaged strain within a certain distance). The given curves differ with respect to the total length $L$ of strain measurement. One curve refers to $L = L_2 = 50$ mm, being the total length of the applied optical grid, whereas the other curve is related to $L = L_1 = 20$ mm. In both cases, the damage localization and fracture zone lies within distance $L$.

The difference of the depicted curves can be easily explained by a crack band or series coupling model [5, 6]. As shown subsequently, this simple model of strain localization applies very well for the material under consideration. In a crack band model, it is assumed that distributed fracture localizes in one segment of length $h$ of the specimen whereas the rest of the specimen with length $L^* - h$ behaves in a linear elastic manner beyond peak load. In the model it is assumed that the strain state is roughly constant over the cross-section. Length $L^*$ is the total free length of the specimen, which in the regarded case is larger than maximum length $L$ of the optical grid.

![Fig. 2: Influence of length of mean strain measurement on global stress strain relation of fiber gypsum board including a crack band](image-url)
Assuming a series coupling model it is obvious that strain energy density \( G_f / L \), being the area under the loading and softening branch of the specimen’s \( \sigma - \varepsilon \) curve is considerably larger for mean strain measurement length \( L_1 \) as compared to \( L_2 \). Hereby, it is tacitly assumed that localization length \( h \) is smaller than \( L_1 \).

The fracture appearance of the tension crack shows a multiplicity of fibers sticking out of the matrix. Some fibers are broken, but predominantly fiber pull out occurs. Figure 3 exemplarily illustrates a fiber with good compound. However, large differences in the matrix-fiber interfaces were observed. One reason for this consists in the different fiber surface properties, as the fibers stem from a very heterogenous source of waste paper.

Fig. 3: Cellulose fiber with small gypsum crystals at the periphery

3.2 Strain localization

Figures 4a and 4b show the strain distribution along total strain measurement length of \( L = 50 \text{ mm} \) at different global tension stress levels within the pre- and post-peak load range. The most interesting feature revealed in the graphs of the pre-peak regime is, that a very early, almost instantaneous strain localization (area of elevated strain) occurs; in Fig. 4a, this is shown for the lowest stress level of \( 0.5 \text{ N/mm}^2 \). The width of the strain localization band grows by a factor
of 2 to 3 until ultimate stress; in the regarded example the ultimate capacity is denoted by a tension strength of $f_t' = 3.09 \text{ N/mm}^2$. The crack band width $h$ at $f_t'$ is about 6 mm.

The post-peak strain distribution is given in Fig. 4b for three decreasing load levels of 2.5, 1.5 and 0.7 N/mm$^2$. The graphs reveal the expressed strain localization in the damage zone and the elastic unloading of the rest of the specimen. The width of the crack band increases in the post-peak regime to about 7 mm.

![Fig. 4a: Strain distribution and localization at different stress levels in the pre-peak load regime](image1)

![Fig. 4b: Strain distribution and localization at different stress levels in the post-peak load regime](image2)
3.3 Average unloading and softening moduli

Figures 5a and 5b show the constitutive behavior of the elastically behaving undamaged series model part (Fig. 5a) and of the damaged part with strain localization (Fig. 5b). In detail, Fig. 5a represents the averaged stress vs. mean strain relationship of length $L - h = 50 - 7\text{ mm}$. A very close matching of the loading and unloading branch is visible. The small damage/quasi-plasticity at peak load is disregarded in this first evaluation. In addition to the experimentally observed material behavior, Fig. 5a gives the averaged unloading modulus of elasticity, determined as

$$\bar{E}_u = 3870\text{ N/mm}^2$$

Figure 5b gives the average $\sigma - \varepsilon$-relationship within the crack band width, here assumed to be 7.0 mm, as discussed in chap. 2.2. From Fig. 5b, the averaged softening modulus of elasticity is derived in a simplified linearized approach from equality of the actual and triangularized specific strain energy area as

$$\bar{E}_s = -52\text{ N/mm}^2$$

3.4 Fracture energy

Fracture energy $G_f$ represents the released strain energy density

$$U = \int \sigma \, d\varepsilon$$

(1)

integrated over the volume resp. length (in case of unit area) of the assumed damage localization, i.e.

$$G_f = h \cdot \int \sigma \, d\varepsilon$$

(2)

This is visualized in Fig. 6. Assuming inequality of the elastic loading and unloading branch within the strain localization length, which in general is the case, fracture energy obviously is [6]

$$G_f = h \left( \int_0^\varepsilon \sigma \, d\varepsilon + \int_{\varepsilon_0}^\infty \sigma \, d\varepsilon \right)$$

(3)

In the following, based on the minor differences of the elastic loading and unloading paths shown in Fig. 5a, eq. (2) is used for the determination of $G_f$. Employing the averaged unloading and softening moduli of elasticity, $\bar{E}_u$ and $\bar{E}_s$, derived
Fracture Characterization of Cellulose Fiber Gypsum Composite

Fig. 5a: Averaged stress-strain-relationship of the undamaged elastic specimen zones

Fig. 5b: Averaged stress-strain-relationship of the softening zone

above, fracture energy can be expressed equivalently by eqs. (2) and (3) as

\[
G_f = \frac{f_r^* h}{2} \left( \frac{1}{E_u} - \frac{1}{E_t} \right)
\]

(4)

For the specimen under consideration, strain energy density \( U = G_f/h \), being the area under the \( \sigma-\varepsilon \)-relationship in Fig. 5b is

\[ U = 0.0892 \text{ N/mm}^2 \]
Hence, fracture energy is roughly \((h \approx 7,0 \text{ mm})\)

\[
G_f = 7,0 \cdot 0,0892 = 620 \, \text{N/m}
\]

Alternatively, when fracture energy is determined from eq. (4) a well conforming value of

\[
G_f = \frac{3,1^2 \cdot 7,0}{2} \left( \frac{1}{3870} - \frac{1}{52} \right) = 660 \, \text{N/m}
\]

is obtained.

Based on the idea of the crack band model with negligible fracture energy contributions from the parts outside the strain localization band, fracture energy can also be derived from strain energy density based on averaged strain \(\varepsilon\) within a deliberate length \(L\) (resp. within volume \(L \times A\)):

\[
G_f = L \cdot \int \sigma \, d\varepsilon_L
\]

So, for instance, strain energy density obtained for \(L = 50 \text{ mm}\), i.e. from the area under the \(\sigma - \varepsilon\) -curve given in Fig. 2, is

\[
U_L = \int \sigma \, d\varepsilon_{50} = 0,0125 \, \text{N/mm}^2
\]

and hence

\[
G_f = 0,0125 \cdot 50 = 630 \, \text{N/m}
\]

As can be seen, a rather good agreement between the various evaluation methods is obtained; this is valid for all specimens tested.
4. CONCLUSIONS

The performed investigations revealed the good applicability of the employed experimental setup for monitoring of strain localization into a damage band of tension loaded fiber gypsum board material. The contact free laser extensometer based strain measurements allow the quantitative separation of stress and/or time dependent strain evolutions in the a priori unknown location of the damaged material zone and the adjacent undamaged parts. The resolution accuracy, being at present 2 mm, allows a sufficiently accurate determination of the crack band width of the regarded fiber gypsum composite material.

Complementary fracture surface imaging by means of electron microscopy revealed one major explanation for the expressed softening characteristics of the material built up of two brittle components, being gypsum and paper fiber: this is bound to the partly very low fiber matrix compound.

Preliminary results of ongoing investigations with quasi static cyclic loading of joints of the material with dowel type fasteners show that joints are characterized by considerable ductility / energy dissipation capacity resulting especially, however not exclusively from the described softening behavior in tension loading. This should qualify the material for bracing of timber frame shear walls subjected to seismic impacts.

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REFERENCES


SUMMARY

The paper reports on different methods of ultrasound signal de-noising. The reduction of noise is especially important for the evaluation of ultrasound transmission measurements in highly damping materials such as wood and glued laminated timber (glulam). In order to enable a reliable identification of characteristic signal parameters (such as time-of-flight or first amplitude) the poor signal to noise ratios (SNR) of ultrasound signals have to be improved by filtering methods. Conventional methods such as multiple signals averaging are used at the expense of huge data requirement, time consuming measurement procedures and signal processing.

As an alternative approach in this paper a fuzzy logic based adaptive filter is applied for de-nosing in an attempt to use a lower number of experiments, i.e. to minimize data requirements. The results are compared to those of the conventional multiple signal averaging and of a moving average filter. Preliminary results demonstrate the feasibility of the application of the fuzzy filter and clearly illustrate its advantages as well as shortcomings over the conventional approach. The presented approach is one step towards the goal of real-time non-destructive testing (NDT) inspection of glulam beams by means of ultrasound methods.
ZUSAMMENFASSUNG


Der vorgestellte Ansatz trägt zur Entwicklung eines ultraschall-basierten, in Echtzeit einsetzbaren Systems zur zerstörungsfreien Untersuchung der Integrität von Brettschichtholzbauteilen bei.

RESUME

L’article rend compte de différentes méthodes de débruitage en ce qui concerne des signaux d’ultrasons. La réduction de bruit est particulièrement importante pour l'évaluation des mesures de transmission d'ultrasons en matériaux fortement atténués tels que le bois et le bois de construction stratifié collé (glulam). Une approche d'essai non destructive est appliquée pour détecter et localiser des fissures longitudinales dans du bois lamellé-collé au moyen d'ultrasons. Une des conditions, capitale et nécessaire, en ce qui concerne l'approche est l'identification des paramètres des signaux (tels que le temps-de-vol ou la première amplitude). Les signaux après avoir traversé le faisceau sont reçus et en-
registered for the analysis and identification of cracks. After having traversed the beam, the signals are considerably corrupted by white noise. The beams of layered wood, analogous to wood, share many common devices with normal wood. They have a low density and a high attenuation effect in relation to their structure. Very low Signal-to-Noise ratios make the parameters of the received signals less recognizable. The conventional multiple average signal treatment method is employed to improve the Signal-to-Noise ratio at the cost of enormous data conditions and prolonged signal processing. A "fuzzy logic" filter is applied for the reduction of noise and improvement of Signal-to-Noise ratio in order to employ a low number of experimental trials (i.e., to reduce to a minimum the condition of data). The proposed filter is a system of neural network based on the theory of "fuzzy logic". The results are compared to those of the conventional multiple signal method making the average, and to those of other filters. Preliminary results demonstrate the practicality of the "fuzzy logic" filter and illustrate clearly its advantages (as well as its imperfections) in comparison with the conventional approach. The approach reduces temporal demands considerably and the condition of storage of high data required by the conventional approach without compromising the resolution of signals. The approach presented is a step towards the goal of non-destructive testing (NDT) of glued timber beams in real-time by ultrasonic methods.

KEYWORDS: Non-destructive testing (NDT), fuzzy logic, ultrasound, glued laminated timber (glulam), wood, cracks, Signal-to-Noise Ratio (SNR), filters

1. INTRODUCTION

In the last decades the use of timber and engineered wood products for construction purposes (as e.g. finger jointed timber, glued laminated timber, laminated veneer lumber or oriented strand board) has increased significantly. One decisive aspect for the future performance of timber in the competition with other building materials will be the question of quality assessment. In other words both the quality control of the production process and the survey of existing structures in service are important issues. The integration of non-destructive testing (NDT) methods into improved quality control systems for timber products, as already standardised for steel or concrete, will be a major
issue for the future. Due to the special characteristics of timber (e.g. growth bound relatively high variability of material parameters, anisotropy, porosity, creep behaviour etc.) one tends to encounter problems when existing non-destructive testing methods are transferred from other materials to timber.

In the case of ultrasound based methods three main aspects can be identified which are most important for the development of reliable and easy to use NDT inspection procedures for timber structures:

- Adaptation of the usual ultrasound equipment to requirements of the material wood (low frequency, high energy, improved coupling between US transducers and the timber surface)

- Modelling of wave propagation in timber in the presence / absence of damage and significant defects (cylindrical anisotropy of elastic and damping properties, influence of inhomogenities, effects of boundary conditions)

- Improved evaluation of measured ultrasound signals (Correlation methods, filtering, de-noising)

This paper deals with the filtering and de-noising of ultrasound signals from transmission measurements emanating from glued laminated timber beams. After a short description of two conventional methods of de-noising, i.e. the averaging of repeated measurements and the use of a moving averaging filter, a more advanced method based on the application of Fuzzy logic is introduced. The “Fuzzy” concept, i.e. the use of classes with boundaries that are not sharply defined, first introduced by [Zadeh 1963] and the “Fuzzy Logic”, whereby the truth of any statement is a matter of degree (e.g. reviewed in [TAKAGI. T. & SUGENO. 1983]) has found numerous applications in different fields ranging from pattern analysis and system design to damage assessment and industrial process control. The Adaptive Network Fuzzy Inference System (ANFIS) is an extension of the application of Fuzzy Logic combined with the idea of Artificial Neuronal Network. ANFIS has found its application in various fields, e.g. in the field of pattern recognition and signal processing. In this paper the ANFIS method is modified to perform as a special filter.
2. METHODOLOGY

2.1 Averaging Filter

Noise is inherent in any procedure for obtaining signals. One major type of noise is the random type. The effect due to random variation can be cancelled out by summing up a number of signal measurements. The seasonal, cyclic (or non-random) components which are the desired signal are left behind. This approach is not useful when the output signals are not static, or in other words, when the output signals come from a moving object. Equation (1) describes the simple averaging filter (AF).

\[ \bar{x}(t) = \frac{1}{n} \sum_{i=1}^{n} x_i(t) \]  

where:  
\( \bar{x}(t) \) = mean value of voltage of samples (at one sampling time)  
\( n \) = number of measurements  
\( x_i(t) \) = voltage of a sample (at one sampling time)

Its mean squared error (MSE) \( s \) is computed by Equation (2).

\[ s = \sqrt{\frac{1}{n-1} \sum_{i=1}^{n} (x_i(t) - \bar{x}(t))^2} \]  

The magnitude of \( s \) is dependent on the measurements - \( x_i(t) \) which in practice is determined by the specimen, measurement device and set up, as well as the number of measurements conducted. Once \( x_i(t) \) is fixed, the larger \( n \) is, the smaller \( s \) becomes. In other words, given a noisy but bounded measurement sequence, we can take a large number of measurements and compute the mean value to give a better estimate of the true signal (assume there is no systematic error or bias in the measurements). It is a conceptually neat approach and often a standard procedure in experimental work. Unlike the other filters, the averaging filter does not remove or change any components of the true signal and hence keeps the original signal information. The averaging filter has good performance both in the time and frequency domains. The unwanted frequency content reduces as the number of averaged signals used increases.

Theoretically, when an unlimited number of measurements are taken, the mean value of all measurements would be the signal without any noise. Natu-
rally the latter is not practical especially in real industrial applications. It is inefficient with respect to its slow convergence rate. Equation (2) demonstrates that the error $s$ is proportional to a factor of $\sqrt{\frac{1}{n-1}}$, which means the amount of effort one puts into the increase of $n$ will not achieve a convergence of $s$ corresponding to the effort invested.

Besides this shortcoming, $n$ times of measurements has to be conducted and the data stored during testing, not terribly suitable for online quality control application. Due to the high damping effect of wood and wood based material, a rather significant number of measurements have to be taken in order to ensure an identifiable signal. Even though relatively high volumes of storage media and high speed microprocessors are currently available at affordable prices, the effort needed for conducting measurements and signal processing can become significant especially when one has to deal with hundreds or perhaps thousands of $n$. We therefore need to seek more efficient ways of processing and analyzing of signals.

### 2.2 Moving Averaging Filter

The moving average filter is implemented as an alternative to the averaging filter. It is the simplest filter among all available digital filters. As its name implies, the moving average filter operates by averaging a number of continuous samples from the input signal to produce one sample as the output signal. It is represented by Equation (3).

$$y[t_i] = \frac{1}{M} \sum_{j=0}^{M-1} x[t_i + t_j]$$  \hspace{1cm} (3)

where,

$y[t_i]$  = the output signal sample  
$x[t_i]$  = the input signal sample  
$M$  = the number of continuous samples from the input signal

As an alternative, the group of samples from the input signal can be chosen symmetrically around the output sample. This is called a Symmetrical Moving Averaging Filter (SMAF) and is characterized by Equation (4).
Equation (4) requires $M$ be an odd number. The moving average filter is optimal for reducing random noise while retaining a sharp step response. However, the moving average filter is the worst filter for frequency domain encoded signals, with little ability to separate one band of frequencies from another (Smith 1999).

### 2.3 Fuzzy Filter

Fuzzy logic (FL) was first presented by Lofti Zadeh [Zadeh 1965] as a way of processing ambiguous, imprecise, noisy information or linguistic variables rather than crisp values. FL is a superset of Boolean logic dealing with the concept of partial truth. Most natural and/or man-made systems can hardly be holistically described using only crisps variables. Computers and electronic devices, for example are designed to manipulate precise or crisp values. FL was invented to allow for the representation of values between 0 and 1, shades of grey, and maybe; it allows partial membership in a set. ANFIS, the tool around which the approach advocated in this undertaking is developed, is based on FL. It implements an artificial neuro-network and provides a computational framework for manipulating and reasoning with respect to imprecise expression of knowledge including complex non-linear functions.

The proposed approach is a custom-designed model hereinafter referred as ‘Y-ANFIS’. It uses first-order Takagi-Sugeno fuzzy rule [Takagi & Sugeno, 1983]. The input to the fuzzy model and number of fuzzy rules are determined by the system dependencies, number of training data pairs and the required accuracy. Both signal and noise are functions of time ($t$), however independent from each other. Signal information is an unknown function of $t$. Noise information is a random function of $t$ and/or the history of $t$. In this work, the input to the fuzzy model is $t$ and output is the amplitude $y$. To exemplify the Y-ANFIS model, a system with one input and three fuzzy rules is used.

The fuzzy rules are constructed as the following,

If $t$ is D1, then $y_1=P_1\ t+C_1$,

If $t$ is D2, then $y_2=P_2\ t+C_2$ and

If $t$ is D3, then $y_3=P_3\ t+C_3$. 

\[ y[t_i] = \frac{1}{M} \left( \sum_{j=1}^{M-1} x[t_i + t_j] + \sum_{j=M}^{M+1} x[t_i + t_j] \right) \]
P1-P3 and C1-C3 are model parameters to be solved. D1-D3 are fuzzy numbers with a generalized bell function. It is shown in Equation (5).

\[
\mu_{D_i}(t) = \frac{1}{1 + \left( \frac{t - c_i}{a_i} \right)^{2b_i}}
\]

(5)

where, \( a_i, b_i \) and \( c_i \) are function parameters. They are given initial values and will be optimized in the Y-ANFIS model.

The outputs of the three fuzzy rules are combined by taking an arithmetic mean of each output taking into consideration the value of their weights (the degree of fulfilment). The combined response is derived in Equation (6).

\[
y = \sum_{ij} w_{ij} \cdot P_i + \sum_{ij} w_{ij} \cdot C_i + \sum_{ij} w_{ij} \cdot C_i = \sum_{ij} w_{ij} \cdot P_i + \sum_{ij} w_{ij} \cdot C_i
\]

(6)

For each \( t \), a corresponding \( y_{ij} \) can be derived using the above equation. For an entire group of signal time-domain samples, a matrix of \( y_{ij} \) can be formed. The function parameters, \( a_i, b_i \) and \( c_i \), are given initial values, which implies \( w_{ij} \) is known. The model parameters, P1-P3 and C1-C3, are left to be solved by means of the Least Square Estimation (LSE) optimization algorithm. After obtaining optimal model parameters, the function parameters are to be optimized by the Gradient Descent (GD) method (using the derivative of the model error). LSE and GD optimization procedures are repeated till they achieve the acceptable error that is previously defined by the modeller. Till here, both the model parameters and the function parameters are optimized accordingly and the overall output can be obtained.

The Y-ANFIS model applied in the signal processing is explained by the following equations. A measured signal is composed of a clean signal and noise as expressed by the addition of noise to signal in Equation (7).

\[
y(t) = x(t) + d(t)
\]

(7)

where,

\[
y(t) = \text{measured signal}
\]

\[
x(t) = \text{uncorrupted signal}
\]

\[
d(t) = \text{original noise signal}
\]
The error of the model is the difference between the measured signal and the modelled clean signal.

\[ \|e(t)\|^2 = \|y(t) - x^*(t)\|^2 = \|x(t) - x^*(t)\|^2 + 2x(t) \cdot d(t) - 2x^*(t) \cdot d(t) + \|d(t)\|^2 \]  

(8)

where,

\[ x^*(t) = \text{the modelled signal} \]

The expected value of \(\|e(t)\|^2\) is derived as Equation (9). The noise in the work is Gaussian white noise with zero mean value which leads \(E[d(t)]\) to zero. The expected values \(x(t) \cdot d(t)\) and \(x^*(t) \cdot d(t)\) are zero due to the fact that clean signal \(x(t)\) and noise \(d(t)\) as well as modelled signal \(x^*(t)\) and noise \(d(t)\) are uncorrelated. First, we consider noise as zero signals, which means clean signals can be obtained and used as input training data in the model to reproduce the signal. However, noise is always present and interfering with the desired signals. Fortunately, the noise is zero-mean, Gauss-Markov theorem still holds to ensure an unbiased LSE. Therefore, to minimize the error is to minimize the squared error between the real signal and the modelled signal.

\[ E[e^2] = E[(x(t) - x^*)^2] + E[d(t)^2] \]  

(9)

The low-frequency noise is shown as an oscillation and prevents Y-ANFIS from recognizing it as noise. Y-ANFIS is combined with the averaging filter to improve its performance in dealing with the low-frequency noise.

3. **RESULTS**

The three different filters, namely the averaging filter, moving average filter and Y-ANFIS are applied to the same sets of signals. The data sets for the comparison of the different filters are exemplarily chosen from US transmission measurements at a glulam exhibiting a longitudinal crack. The schematic test set-up is shown in Fig. 1. For the details of the measurements and the evaluation of the (unfiltered) signals see [Aicher et al. 2002]. Two sets of signals are evaluated: first, the results of transmission measurement at a location including a crack (measurement I with low SNR) and second, the results of transmission measurements at a location in the crack-free zone (measurement II with relatively high SNR).
Fig. 1 Schematic picture of the test set-up for ultrasound transmission measurements of glulam beams with longitudinal crack

Results from the AF with the original signals are shown in Fig 2 and Fig 3. The AF produces fairly good result as the high frequency noise contained in the averaged signal is removed to a considerable extent. Time required to process 26 measurements using the AF is approximately 24 seconds on an IBM R40e laptop with 2.6GHz processor and 256 MB of RAM.

Results from the SMAF are shown in Fig 4 and Fig 5. Noise corruption is indeed decreased by the SMAF. But compared to the results of the averaging filter, SMAF reduces noise while keeping a residue of high frequency noise. The SNR is also not improved by much. If the signal parameters such as the Time of Flight (TOF) and the first amplitude (Aicher et al., 2002) are to be quantified out of the filtered signal, difficulties occur as they are not easily recognizable. Time required to process a 41-sample SMAF is approximately 28 seconds on the same IBM machine. It is worth to note that unlike the averaging filter, the SMAF achieves a relatively clean signal with only one measurement, which means it saves measurement time and data storage space.

Results from the Y-ANFIS model are shown in Fig 6 and Fig 7. The AF is used after applying the Y-ANFIS model to further remove the low-frequency noise. The mean value of 10 continuous signals treated by the Y-ANFIS is computed as the final output. Results from the AF are included in the analysis of the Y-ANFIS results to facilitate a direct comparison. When one compares the re-
sults of the averaging filter, Y-ANFIS reduces high-frequency noise (above 50 kHz) while keeping low-frequency noise (below 10 kHz). Y-ANFIS is not able to recognize low-frequency noise, instead; Y-ANFIS treats it as signal. As this above-mentioned signal is essentially noise composed of random samples, the AF is able to remove the random effect. Time required to do processing using Y-ANFIS with 20 membership functions and 5 iterations is approximately 22 seconds on the IBM machine. When 10 Y-ANFIS output signals are averaged, additional 75 seconds are consumed on the same machine. It is worth noting that in the application of the Y-ANFIS model, a partial signal that is 3000 samples corresponding to 0.15-0.45 ms is used instead of the complete signal samples. This range covers the part where the signal is about to start and the first few peaks after the commencement of signal. If the complete signal is to be treated by the Y-ANFIS model, more membership functions and iterations would be needed and thus longer computing time.
Fig. 2: US transmission signal (measurement I in the crack zone) treated by averaging filter (AF) (a) complete samples, (b) partial samples
Fig. 3: US transmission signal (measurement II in the crack-free zone) treated by averaging filter (AF) (a) complete samples, (b) partial samples
Fig. 4: US transmission signal (measurement I in the crack zone) treated by SMAF (partial samples)

Fig. 5: US transmission signal (measurement II in the crack-free zone) treated by SMAF (partial samples)
Fig. 6: US transmission signal (measurement I in the crack zone) treated by \textit{Y-ANFIS + AF} (partial samples)

Fig. 7: US transmission signal (measurement II in the crack-free zone) treated by \textit{Y-ANFIS + AF} (partial samples)
4. CONCLUSION AND RECOMMENDATION

The averaging filter is the most reliable approach among the methods tested; namely the averaging filter (AF), symmetrical moving averaging filter (SMAF) and fuzzy based adaptive filter (Y-ANFIS). In other words its influence on the original signal is minimal. On the other hand, it is most time-consuming with respect to the NDT signal analysis procedure as well as most demanding in terms of storage space. SMAF shows an acceptable performance with much less time consumed in testing and signal processing. Y-ANFIS + AF show excellent results with regard to noise elimination although a great deal of computational effort is required for the non-linear mapping. Considering the fact that the AF does not alter the content of the true signal, it is the simplest method that can be considered as a reference with which to judge the performance of the other signal processing methods. If the testing device and the specimen under consideration are compatible to allow the registration of measurements with high SNR and within an acceptable time, the AF shall remain the first choice vis-à-vis the other signal processing methods. In the aforementioned situation, Y-ANFIS does not show much superiority over the AF. The AF becomes ineffective when the testing device and the specimen properties lead to signals with low SNR, e.g. in the case of the US transmission testing of real structures with large dimensions. In all cases when repeated measurements are principally not possible, e.g. acoustic emission testing, digital signal processing by means of Y-ANFIS is a reliable approach for noise reduction.

Y-ANFIS + AF could be replaced by a windowed filter (high-pass filter) combined with Y-ANFIS, thus the high-pass filter eliminates low-frequency noise before Y-ANFIS is applied. Windowed filter + Y-ANFIS can further reduce the number of measurements needed for each point under consideration. The latter could be set a research agenda that could be undertaken in a future study. Together with improvements in the measurement device, the real-time application of NDT in the field of timber engineering can be achieved.
ACKNOWLEDGEMENT

The work described in this paper is partially supported by a grant from the German Research Foundation (Deutsche Forschungsgemeinschaft - DFG) under the theme of the Special Research Area (Sonderforschungsbereich - SFB), SFB 381, subproject A8 “Damage characterisation and non-destructive testing of the natural composite wood”. Its contents are solely the responsibility of the authors and do not necessarily represent the official position or policy of the German Research Foundation.

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SUMMARY

Transient elastic waves emitted by sudden failure within a specimen comprise information about the global stress field, processes in the sources, and effects of wave propagation in the medium. Signal based acoustic emission (AE) analysis allows for a comprehensive investigation of the alteration of the material, and fracture processes as AE sources. The mechanism of an AE source can be evaluated by the seismic moment tensor concept. This concept describes an AE source by a system of equivalent body forces. To handle such a complex problem, various assumptions are made in practice. One important assumption is the point source approximation. An AE source can be considered as a point source if both the distance of the sensor from the source and the wavelength of the signals are much greater than the linear dimension of the source. These requirements are fulfilled in case of a short duration of the source-time function observed in greater distance from the source. Since in AE measurements the wave field is often observed in smaller distances, near-field effects can occur. In the following, near-field effects are discussed analytically.

ZUSAMMENFASSUNG

Transiente, elastische Wellen, die durch Bruchprozesse in einem Probekörper ausgesendet werden, enthalten eine Vielzahl von Informationen über das globale Spannungsfeld, die Abläufe in der Bruchzone und Effekte des Wellenausbreitungsmediums. Mit Hilfe der signal-basierten Schallemissionsanalyse können Veränderungen der Materialeigenschaften beobachtet werden. Eine um-

RESUMÉ

Des ondes elastiques qui ont emette d'une fracture dans une epreuve contennissent beaucoup d'informations sur la tension dans l'epreuve, sur le deroulement de la fracture et sur des effets visible dans la structure des ondes a cause de l'epreuve. Enregistrer tout le signal des emissions acoustiques on peut observer des changements dans le materiel. Des investigations globales devraient possible avec le brouillon du tensor du moment sismique. La source de l'emission acoustique est decrit d'un system des moment equivalent. La solution de cette probleme inverse est seulement possible avec quelques suppositions. Une supposition importante est que la source des ondes elastique est approxime d'un point. C'est possible quand la distance entre source et recepteur est grande par rapport a la region de source et quand la fonction qui decrit la source est une impulsion tres courte. A cause de la geometrie de l'epreuve la distance entre source et recepteur n'est pas toujours tres grande quand on enregistre des emissions acoustiques. Alors, il faut attendre a des derangements de la methode du champ roche. Dans ce quit suit les effets de la methode du champ roche devrai presente en detail.

KEYWORDS: acoustic emission, moment tensor, near-field
1. INTRODUCTION

Fracturing is a common and well-known procedure over a large range of scales. Knowledge about fracture processes helps to optimize the design of structures, to prevent accidents, and to understand the dynamics of seismic sources, e. g. earthquakes. Numerous methods applied in signal based acoustic emission analysis are related to seismological techniques. The observation of teleseismic events led to the accepted model of a double-couple mechanism as the origin of tectonic earthquakes. Typically, these are characterized by the relative slip movement of two tectonic plates, initiated in the so-called hypocenter. Meanwhile, a global network monitors earth seismicity and tectonic earthquakes in the far field and the source mechanisms are routinely determined using the moment tensors method [DZIEWONSKI et al., 1983]. The moment tensors give information about the seismic moment (magnitude), the orientation of the rupture plane, and the source type. The determination of the source type is important to identify explosions (e. g. nuclear tests) as well as for a detailed investigation of fracture processes [e. g. DAHEN & TROMP, 1998]. Basic investigations in AE as well as material sciences are event counting and statistic analysis. Improved signal analysis (e.g. detection of the signal onset times and amplitudes) allows for a localization of the sources and the determination of source mechanisms using the moment tensor method. Various moment tensor inversion techniques were applied on AE data sets revealing plausible results for the stress field and failure mechanisms [e. g. OHTSU ET AL., 1991, GROSSE, 1996, MANTHEI ET AL., 2001, FINCK ET AL., 2003].

In most applications, the localization of AE events reveals a point in the coordinate system, or at least a solution which is very small compared to the wave length of the signal. Thereby, the point-source approximation appears for the first time. When analyzing the elastic wave field to gather information about source mechanisms the point source approximation is one important assumption. In practice all inversion techniques neglect near-field effects to simplify the algorithms. Often, the validity of this assumption was queried in the case of small-scale experiments in the laboratory. The development of a macro crack or a fault system is accompanied by a number of discrete fracturing events. BEN-ZION [2001] and STÖCKL & AUER [1976] published velocities of crack propagation in the order of the S-wave velocity in the respective medium. The extension of microcracks in rocks and therewith the source volume corresponds to the grain size in the material under quasi static loading conditions.
2. WHAT ARE NEAR-FIELD EFFECTS?

Two different approaches can be made to examine near-field effects. First, fracturing is a permanent deformation altering the material – a crack is opening or the crack surfaces are sliding along each other in a “short” period of time. The displacement in the source is characterized by non-linear effects which lead to a permanent offset in the signal. These effects vanish in great distances from the source and the signal can be described as linear-elastic wave. These non-linear effects are formulated as near-field and intermediate-field terms in seismology [e. g. Aki & Richards, 2002, Ben-Menahem & Singh, 1981].

Second, we take a closer look into that “short” period of time. According to the stress field and material properties, crack growth and development is also accompanied by phenomena like flaking, splitting, de-bonding, and friction during the slip of the crack surfaces. Also partial melting in the fracture zone can occur. Though these phenomena can be found over all scales, under the microscope as well as in the San Andreas fault system [e. g. Eibach, 1996], they are usually hidden for a direct observation and will not be investigated in this article. Anyway, these effects will contribute to the complexity of the registered signals, especially at high frequencies. It is obvious, that the point-source approximation ignores these phenomena, which are depending on time and space within the source.

3. ANALYTICAL DESCRIPTION OF THE WAVE FIELD

The concept of the moment tensor and its application on acoustic emission signals was already discussed in numerous publications [e. g. Mantel, 1991, Grosse, 1999]. In these publications the evaluated AE sources were considered as point sources and near-field terms were neglected a priori.

Studying near-field effects, we will get into the problem at an advanced point. Aki & Richards [2002] stated the initial point of our investigation with Eqn. 1. The moment tensor $\mathbf{M}$ is convoluted with simplified Green’s functions $\mathbf{G}$ for a homogenous and isotropic full space. This equation describes the $n^{th}$-component of the displacement field. $\gamma_i$ are the direction cosines of the sensor position relative to the source location, $r$ is the distance between source and receiver, $\alpha$ and $\beta$ are the P- and S-wave velocities, respectively.
On near field effects in signal based acoustic emission analysis

\[ M_{pq} G_{np,q} = \left( \frac{15\gamma n p q - 3\gamma n p q - 3\gamma p q - 3\gamma q p - 3\gamma q p}{4\pi \rho} \right) \frac{1}{r^4} \int_{r/\alpha}^{\beta} \tau M_{pq}(t-\tau)d\tau \\
+ \left( \frac{6\gamma n p q - \gamma n p q - \gamma p q - \gamma q p}{4\pi \rho \alpha^2} \right) \frac{1}{r^2} M_{pq} \left( t - \frac{r}{\alpha} \right) \\
- \left( \frac{6\gamma n p q - \gamma n p q - \gamma p q - \gamma q p}{4\pi \rho \beta^2} \right) \frac{1}{r^2} M_{pq} \left( t - \frac{r}{\beta} \right) \\
+ \frac{\gamma n p q}{4\pi \rho \alpha^3} \frac{1}{r} \dot{M}_{pq} \left( t - \frac{r}{\alpha} \right) \\
- \left( \frac{\gamma n p q - \delta p q}{4\pi \rho \beta^3} \right) \frac{1}{r} \dot{M}_{pq} \left( t - \frac{r}{\beta} \right) \tag{1} \]

The first term of Eqn. 1 describes the near-field term which is proportional to \(1/r^4\). The following terms describe the intermediate-field and the far-field of the P- and S-waves which are proportional to \(1/r^2\) and \(1/r\), respectively.

To calculate the Green’s function according to Eqn. 1 we used a stepwise parabolic function (Eqn. 2) with a very short duration of only four samples (sampling interval is 100 ns which corresponds to 10 MHz) [KÜHNICKE, 1986]. The time derivation of this function is a triangular pulse, which gives “needle” pulses at times of the P-wave and the S-wave arrival.

\[ f_{\text{pulse}}(t) = \begin{cases} 0 & ; t < 0 \\
\frac{2t^2}{T_p^2} & ; t < T_p / 2 \\
\frac{4t}{T_p} - \frac{2t^2}{T_p^2} - 1 & ; t > T_p / 2 \\
1 & ; t > T_p \end{cases} \tag{2} \]

Then we convolved the Green’s functions with the wavelet representing the source-time function. We chose a wavelet proportional to \(f(t)\) (Eqn. 3) described by [MÜLLER, 1987].

\[ f(t) = \sin\left(2\pi \frac{t}{T_a} \right) - \frac{1}{2} \sin\left(4\pi \frac{t}{T_a} \right) \tag{3} \]

This wavelet corresponds to the second time derivative of a step function with rise time \(T_a\). Convolution of the Green’s function with the first derivative
\( \dot{f}(t) \) of the source-time function yields the displacement signal which is a pulse in the far field of the P- and S-waves. Fig. 1 shows the step function \( f(t) \) (continuous line) and its first and second time derivatives \( \dot{f}(t) \) (dashed line) and \( \ddot{f}(t) \) (dash-dotted line), respectively.

![Graph showing source-time function and its derivatives](image)

**Fig. 1:** Source-time function (continuous line) with rise time \( T_a \) and its first (dashed line) and second time derivatives (dash-dotted line) [MÜLLER, 1987].

On the basis of Eqns. 1 to 3 a MATLAB\textsuperscript{®} routine was developed to calculate synthetic seismograms. In this program the sensor position and its direction, the source mechanism (moment tensor), the rise time, and additional parameters concerning material properties can be entered.

### 4. RESULTS

The displacement field was studied with respect to source type, distance between source and receiver and duration of the source-time function. Furthermore, the influence of the sensor orientation was examined. In the following, various examples of synthetic calculations will be presented. An estimation of the influence of near-field and intermediate-field effects on the entire displacement is performed at the end of this section. The calculations are performed for an isotropic and homogeneous full space. P-wave velocity \( \alpha = 4000 \text{ m/s} \), S-wave velocity \( \beta = \alpha/1.71 \), and mass density \( \rho = 2700 \text{ kg/m}^3 \) were used, as these parameters are typical for concrete.
Eqn. 1 yields a transient 3-dimensional wave field. To plot this field, a projection is useful to illustrate the effect of the three displacement terms. Fig. 3 shows the displacement signal of a double-couple (DC) mechanism (Fig. 2) in a distance of 0.2 m and a source-time duration of 10 µs. The displacement signal is decomposed into a radial P-wave component (left-hand side) and a horizontal polarized S-wave component (in the middle). The right-hand side of Fig. 3 displays the absolute amplitudes of the superposition of both wave types. Furthermore, the figures show the contribution of the near-field term (dash-dotted line), the intermediate-field term (dotted line), and finally the far-field term (dashed line). The observation was performed at an angle of $\varphi = 22.5^\circ$ relative to the T-axis of the double-couple mechanism (see Fig. 2). In this direction no nodal plane occurs in the radiation pattern of the P- and S-wave.

It can be seen, that the P-wave pulse occurs only in the radial (left-hand side of Fig. 3) and the S-wave pulse in the tangential direction (middle of Fig. 3). The right-hand side of Fig. 3 shows both wave types. The near-field and intermediate-field terms appear between the P- and S-wave pulses, which lead to a permanent signal offset after the S-wave pulse. The maximum amplitude of the S-wave measured at an angle of $\varphi = 45^\circ$ is about 5 times of the maximum amplitude of the P-wave measured at an angle of $\varphi = 0^\circ$. 

Fig. 2: Stereographic projection of the radiation pattern of a double-couple mechanism. Sinistral slip on one of the nodal planes is indicated by the arrows. The P-wave amplitude is maximal in direction of the T-axis.

Fig. 3: Far-field, intermediate-field and near-field terms of displacement signal in a distance of 0.2 m with source-time duration of 10 µs. 

It can be seen, that the P-wave pulse occurs only in the radial (left-hand side of Fig. 3) and the S-wave pulse in the tangential direction (middle of Fig. 3). The right-hand side of Fig. 3 shows both wave types. The near-field and intermediate-field terms appear between the P- and S-wave pulses, which lead to a permanent signal offset after the S-wave pulse. The maximum amplitude of the S-wave measured at an angle of $\varphi = 45^\circ$ is about 5 times of the maximum amplitude of the P-wave measured at an angle of $\varphi = 0^\circ$. 

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Fig. 4 and Fig. 5 show the synthetic displacement and velocity signals, respectively, of a double-couple mechanism observed in distances of 0.1 m, 0.2 m, and 0.3 m. Two values of the source-time duration \( T_a \) were used, 10 \( \mu s \) and 20 \( \mu s \), which corresponds to typical frequencies of 100 kHz and 50 kHz observed in laboratory tests on concrete specimens. The radial component of the total displacement field is plotted as a continuous line and the radial component of the far-field term is plotted as a dashed line.

![Graphs showing synthetic displacement and velocity signals](image)

**Fig. 4:** Plot of calculated displacement signals for a point source in an infinite medium as received in various distances \( r \) from the source with source-time duration of 10 \( \mu s \) and 20 \( \mu s \). Continuous lines show the complete displacement signal, dashed lines show only the far-field displacement signal. The pulses correspond to the P- and S-wave onsets, respectively.

The figures clearly illustrate the dependency of the signals with distance between source and receiver (from left to right) and with source-time duration. In a short distance the P-wave and S-wave pulses are overlapped, especially for a longer pulse duration (left-hand side of Fig. 4). The contribution of the near-field terms of the displacement signal is obvious at times between P- and S-wave arrivals. As the travel paths become longer or the pulse duration becomes
shorter the difference between the complete displacement and the far-field term alone becomes less apparent.

![Fig. 5: Same plot as Fig. 4 for velocity signal.](image)

In the case of a velocity signal (Fig. 5), which is more realistic in AE, because broadband AE sensors are supposed to be a velocity sensor, the effect of the near-field terms significantly decreases.

On the basis of these calculations, we estimated a critical source-time duration \( \tau_c \) as a function of rise time \( T_a \). For source-time durations \( \tau < \tau_c \) at distances \( r < r_c \) (with \( r = \alpha \tau \) and \( r_c = \alpha \tau_c \)) the influence of the near-field terms on the P-wave amplitudes was greater than 10%. In this case we assume that near-field effects can not be ignored in application of the moment tensor method. Fig. 6 shows the critical source-time duration \( \tau_c \) versus rise time \( T_a \) for a pure double-couple mechanism and a pure tensile mechanism. In both calculations, the radial component at \( \varphi = 0^\circ \) was observed. A linear relation was obtained. According to these calculations, the near-field term can be ignored in the
case of a double-couple mechanism in distances greater than approximately 6 wavelengths of the P-wave. For a tensile crack (mode I), near-field effects are attenuated already after approximately 3.5 wavelengths of the P-wave. In the case of the double-couple, the position of the sensor relative to the source did not affect the results on the radial component, but for mode I a minor variation was observed.

Fig. 2 shows that the near-field also has a transversal component. When the sensor is not oriented radial to the source, this has an additional consequence on the registered wave-field. To visualize this phenomenon, the number of wavelengths necessary for near-field effects to decrease under the 10% threshold was calculated as a function of the sensor position to the source as well as the relative orientation of the sensor to the source. Results are given as two matrices in Fig. 7, where the number $n$ of wavelengths is coded due to a grayscale. On the x-axis, the angle $\sigma$ is giving the orientation of the sensor to the vector from source to receiver. The calculation was performed for $\sigma = -85^\circ$ in steps of $10^\circ$ to $\sigma = 85^\circ$. On the y-axis, the angle $\varphi$ is plotted from $\varphi = 0^\circ$ to $\varphi = 180^\circ$. The left image shows results for the double couple, the right image for a tensile crack.

Fig. 6: Critical source-time duration $\tau_c$ versus rise time $T_a$ for a pure double-couple mechanism (continuous line) and a pure tensile mechanism (dotted line) observed on the radial component and $\varphi = 0^\circ$. 
Critical number of wavelengths as function of position and orientation of the sensor to the source for double-couple (left) and a pure tensile mechanism (mode I, right).

Depending on the orientation of the sensor, a decrease or an increase of disturbing near-field effects on the measured amplitudes is obtained. This is caused by an interference of near-field effects on the tangential and the radial component. In both figures, the symmetry of the matrices due to the radiation patterns of the source types is evident. In particular, close to the nodal planes of the double couple, where P-wave amplitudes are weak anyway, the registered data are highly influenced by near field effects.

5. CONCLUSION

Our investigations show, that near-field effects can not generally be neglected in signal based acoustic emission evaluations. A 10% residual is inherent for a pure double-couple mechanism below 100 kHz measured in distances smaller than 0.24 m in concrete. Also, the relative orientation of the sensor axis to the source and position of the sensor seems to affect the results. In the future, we will investigate the influence of near field effects on moment tensor inversion, also for mixed-mode source types. Furthermore, the influence of the sensor characteristic will be taken into account. First calculations show, that a high-pass filter with a corner frequency of approximately 30 kHz further reduces near-filed effects. Our piezoelectric transducers can be regarded as high-pass filters.

Concerning our results, the experimental set-up should be optimized. A three component registration would allow for the evaluation of a radial component, independent from the geometry of the specimen. To minimize the influ-
ence of near-field effects, normal incidence of the wave field on the sensors should be preferred.

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REFERENCES


PRACTICAL INVESTIGATION OF THE SULFATE RESISTANCE OF CONCRETE FROM CONSTRUCTION UNITS

PRAXISNAHE UNTERSUCHUNG DES SULFATWIDERSTANDES VON BETON AUS BAUTEILEN

ETUDE PRATIQUE DE RESISTANCE DU BETON DES ELEMENTS SOUS ACTION DE LE SULFATE

Oliver Mielich, Christian Öttl

SUMMARY

To determine the sulfate resistance of a sports hall foundation concrete samples (drill-out cores) were taken out of the foundation structures for being compared with lab-manufactured concretes and mortars under varying conditions. Following here the so called flat prism procedure, samples (drill-out cores) from the new building and different composed types of concrete drill-out cores as well as flat mortar prisms were immersed in a sodium sulfate solution (29800 mg sulfate/l). The expansion of the specimen were measured up to 150 days storage as well as 540 days of a long-term storage. With regard to the temperature conditions of the foundation in situ the tests have been carried out not only at 20 °C but also under more severe conditions at 6 °C.

ZUSAMMENFASSUNG

RESUME

Pour déterminer la résistivité de sulfate d’un béton de fondations d’une salle omnisports des éléments échantillons (centres d’enfonce) étaient prélevés des corps de fondation de la nouvelle construction, et les étaient comparés avec les centres d’enfonce et prismes du mortier fabriqués en laboratoire dans des conditions de conservation variés. Les éléments échantillons (centres d’enfonce) de la nouvelle construction de la salle d’omnisports et des centres d’enfonce du béton composés divers et prismés du mortier étaient montés dans une solution du sodium sulfate à 4,4 pour cent suivant l’exemple du procédé des prismes, et les extensions étaient déterminées jusqu’à une durée de conservation des 150 jours et après une conservation de longue durée des 540 jours. Les épreuves étaient exécutées aux 20 °C et aussi aux 6 °C dans des conditions plus sévères, avec le but de reproduire les conditions de température axé sur la pratique en domaine de zone de fondation d’une nouvelle construction d’une salle omnisports.
1. INTRODUCTION

At the end of the construction of a new building of a sports hall the responsible city administration contacted the Otto-Graf-Institut of the MPA University of Stuttgart. Because of contradictory test results it was unclear if the required compressive strength of the foundation of the sports hall was reached or not. Furthermore a cracking of the concrete floor and an apparently defective concrete surface were criticised. In addition to this the resistance of the foundation concrete was to be investigated in relation to strong sulfate attack because the present groundwater contained significant amounts of sulfate and the utilised water/cement ratio (w/c-ratio) was not in accordance with relevant standards.

2. REGULATIONS CONCERNING SULFATE ATTACK ON CONCRETE

Concrete units exposed to sulfate attack may be damaged by expansion by the cause of the formation of secondary ettringite and secondary gypsum [1]. At low temperatures predominating in foundation structures it is known that under unfavorable conditions an additional damage may occur by the formation of thaumasite. To protect a secure production method of concrete building, concrete which is exposed to a sulfate attack must have a high resistance against chemical attack according to DIN 1045 [2] and DIN EN 206-1 [3]/DIN 1045-2 [4] respectively. In compliance with the regulations no damages occurred so far [1].

The resistance of concrete against sulfate attack consists of a chemical and a physical resistance. For chemical resistance the used cement and/or the used cement-fly ash combination is relevant. For physical resistance the microstructure density is significant which is usually controlled by a suitable w/c-ratio.

2.1 Current State of Standardization

According to DIN 1045 [2], chapter 6.5.7.5, the resistance of the concrete structure against chemical attack depends basically on its density. The w/c-ratio of the concrete may therefore not exceed by „weak“ attack 0.60 and 0.50 by „strong“ attack. Further a cement with high sulfate resistance, according to DIN 1164-10 [5], must be used if the sulfate content exceeds 600 mg per litre water, without seawater present. In case of foundations the threshold value is 3000 mg sulfate per kg soil.
The concrete standard DIN EN 206-1 [3]/DIN 1045-2 [4] which must be used starting from 01.01.2005 classifies concretes depending on the ambient conditions in exposure classes. Thereby the exposure classes for concretes exposed to chemical attack by natural soil, groundwater, sea water and waste water are assigned as followed:

- **XA1**: chemically weak corrosive
- **XA2**: chemically moderate corrosive
- **XA3**: chemically strong corrosive

The particular requirements of the exposure classes XA1, XA2 and XA3 are represented in table 1.

*Tab. 1: Measures for the production of concrete with high sulfate resistance against water containing sulfate.*

<table>
<thead>
<tr>
<th>Aggressive surrounding area</th>
<th>chemically weak</th>
<th>chemically moderate</th>
<th>chemically strong</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exposure classes</td>
<td>XA1</td>
<td>XA2</td>
<td>XA3</td>
</tr>
<tr>
<td>Criterion for SO$_4^{2-}$ in mg/l in groundwater</td>
<td>$\geq 200$ and $\leq 600$</td>
<td>$&gt; 600$ and $\leq 3000$</td>
<td>$&gt; 3000$ and $\leq 6000$</td>
</tr>
<tr>
<td>Max. w/c-ratio</td>
<td>0.60</td>
<td>0.50</td>
<td>0.45</td>
</tr>
<tr>
<td>Minimum compressive strength class</td>
<td>C 25/30</td>
<td>C 35/45$^{1)}$ (C 30/37$^{1)}$)$^{2)}$</td>
<td>C 35/45$^{1)}$</td>
</tr>
<tr>
<td>Minimum cement content in kg/m$^3$</td>
<td>280</td>
<td>320</td>
<td>320</td>
</tr>
<tr>
<td>Minimum cement content by charging for additives in kg/m$^3$</td>
<td>270</td>
<td>270</td>
<td>270</td>
</tr>
</tbody>
</table>

1) by use of air-entrained concrete, for example as result of coexistent requirements from the exposure class XF, one compressive strength class lower
2) according to the draft to the A1 amendment of the DIN 1045-2 [6] a reduction of the compressive strength is intended
2.2 Cements with High Sulfate Resistance

Cements with high sulfate resistance (HS-cements) must meet the requirements of DIN EN 197-1 [7] and DIN 1164-10 [5]. Blast furnace cement CEM III/B is considered as a HS-cement with high sulfate resistance due to its high granulated blast furnace slag content (≥ 66 % by mass).

2.3 Influence of Additives on the Sulfate Resistance

The sulfate resistance of blast furnace cements increases with granulated blast furnace slag contents above approximately 60 % by mass to high values [8]. Reason for this is the high diffusion resistance of the hardened blast furnace cement structure at higher slag contents which cannot be penetrated by sulfate ions. Starting from a granulated blast furnace slag content of at least 65 % by mass an increase of the granulated blast furnace slag content affects the increase of the sulfate resistance substantially more than a reduction of the w/c-ratio [8].

The increase of the sulfate resistance by the puzzolanic effect of hard coal fly ash is basically attributed to higher concrete tightness and the guarantee of improved diffusion resistance against sulfate ions by additional formation of CaO-poorer C-S-H-phases [8].

3 TESTING METHODS FOR THE DETERMINATION OF THE SULFATE RESISTANCE

3.1 Testing Methods for Cement

For the determination of the sulfate resistance of cement there are different national and international testing methods common. In the majority the acceleration of the sulfate entry into the samples is accomplished by storing in very high sulfate concentrations. These concentrations are partially ten times higher than the accepted sulfate concentrations in practice. The assessment of the majority of testing methods is done by measuring the expansion of flat prisms during sulfate storage and reference storage which are compared with a predefined criterion. In addition to common test methods by expansion-measurements the test method of Koch-Steinegger [8] as well as the MNS procedure [9] are practiced in Germany. It is not the intention of this article to deal with these testing methods in greater detail. Table 2 shows a comparison of the accelerated test methods by expansion measurements according to Wittekindt [10], SVA [8] and CEN [8].
Tab. 2: Comparison of accelerated test methods by means of expansion measurement

<table>
<thead>
<tr>
<th>Test pieces</th>
<th>Wittekindt</th>
<th>SVA</th>
<th>CEN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10 x 40 x 160 mm³</td>
<td>10 x 40 x 160 mm³</td>
<td>20 x 20 x 160 mm³</td>
</tr>
<tr>
<td>w/c-ratio</td>
<td>0.60</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>Sulfate solution</td>
<td>4.4 % (29800 mg sulfate/l)</td>
<td>4.4 % (29800 mg sulfate/l)</td>
<td>2.4 % (16000 mg sulfate/l)</td>
</tr>
<tr>
<td>Storage period</td>
<td>56 days</td>
<td>91 days</td>
<td>not defined</td>
</tr>
<tr>
<td>Criterion</td>
<td>≤ 0.50 mm/m</td>
<td>≤ 0.50 mm/m</td>
<td>not defined</td>
</tr>
</tbody>
</table>

2.2 Testing Methods for Mortar and Concrete

At present the evaluation of the sulfate resistance of concrete is still subject of research. The today's used testing methods for concrete orientate at the testing methods for cement and are performed in this or in modified ways [9].

In Germany the determination of the sulfate resistance of mortar and concrete is carried out with several methods. In order to determine the sulfate resistance of concrete the testing method of the committee of experts (SVA) of the German Institute for Civil Engineering appeared to be suitable in this case.

In this testing method flat prisms according to DIN EN 196-1 [11] are manufactured with a w/c-ratio = 0.50 and are cast in prisms 10 x 40 x 160 mm³. The specimen are demoulded after 2 days storage in moist air and are additionally stored 12 days in saturated Ca(OH)₂-solution. The following storage takes place in a 20 °C sodium sulfate solution (29800 mg sulfate/l). The solutions are monthly renewed. The sulfate resistance is rated by the expansion difference (Δε ≤ 0.50 mm/m) after 91 days (=HS-criterion).

4 TEST PROGRAM

4.1 Intention

The investigations are focused on two aspects. On the one hand the expansion behavior should be compared with mortar prisms that were produced with a binder combination of blast furnace cement and fly ash and were stored in a sodium sulfate solution (29800 mg sulfate/l). By the way the binder components of the foundation concrete and the mortar prisms derived from the same origin. Apart from the usual storage temperature of 20 °C the samples were also stored at 6 °C which is regarded as an intensification of the test conditions. On the
other hand there is the question what effect an increase of the acceptable w/c-ratio has on the structure density and thus on the resistance of the foundation structure in regard to sulfate attack. For this reason mortar prisms with different cement types and varying w/c-ratio were examined using the flat prism test. In addition to this samples from the foundation concrete were compared to standard manufactured concrete specimen produced in accordance to DIN 1045 [2] and DIN EN 206-1 [3]/DIN 1045-2 [4] respectively.

4.2 Mortar Tests according to the Flat Prism Procedure

Following the flat prism procedure three mortar mixtures were produced. The w/c-ratio of the mixtures 1 and 2 meets the guidelines of the flat prism procedure. For the mixture 3 the w/c-ratio of 0.68 was selected according to the composition of the foundation concrete in situ. The mortar compositions are presented in table 3:

<table>
<thead>
<tr>
<th>Description</th>
<th>Mixture 1</th>
<th>Mixture 2</th>
<th>Mixture 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement type</td>
<td>Portland cement</td>
<td>Blast furnace cement</td>
<td>Blast furnace cement</td>
</tr>
<tr>
<td></td>
<td>CEM I 32.5 R</td>
<td>CEM III/B 32.5 – NW/HS</td>
<td>CEM III/B 32.5 – NW/HS</td>
</tr>
<tr>
<td>Cement content in g</td>
<td>450.0</td>
<td>420.5</td>
<td>420.5</td>
</tr>
<tr>
<td>Fly ash content in g</td>
<td>–</td>
<td>73.8</td>
<td>73.8</td>
</tr>
<tr>
<td>Water content in g</td>
<td>225</td>
<td>225</td>
<td>306</td>
</tr>
<tr>
<td>$w/(c + k \cdot f)$</td>
<td>0.50</td>
<td>0.50</td>
<td>0.68</td>
</tr>
</tbody>
</table>

$k = 0.4$
The mixed mortar was cast in the moulds for flat prisms with the dimensions 10 x 40 x 160 mm$^3$ and was compacted on the vibration table according to DIN EN 196-1 [11]. A two-day storage of the prisms followed at 20 °C in moist air of ≥ 90 % relative humidity. Subsequently the specimen were demoulded and were stored 12 days in saturated Ca(OH)$_2$-solution. After this the initial length was measured. The following storage conditions subdivide into:

- 20 °C in saturated Ca(OH)$_2$-solution (reference solution)
- 20 °C in sodium sulfate solution (29800 mg sulfate/l) (test solution)
- 6 °C in saturated Ca(OH)$_2$-solution (reference solution)
- 6 °C in sodium sulfate solution (29800 mg sulfate/l) (test solution)

The length change of the specimen was measured at 14, 28, 56 and 91 days. A replacement of the Na$_2$SO$_4$-solution was carried out in fortnights.

### 4.3 Concrete Tests following the Flat Prism Procedure

At first concrete samples (drill-out cores ∅ = 50 mm) from the foundation structure of the sports hall building were taken. The composition of the utilised concrete is shown in table 4:

<table>
<thead>
<tr>
<th>Description</th>
<th>S IV (construction units)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement type</td>
<td>Blast furnace cement</td>
</tr>
<tr>
<td>Cement content in kg/m$^3$</td>
<td>CEM III/B 32.5 – NW/HS 285</td>
</tr>
<tr>
<td>Fly ash content in kg/m$^3$</td>
<td>50</td>
</tr>
<tr>
<td>Water content in dm$^3$/m$^3$ w/(z+0.4-f)</td>
<td>207.4 0.68</td>
</tr>
<tr>
<td>Aggregates</td>
<td>sand, shell-limestone</td>
</tr>
<tr>
<td>Grading curve</td>
<td>A/B 22</td>
</tr>
<tr>
<td>Content (dry) in kg/m$^3$</td>
<td>1771</td>
</tr>
</tbody>
</table>
Furthermore three additional concretes were produced for comparison (S I, S II and S III). Their composition is shown in table 5:

**Tab. 5:** Composition of the comparison concrete S I, S II and S III

<table>
<thead>
<tr>
<th>Description</th>
<th>S I</th>
<th>S II</th>
<th>S III</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement type</td>
<td>Portland cement CEM I 32.5 R</td>
<td>Blast furnace cement CEM III/B 32.5 – NW/HS</td>
<td>Blast furnace cement CEM III/B 32.5 – NW/HS</td>
</tr>
<tr>
<td>Cement content in kg/m³</td>
<td>305</td>
<td>285</td>
<td>285</td>
</tr>
<tr>
<td>Fly ash Content in kg/m³</td>
<td>–</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>Limestone meal Content in kg/m³</td>
<td>30</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Water content in dm³/m³</td>
<td>207.4</td>
<td>183.0</td>
<td>152.5</td>
</tr>
<tr>
<td>w/(z+0.4-f)</td>
<td>0.68</td>
<td>0.60</td>
<td>0.50</td>
</tr>
<tr>
<td>Aggregates</td>
<td>S/M¹)</td>
<td>S/M¹)</td>
<td>S/M¹)</td>
</tr>
<tr>
<td>Grading curve</td>
<td>A/B 22</td>
<td>A/B 22</td>
<td>A/B 22</td>
</tr>
<tr>
<td>Content (dry) in kg/m³</td>
<td>1746</td>
<td>1792</td>
<td>1871</td>
</tr>
<tr>
<td>Concrete admixtures</td>
<td>–</td>
<td>–</td>
<td>Superplasticizer</td>
</tr>
<tr>
<td>Content in ml/kg²)</td>
<td>–</td>
<td>–</td>
<td>7</td>
</tr>
</tbody>
</table>

¹) sand, shell-limestone
²) related to the cement content

For each concrete mix S I, S II and S III three cubes with 200 mm of edge length were cast. The cubes were stored after the production at 20 °C in moist air of ≥ 95% relative humidity. At the age of two days four drill-out cores Ø = 50 mm were taken from each cube.

Before immersion the lab manufactured drill-out cores were sawn to a length of 150 mm and were flat polished at the top surfaces similar to the construction samples (S IV). Furthermore stain plugs were fixed at the surface to measure length variation. Until the initial measurement the drill-out cores were stored at 20 °C and 65 % relative humidity for 12 days.
Following the flat prism procedure (see section 3.2) in each case three drill-out cores of the concretes S I, S II, S III and S IV (construction samples) were immersed at 20 °C in a sodium sulfate solution (test solution) as well as in water (reference solution). According to [8] the flat prism procedure at 20 °C storage temperature doesn’t always sufficiently represent the environmental conditions. For this reason three drill-out cores of the concretes S I, S II, S III and S IV were stored at 6 °C sodium sulfate solution and in water additionally. The immersion at 6 °C is regarded as a more severe test condition in comparison to 20 °C storage temperature.

The length change of the specimen was measured at 0 (initial value), 2, 5, 7, 14, 28, 56, 91, 120 and 150 days of immersion. The sodium sulfate solution was also changed in fortnights.

For the determination of the long-term behaviour the drill-out cores of the comparison concrete S I and the drill-out cores from the building (S IV) were finally measured after 540 days beyond the regular storing range.

5 RESULTS

5.1 Results on Mortar Prisms

To quantify the sulfate resistance of the specimen the expansion of ≤ 0.50 mm/m after 91 days immersion was chosen as a threshold value. Fig. 1 and 2 show the expansion behaviour of flat mortar prisms stored in a 20 °C and 6 °C sodium sulfate solution (29800 mg sulfate/l).
Practical investigation of the sulfate resistance of concrete from construction units

Fig. 1: Expansion behaviour of flat mortar prisms immersed in sodium sulfate solution (29800 mg sulfate/l). Storage temperature 20 °C

Fig. 2: Expansion behaviour of flat mortar prisms immersed in sodium sulfate solution (29800 mg sulfate/l). Storage temperature 6 °C
The mortar prisms of the mixtures 2 and 3 passed the high sulfate resistance criterion both at 20 °C and 6 °C after 91 days storage in sodium sulfate solution. The prisms of mixture 1 did not fulfill the high sulfate resistance criterion both at 20 °C and 6 °C sodium sulfate solution after 91 days storage as expected.

5.2 Results on Concrete Specimen

The threshold expansion of \( \leq 0.50 \text{ mm/m} \) after 91 days from chapter 5.1 was also taken to assess the resistance of concrete specimen against sulfate attack. Fig. 3 and 4 show the expansion behaviour of concrete drill-out cores immersed at 20 °C and 6 °C in a sodium sulfate solution.

![Expansion behaviour of concrete drill-out cores immersed in sodium sulfate solution (29800 mg sulfate/l). Storage temperature 20 °C](image)

*Fig. 3: Expansion behaviour of concrete drill-out cores immersed in sodium sulfate solution (29800 mg sulfate/l). Storage temperature 20 °C*
The drill-out cores from the comparative concretes (S I, S II, S III) and the drill-out cores taken from the building (S IV) fulfilled the high sulfate resistance criterion of the flat prism procedure at the storage temperature of 20 °C. The measured expansion after 150 days immersion in a sodium sulfate solution (29800 mg sulfate/l) were significantly lower as the threshold value of 0.50 mm/m.

However the drill-out cores from the comparison concrete S I showed a very high expansion at 6 °C and did not meet the criterion for a high sulfate resistance. The other test specimen S II and S III as well as the samples taken from the sports hall concrete presented no significant expansion increase and were not damaged even after a period of 150 days storage.

The expansion of the specimen from comparison concrete S I and the drill-out cores derived from the building (S IV) have also been stored in a long term test for 540 days. Table 6 shows the results after 540 days of immersion.
Tab. 6: Expansion behaviour of the drill-out cores S I and S IV after a storage duration of 540 days. Immersion in sodium sulfate solution at 6 °C and 20 °C.

<table>
<thead>
<tr>
<th>Immersion time [d]</th>
<th>S I</th>
<th>S IV</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>20 °C</td>
<td>6 °C</td>
</tr>
<tr>
<td>150</td>
<td>0.04 mm /m</td>
<td>destroyed</td>
</tr>
<tr>
<td>540</td>
<td>3.46 mm /m</td>
<td>destroyed</td>
</tr>
</tbody>
</table>

Even after 540 days in a 20 °C and 6 °C sodium sulfate solution the drill-out cores from the building (S IV) have still shown a low expansion far beneath the threshold value of 0.50 mm /m. Furthermore no damages were visible in form of cracks or spallings. On the other hand the expansion of comparison concrete S I lay clearly above the threshold value of 0.50 mm /m after 540 days storage in a 20 °C sodium sulfate solution while an immersion at 6 °C has led to total disruption.

Fig. 5 (left) shows the specimen at the beginning and fig. 5 (right) after a storage duration of 150 days. The complete destruction of the drill-out cores S I is shown in fig. 5 (right).

Fig. 5: Concrete drill-out cores at the beginning of the test (left picture) and destroyed drill-out cores S I (right figure) after 150 days immersion.

The destroyed specimen from concrete S I, see fig. 5 (right), show a classical damage by sulfate attack. The damages mainly occurred due to the formation of secondary ettringite and secondary gypsum. In addition to this sulfate attack can also lead to a destructive formation of thaumasite, which weakens the microstructure at low temperatures (< 15 °C) [12]. All three causes for damage by sulfat attack were detected by scanning electron microscope (fig. 6) and x-ray-diffraction (fig. 7).
Fig. 6: X-Ray-Diffraction exposure of a destroyed concrete section (S I) after 150 days immersion in sodium sulfate solution (29800 mg sulfate/l). Storage temperature 6 °C

Fig. 7: Scanning electron microscope – Secondary ettringite and secondary gypsum of a concrete section (S I) in the crack area after 150 days immersion in sodium sulfate solution (29800 mg sulfate/l). Storage temperature 6 °C
6 SUMMARY AND EVALUATION OF THE RESULTS

To determine the sulfate resistance of a sports hall foundation concrete samples (drill-out cores) were taken out of the foundation structures for being compared with lab-manufactured concretes and mortars under varying conditions. Following here the so called flat prism procedure, samples (drill-out cores) from the new building and different composed types of concrete drill-out cores as well as flat mortar prisms were immersed in a sodium sulfate solution (29800 mg sulfate/l). The expansion of the specimen were measured up to 150 days storage as well as 540 days of a long-term storage. With regard to the temperature conditions of the foundation in situ the tests have been carried out not only at 20 °C but also under more severe conditions at 6 °C.

In particular, the tests have shown that:

- Concrete cores made with a blast furnace cement/fly ash mix with a w/c-ratio of 0.50, 0.60 as well as 0.68 (foundation concrete) met the high sulfate resistance criterion of the rapid test in a 20 °C and 6 °C sodium sulfate solution (29800 mg sulfate/l) after 91 days of immersion. The test criterion was even achieved after 150 days storage.

- Concrete cores made with mixtures of blast furnace cement and fly ash with a w/c-ratio of 0.68 (foundation concrete) fulfilled the high sulfate resistance test criterion in a 20 °C and 6 °C sodium sulfate solution after a 540 days long-term storage.

- Concrete cores made with mixtures of ordinary Portland cement and limestone-meal did not meet the requirement of the test criterion in a 20 °C sodium sulfate solution after the 540 days long-term storage.

- Flat mortar prisms made with mixtures of blast furnace cement and fly ash with a w/c-ratio of 0.50 and 0.68 fulfilled the high sulfate resistance test criterion in 20 °C as well as in 6 °C sodium sulfate solution after a 91 days storage.

- Flat mortar prisms made with ordinary Portland cement showed a significantly lower sulfate resistance at 6 °C as well as at 20 °C and did not meet the high sulfate resistance criterion of the flat mortar procedure.
Due to the similar results of the concrete and mortar investigations at a storage temperature of 6 °C it can be assumed that the high sulfate resistance criterion from the flat prism procedure is comparable with the concrete investigations. It can also be concluded that the concrete of the sports hall made with a unduly w/c-ratio of 0.68 showed a similar behaviour under sulfate attack as concrete in accordance to relevant standards.

REFERENCES


LEACHING PROPERTIES OF SELF COMPACTING CONCRETE (SCC)

ELUTIONSVERHALTEN VON SELBSTVERDICHTENDEM BETON (SVB)

COMPORTEMENT D'ELUTION DU BETON AUTOCOMPACTANT (BAC)

Uwe Herterich, Gerhard Volland, Timo Wüstholz, Michael Stegmaier

SUMMARY

In order to obtain the characteristic properties of self compacting concrete (SCC) highly effective water reducing agents (superplasticizers) based on polycarboxylate esters (PCE) are necessary. Compared to conventional concrete higher concentrations of superplasticizer (1-2 % relative to cement) are added.

The determination of mobile organic compounds of PCE-plasticizers in leachates of monolithic SCC-bodies with various w/c ratios (w/c from 0.34 to 0.70) by nuclear magnetic resonance spectroscopy (\(^1\)H NMR) and total organic carbon (TOC) shows that only small ratios of originally added PCE are mobile via ground water. Even under "worst case" conditions (concrete bodies aged for 3 days and deionized water as leachate) less than 2 % of the total PCE are mobile within 56 days. Though their plasticizer concentrations are higher, SCC concretes with lower w/c-ratios (e.g. M85 w/c-ratio 0.34 and plasticizer concentration up to 1.5% relative to cement) lead to significant smaller amounts of leached organic compounds (less than 0.5 % of added PCE). Thus the present investigations give hints that the mobility of soluble PCE components (long-chain polyalcohols) is reduced in concrete with smaller pore radii.

The investigation also shows that transport processes for most of the leached organic compounds are controlled by diffusion processes. For larger organic molecules like long-chain polyalcohols (e.g. polyethylene glycol) as soluble part of admixtures based on PCE combined with concrete with small pore radii pronounced deviations are detectable. After an initial wash-off of surface
bound amounts almost no further emission of those soluble compounds (polyalcohols) can be detected.

This combination of properties of concretes and admixtures leads to types of concrete which can be regarded as suitable concerning environmental compatibility.

**ZUSAMMENFASSUNG**

Zur Herstellung von selbstverdichtendem Beton (SVB), werden hochwirksame Fließmittel auf der Basis von Polycarboxylatethern (PCE) in relativ hoher Dosierung (1-2 % bezogen auf den Zementgehalt) der Frischbetonmischung zugegeben.

Die Bestimmung der mobilen organischen Anteile von PCE-Fließmitteln in den Eluaten monolithischer SVB-Probekörper mit unterschiedlichen w/z-Werten (0,34 ≤ w/z ≤ 0,7) mittels Kernresonanzspektroskopie (1H NMR) bzw. durch Bestimmung des gesamten organischen Kohlenstoffs (TOC) zeigt, dass nur kleine Bruchteile des ursprünglich zugegebenen PCEs über den Grundwasserpfad mobilisierbar sind. Selbst unter “worst case“ Bedingungen (3 Tage alte Betonproben und vollentsalztes Wasser als Elutionsmittel) sind weniger als 2 % des gesamten PCEs innerhalb von 56 Tagen mobilisierbar. SVB-Typen mit niedrigem w/z-Wert (z.B. M85 w/z-Wert = 0,85, Fließmittelgehalt bis zu 1,5 % bezogen auf Zement) weisen trotz ihres höheren Fließmittelgehalts deutlich kleinere eluierte Mengen an organischen Verbindungen auf (weniger als 0,5 % des zugegebenen PCEs). Damit deuten die vorliegenden Untersuchungen darauf hin, dass die Mobilität der löslichen Anteile des PCEs (langkettige Polyalkohole) in Betonen mit kleinen Porenradien deutlich reduziert ist.

Leaching properties of self compacting concrete (SCC)

1. INTRODUCTION

Self compacting concrete (SCC) was developed in the middle of the 1980’s in Japan. SCC flows under its own weight, deaerates and consolidates without adding any additional energy and is resistant to segregation. These characteristics were made possible by the development of highly effective water reducing agents (superplasticizers), usually based on polycarboxylate esters (PCE) (fig.1).

![Polycarboxylate ester (PCE): copolymer of methacrylic acid and methacrylic acid esterified with polyethylene glycol methylether.](image)

KEYWORDS: self compacting concrete, porosity, NMR, TOC, leaching, environmental risks
These superplasticizers consist of a polymethacrylic acid backbone which is partially esterified with polyethylene glycol methylether sidechains. Upon contact with cement there is saponification of the PCE. Of the two resulting parts – polymethacrylic acid backbone and free polyethylene glycol(derivative) – only polyethylene glycol(derivative) is mobile in aqueous solutions [1].

The added superplasticizers are not considered as biologically easily degradable and may not directly discharged in discharge systems, ground- and surface water [2].

The presented investigation serves to quantify mobile substances leached out of monolithic SCC bodies in contact with water as was done for conventional concrete containing naphthaline sulfonate as concrete admixture by other authors [3,4,5]. For the investigations “worst case” conditions were chosen. The obtained results can thus be regarded as maximum values for leaching monolithic SCC bodies in contact with ground water.

2. INVESTIGATIONS

2.1 Investigated concrete mixtures

Cylindrical shaped monolythic bodies of SCC were investigated. Leaching experiments were performed on powder type SCC’s M25 and M85 (strength classes C30/37 and C60/75, respectively) and on the viscosity agent type SCC S25 (strength class C30/37). Technical parameters of the concretes are as follows:

M25: w/c = 0.7, content of superplasticizer: 1.25% by mass of cement
M85: w/c = 0.34, content of superplasticizer: 1.45% by mass of cement
S25: w/c = 0.65, content of superplasticizer: 1.5%. by mass of cement.

Samples were hardened at 20°C for 3 days in closed beakers out of polyethylene. For comparison reasons additionally SCC type M25 without admixtures (and thus not self compacting) was investigated as a reference sample.

The detailed composition of the concretes investigated are shown in table 1.
Table 1: Composition of the investigated concretes

<table>
<thead>
<tr>
<th></th>
<th>M25</th>
<th>S25</th>
<th>M85</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>CEM II/A-</td>
<td>CEM II/A-</td>
<td>CEM II/A-</td>
</tr>
<tr>
<td></td>
<td>LL32.5R</td>
<td>LL32.5R</td>
<td>LL42.5R</td>
</tr>
<tr>
<td>cement content [kg/m³]</td>
<td>240</td>
<td>247</td>
<td>500</td>
</tr>
<tr>
<td>water content [kg/m³]</td>
<td>168</td>
<td>160</td>
<td>183</td>
</tr>
<tr>
<td>lime stone powder [kg/m³]</td>
<td>338</td>
<td>149</td>
<td>0</td>
</tr>
<tr>
<td>fly ash [kg/m³]</td>
<td>0</td>
<td>0</td>
<td>137</td>
</tr>
<tr>
<td>sand 0/4 mm [kg/m³]</td>
<td>752</td>
<td>833</td>
<td>705</td>
</tr>
<tr>
<td>gravel 4/16 mm [kg/m³]</td>
<td>856</td>
<td>959</td>
<td>819</td>
</tr>
<tr>
<td>powder content [kg/m³]</td>
<td>594</td>
<td>416</td>
<td>648</td>
</tr>
<tr>
<td>superplasticizer [% by mass of cement]</td>
<td>1.25</td>
<td>1.50</td>
<td>1.45</td>
</tr>
<tr>
<td>viscosity agent [% by mass of cement]</td>
<td>0.00</td>
<td>0.45</td>
<td>0.00</td>
</tr>
<tr>
<td>(w/c) eq [-]</td>
<td>0.70</td>
<td>0.65</td>
<td>0.34</td>
</tr>
</tbody>
</table>

The manufactured concrete mixes were tested on their rheological behaviour directly after mixing. Table 2 shows the results of these tests.

Table 2: Rheological behaviour of the investigated concretes

<table>
<thead>
<tr>
<th></th>
<th>M25</th>
<th>S25</th>
<th>M85</th>
</tr>
</thead>
<tbody>
<tr>
<td>v-funnel time [s]</td>
<td>11.0</td>
<td>5.0</td>
<td>12.0</td>
</tr>
<tr>
<td>slump flow [mm]</td>
<td>780</td>
<td>720</td>
<td>770</td>
</tr>
<tr>
<td>t₅₀₀ [s]</td>
<td>6.0</td>
<td>4.0</td>
<td>8.0</td>
</tr>
<tr>
<td>slump flow with J-ring [mm]</td>
<td>785</td>
<td>685</td>
<td>730</td>
</tr>
</tbody>
</table>

2.1.1 Investigation of the pore size distribution of the concrete mixes

Prior to the start of the leaching experiments at the concrete age of 3d the pore size distribution of the concretes was investigated by means of the mercury intrusion porosimetry. Therefore parts of the specimens were dried in an oven at 105 °C until there was no further change in mass.
2.1.2 Investigation of the compressive strength of the concretes

The compressive strength of the SCC’s was investigated at the age of 3d and 28d of the mixes. Therefore cubes with an edge length of 150 mm were cast. The specimens for the test at an age of 3d were stored constantly at 20 °C and 100 % r.h.. The cubes for the test at an age of 28d were stored for 7d at 20 °C and 100 % r.h. and 21d at 20 °C and 65 % r.h..

The compressive strength of the SCC’s at an age of 56d was not measured experimentally. But it is possible to calculate this strength with the following formula [6]:

\[
\frac{f_{cm,56}}{f_{cm,28}} = \exp \left\{ s \left( 1 - \left( \frac{28}{56} \right)^{1/2} \right) \right\}
\]

(1)

with:

- \( f_{cm,56} \) = mean compressive strength at 56d [N/mm²]
- \( f_{cm,28} \) = mean compressive strength at 28d [N/mm²]
- \( s = 0.25 \) for CEM 32.5R
- \( 0.20 \) for CEM 42.5R

2.2 Leaching experiments

Deionized water with its higher dissolving power compared to ground water was chosen as medium for leaching. This was done on the one hand to determine an upper limit for possible emissions and on the other hand because of the good reproducibility and the extremely low TOC concentration of deionized water compared to ground water with its diverse consistency.

The eluent was renewed in specified time intervals, to avoid saturation of the analytes and to maintain a high gradient of concentration between concrete matrix and eluent. Time intervals for renewal of eluent were defined according to a modified Standtest Arbeitsprogramm DAFStb [7]: after 1, 3, 7, 16, 32 and 56 days, respectively.
2.3 Investigations of leachates

2.3.1 Determination of TOC emissions

Of the 6 leaching steps conducted, for leaching steps No. 1, 3 and 5 the TOC concentrations of the leachates were determined for all of the four investigated SCC bodies.

2.3.2 $^1$H NMR investigation of aqueous leachates

Contrary to TOC as a sum parameter for all organic components $^1$H NMR measurements of the leachates No. 1-6 allow the identification and quantification of the individual organic compounds.

3. RESULTS

3.1 Pore size distribution

The results of the pore size distribution investigated by mercury intrusion porosimetry are shown in the following figures.

![Fig. 2a: Pore size distribution of M25](image)

![Fig. 2b: Pore size distribution of M85](image)

![Fig. 2c: comparison of the cumulative pore volume of the tested SCC’s](image)
As can be seen in fig. 2c there is a difference in the pore size distribution of M25 and M85. The high strength concrete M85 contains more smaller pores than the normal strength concrete M25. This can be explained by the difference in the \((w/c)_{eq}\)-ratio. There is also a difference in total porosity of these two concretes. M25 has a total porosity of 15.6 Vol.-% and M85 of 13.5 Vol.-% (values also obtained by mercury intrusion porosimetry). This difference in total porosity can also influence the leaching behaviour of the individual concretes.

The varying of pore size distribution and total porosity should have an influence on the leaching behaviour of the SCC’s.

3.2 Strength development of the SCC’s

The measured and calculated compressive strengths of the three SCC’s are shown in table 3. As mentioned before, the values at 3d and 28d are measured compressive strengths while the compressive strength at 56d was calculated by using formula (1) [6].

<table>
<thead>
<tr>
<th>compressive strength [MPa]</th>
<th>3 d</th>
<th>28 d</th>
<th>56 d</th>
</tr>
</thead>
<tbody>
<tr>
<td>M25</td>
<td>25.2</td>
<td>40.2</td>
<td>43.3</td>
</tr>
<tr>
<td>M85</td>
<td>57.3</td>
<td>78.2</td>
<td>82.9</td>
</tr>
</tbody>
</table>

3.3 TOC emissions

Fig. 3 shows TOC emissions of the 4 SCC bodies investigated for leaching steps No. 1, 3 and 5, respectively. In the first leaching step emissions are about twice in magnitude than in successive steps due to the effect of initial wash-off (see below). A comparison of the different samples gives similar TOC ratios for all leaching steps. In total TOC emissions decrease in the following order:

\[ M25 > S25 >> M85 \approx M25 \text{ without admixtures} \]

i.e. sample M85 with the largest added amount of admixtures gives the lowest TOC emissions.
For sample M85 the rapid decrease of the TOC emissions in the course of time is striking. So for leaching step No. 5 TOC emission of M85 is already below that of the reference sample M25 without admixtures whereas in preceding leaching steps M85 gave larger TOC emissions than M25.

A comparison of sample M25 and the reference sample M25 without admixtures shows that about 60% of the TOC emission is due to concrete admixtures.

According to the current stage of discussion on the evaluation of substances hazardous to ground water in Germany, TOC (and DOC) emissions > 20 mg/l measured in a static test after 24 h have to be investigated in more detail. The TOC concentrations of 1-2 mg/l here obtained after 24 h are thus about 1/10 of the presently discussed tolerable TOC in leachates i.e. risks for ground water being in contact with SCC are low.

### 3.4 $^1$H NMR investigation

#### 3.4.1 Qualitative evaluation of $^1$H NMR spectra of leachates

In leachates of SCC bodies with admixtures the soluble part of the active component polyethylene glycol(derivative) and the processing agent p-toluenesulfonic acid as main components of the applied superplasticizer are
identified. For SCC sample S25 of the viscosity agent type additionally components of the viscosity agent (naphthaline sulfonate(derivative), crotonate, 3-hydroxybutyrate, isopropanol) are determined. So besides the proof of the superplasticizer the proof of the viscosity agent in a given concrete body is possible as well.

Furthermore compounds independent of admixtures can be identified: besides the ubiquitous compounds acetate, formiate and lactate ethanolamines (particularly triethanolamine applied as a processing agent in the milling of cement) and mineral oil hydrocarbons are found.

For the four SCC bodies investigated the individual components of the leachates identified by $^1$H NMR are listed in table 4. For a given SCC sample the same species are identified for all leaching steps – there are only differences in the respective amounts quantified.

<table>
<thead>
<tr>
<th></th>
<th>M25</th>
<th>M85</th>
<th>S25</th>
<th>M25 without admixtures</th>
</tr>
</thead>
<tbody>
<tr>
<td>Components of superplasticizer</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Polyethylene glycol (derivative)</td>
<td>+</td>
<td>+</td>
<td>+</td>
<td>-</td>
</tr>
<tr>
<td>Polycarboxylate Backbone</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>p-Toluenesulfonic acid</td>
<td>+</td>
<td>+</td>
<td>+</td>
<td>-</td>
</tr>
<tr>
<td>Components of viscosity agent</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Naphthaline sulfonate (derivative)</td>
<td>-</td>
<td>-</td>
<td>+</td>
<td>-</td>
</tr>
<tr>
<td>Crotonate</td>
<td>-</td>
<td>-</td>
<td>+</td>
<td>-</td>
</tr>
<tr>
<td>3-Hydroxybutyrate</td>
<td>-</td>
<td>-</td>
<td>+</td>
<td>-</td>
</tr>
<tr>
<td>Components independent of admixtures</td>
<td>+</td>
<td>-</td>
<td>+</td>
<td>+</td>
</tr>
</tbody>
</table>
3.4.2 Quantitative evaluation of $^1$H NMR spectra of leachates

For $^1$H NMR intensity of signals is directly proportional to the concentration of the corresponding protons. For this reason besides the above qualitative statements quantitative conclusions can be drawn.

Figures 4 a-d display leached amounts of (a) soluble part of active component (corresponding to 93% of the residue after drying of the initial product) (b) p-toluenesulfonic acid and (c) formiate for leaching steps 1-6.

![Graph showing leaching amounts](image)

*Fig. 4: Amounts of (a) active component, (b) p-toluenesulfonic acid and (c) formiate emitted in leaching steps 1-6*
Fig. 4 (continued)
A comparison of leached amounts of the active component for the different samples shows similar proportions for all leaching steps. With the exception of leaching step No. 1 (here S25 > M25) leached amounts decrease in the order:

M25 > S25 >> M85

Leached amounts of the soluble part of the active component correlate with the respective TOC emissions. Again for sample M85 the rapid decrease of the leached amounts of active component in the course of time is remarkable.

Leached amounts of the processing agent p-toluenesulfonic acid and of components independent of admixtures like formiate correlate with leached amounts of the active component. For all leaching steps leached amounts decrease in the order: M25 > S25 >> M85 for p-toluenesulfonic acid and M25 > M25 without admixtures > S25 >> M85 for formiate, respectively.

The similar behaviour of components of the admixture and of components independent of admixtures shows that primarily the pore structure and thus the w/c-ratio is essential for the leaching properties of SCC bodies i.e. the composition of the concrete (content of cement and admixtures) is only secondary.

Moreover a comparison of leached amounts of formiate of sample M25 with that of the reference sample M25 without admixtures shows that the addition of admixtures leads to higher leached amounts even of components independent of admixtures.

Interestingly the rapid decrease of leached amounts of active component observed for sample M85 cannot be observed (in this extend) for the other leached compounds of M85. This rapid decrease (of leached amounts of active component) leads to a depletion of active component relative to the other components in the leachates in the course of time.

Compared to the original product (Polyethylene glycol(derivative)/ p-toluenesulfonic acid = 30) there is a strong depletion of active component for all leachates investigated (Polyethylene glycol/ p-toluenesulfonic acid = 0.8 –2.7). On the one hand this is due to the much stronger adsorption of the active component on cement grains, on the other hand due to its large molecular size the active component presumably shows the smallest diffusion coefficient.
Fig. 5 displays the leached amounts of active component for the SCC bodies investigated in the course of time. Leached amounts of active component summed up over 56 days range from 140 mg/m² (M85) up to 320 mg/m² (M25).

The respective graphs indicate that for M25 and S25 leaching of the active component is predominantly controlled by diffusion whereas for M85 in the case of the active component the effect of initial wash-off dominates (see discussion).

---

**Fig. 5: Leached amounts (summed up) of active component as a function of time.**

In Table 5 amounts of active component and p-toluenesulfonic acid originally added and amounts of these compounds leached within 56 days are given.
Table 5: Amounts of active component (soluble part) and p-toluenesulfonic acid (p-TSS) added and amounts of these compounds leached within 56 days.

<table>
<thead>
<tr>
<th></th>
<th>Active component added (soluble part) [mg]</th>
<th>p-TSS added [mg]</th>
<th>Active component leached [mg] (after 56 days)</th>
<th>p-Toluene-sulfonic acid leached [mg] (after 56 days)</th>
<th>leached/added amount of active component in % (after 56 days)</th>
<th>leached/added amount of p-TSS in % (after 56 days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M25 (w/c = 0.7)</td>
<td>790</td>
<td>24</td>
<td>15</td>
<td>7</td>
<td>1.9</td>
<td>30</td>
</tr>
<tr>
<td>M85 (w/c = 0.335)</td>
<td>1900</td>
<td>58</td>
<td>6</td>
<td>5</td>
<td>0.3</td>
<td>9</td>
</tr>
<tr>
<td>S25 (w/c = 0.65)</td>
<td>950</td>
<td>29</td>
<td>15</td>
<td>7</td>
<td>1.6</td>
<td>24</td>
</tr>
</tbody>
</table>

For SCC of type M85 (w/c = 0.34) the amount of active component (i.e. soluble part of active component, specifically polyethylene glycol (derivative)) leached over 56 days is about half of that for SCC of type M25 (w/c = 0.7).

Whereas for SCC of type M25 about 2 % (i.e. 15 mg of a total of 790 mg) of the added amount are leached, for SCC of type M85 the fraction of active component mobile in water drops to 0.3 % (i.e. 6 mg of a total of 1900 mg). Although for M85 the added amount of active component is considerably larger (by the factor 2.5) compared to M25, significantly smaller amounts of active component (approx. 50 % of that for M25) are mobile via ground water.

For p-toluenesulfonic acid the recovery rates are as follows: after 56 days 30 % (M25), 24 % (S25) and 9 % (M85), respectively, of the added amounts of p-toluenesulfonic acid are leached. So for M25 the ratio of leached/added amounts of p-toluenesulfonic acid is by the factor 3.5 higher than that for M85 (compared to a respective factor of 5.5 for the active component). Thus p-toluenesulfonic acid can be considered as significantly more mobile compared to the soluble part of the active component.
4. Discussion

For elutions controlled by diffusion leached amounts of substances (E) are time dependent as follows:

\[ E = C \sqrt{t_i} \]

In a logE/logt-diagram ideally a straight line with slope \( \frac{1}{2} \) is obtained.

Yet this kind of presentation has a distinct disadvantage. Due to the summation of leached amounts deviations from diffusion control in a single leaching step have consequences on subsequent leaching steps, too. Thus the interpretation of leaching behaviour in the course of time is made difficult.

Better results are obtained by a differential analysis. Hereby total leached amounts by the end of leaching step \( i \) are calculated on the basis of the leached amounts measured for the single leaching step \( i \). For the calculations pure diffusion control is assumed for the entire leaching process [8].

The respective total amounts can be calculated according to

\[ E_{cal,i} = E_i \sqrt{t_i} / (\sqrt{t_i} - \sqrt{t_{i-1}}) \]

with:

- \( E_{cal,i} \): calculated total leached amount after leaching step \( i \)
- \( E_i \): measured leached amount for leaching step \( i \)
- \( t_i \): total time after leaching step \( i \)
- \( t_{i-1} \): total time before leaching step \( i \)

If the measured and the calculated total leached amounts are identical, the elution mechanism is purely controlled by diffusion.

Figures 6 a-c display logE/logt diagrams of measured and calculated leached amounts (summed up) of formiate (a), p-toluenesulfonic acid (b) and the soluble part of the active component (c).
Fig. 6: Experimental and calculated logE/logt-diagrams for formiate (a), p-toluenesulfonic acid (b) and the soluble part of the active component (c) (pure diffusion control is assumed for the calculation).
An analysis of the leaching behaviour can be done by classification of the leaching steps in 3 different stages according to a modified version of NEN 7345 [8]:

Initial stage: leaching steps 1 and 2 (0-3 days)
Intermediate stage: leaching steps 2 to 4 (3-16 days)
Final stage: leaching steps 4 to 6 (16-56 days)

In experimental and calculated logE/logt-diagrams, respectively for each stage the slopes of the best fit straight lines were considered. If the slopes of the experimental and the calculated data are equal and thus about ½ leaching is controlled by diffusion for the stage considered. A smaller slope of the calculated data compared to the experimental one may be due to the following reasons: in the initial stage it indicates the effect of initial wash-off, in the final stage it indicates the influence of pore size reduction and/or depletion of the component considered.

For formiate interpretation is quite simple (Fig. 6a): for all three samples calculated leached amounts are slightly smaller than the experimental ones due to initial wash-off of surface bound formiate. This is indicated by the smaller slopes of the calculated graphs compared to the experimental ones in the initial
stage. From leaching step No. 2 on graphs of calculated and experimental leached amounts are almost parallel with slopes of about \( \frac{1}{2} \).

Except for initial wash-off formiate shows practically no deviation from diffusion control. This is valid for all three SCC types investigated, independent of their different w/c ratios.

A different situation arises for the larger p-toluenesulfonic acid (Fig. 6b): Again the effect of initial wash-off is observed, to a greater extent for the sample with the smallest w/c ratio M85 and to a lesser extent for sample M25. In the intermediate stage for all samples clearly positive slopes of the calculated graphs indicate that leaching is predominantly controlled by diffusion. In the final stage there are large deviations from diffusion control especially for sample M85 (negative slope of the calculated graph in the final stage). In our opinion this is due to the influence of pore size reduction in the course of hydration.

For the large polyethylene glycol(derivate) molecules deviation from diffusion control is even more pronounced (Fig. 6c): In the initial stage the effect of initial wash-off is generally more noticeable in the order M85 > S25 > M25. In the final stage for all samples large deviations from diffusion control are observed (M85 > S25 > M25), which in our opinion is again due to the influence of pore size reduction. Even in the intermediate stage there are pronounced deviations from diffusion control especially for sample M85.

In conclusion for high strength concrete of the type M85 transportation of the soluble part of the active component is almost limited to the effect of initial wash-off. In the course of hydration the concrete matrix gets more and more dense making transportation of large molecules (e.g. active component) increasingly difficult. This effect is possibly due to the hydraulic binding of the added fly ash (see table 1). So it is described in the literature, that the hydration of puzzolanic additives causes a reduction of the pore volume whereby transportation of moisture and material in the concrete matrix is limited [9,10]. For practical use of this concrete this means that after an initial wash-off of surface bound species there is almost no further emission of active component to the ground water making this concrete highly environmentally friendly.
5. CONCLUSION

- For SCC bodies of the powder type (M25, M85) and of viscosity agent type (S25) leaching experiments were performed according to a modified version of the “Standtest Arbeitsprogramm” proposed by the DAfStb.

- Experiments were performed under “worst case” conditions: age of the concrete bodies 3 days and deionized water was used for leaching.

- The organic content of the leachates was determined
  a) in total by TOC determination
  b) component-wise by $^1$H NMR investigation.

In the following some essential results are compiled briefly:

- In the leachates the soluble part of the active component polyethylene glycol (derivative) and the processing agent p-toluenesulfonic acid as main components of the applied superplasticizer can be determined. For the concrete body of the viscosity agent type (S25) components of the viscosity agent (naphthaline sulfonate (derivative), crotonate, 3-hydroxybutyrate) can be determined additionally. It is thus possible to determine the applied superplasticizer as well as the applied viscosity agent by $^1$H NMR of leachates.

- For all SCC bodies investigated environmental risks are low. Within 24 h the TOC concentration in leachates is about 1-2 mg/l. These concentrations are about 1/10 of the presently discussed tolerable TOC in leachates.

- It was shown that about 60% of the TOC emission is due to concrete admixtures.

- Amounts of leached compounds and TOC emissions decrease in the order M25 > S25 >> M85 ≈ M25 without concrete admixtures. M85 with the lowest w/c ratio (w/c=0.34) and the highest content of admixtures displays the lowest emission rates. So the most important factor for emission is the pore structure and thus the w/c ratio and only to a lesser degree the composition of the concrete (content of cement and admixture).

- Recovering rates for the active component are generally low and vary between 0.3% (M85) and 2% (M25).

- It is shown that transportation is mainly due to diffusion in water filled capillary pores. Deviations from diffusion control are mainly observed in the
initial and final stages of leaching. These deviations are due to the effect of initial wash-off (initial stage) and to the reduction of pore size in the course of hydration (final stage). Effects are most prominent for low w/c ratios (M85) and for large molecules (active component).

- Consequently for high strength concrete M85 transportation of active component is almost limited to the effect of initial wash-off. In the course of hydration the concrete matrix gets more and more dense making transportation of large molecules (active component) increasingly difficult. This effect is possibly due to the hydraulic binding of the added fly ash. For practical use of this concrete this means that after an initial wash-off of surface bound species there is almost no further emission of active component to the ground water making this concrete highly environmentally friendly.

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Leaching properties of self compacting concrete (SCC)
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