Otto Graf Journal
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In July 2003, the two research and testing establishments FMPA and MPA have been reunified. The history starts with the funding of the testing establishment (in German: “Materialprüfungsanstalt”) at the Polytechnical College in 1884. The Polytechnical College changed the name to Institute of Technology (“Technische Hochschule”) and finally to University. The MPA split into two divisions in 1927. One division dealt with mechanical engineering, the other with civil engineering, the first director of which was Otto Graf. After a separation of 76 years, the two divisions joined each other under the name MPA University of Stuttgart (Materialprüfungsanstalt Universität Stuttgart - MPA Stuttgart, Otto-Graf-Institut (FMPA)). The MPA is an autonomous institute under the umbrella of the University of Stuttgart.

This issue contains a large spectrum of topics. There are five papers on non-destructive testing of materials of various kinds. Phase spectroscopy and ground penetrating radar are applied to concrete. A new recorder has been developed for acoustic emission. Acoustic emission has been used for locating sources of interior damage in wood. One paper deals with measurements of the acoustic anisotropy of wood and reveals the sensitivity of velocity variations on burst source location. NMR was a useful means for detection of organic admixtures in concrete. New materials have been developed in the last period of time. These are high strength stainless steel for prestressing of concrete members. Another material concerns fibre reinforced drainage concrete which causes considerable reduction of traffic noise. Self-compacting concrete has been investigated with respect to its fresh properties. Timber-concrete composites are gaining increasing importance in building constructions, whereby the connectors determine the compound efficiency. It is reported on recently MPA-tested systems using nails and nail-plates as connections.

Modeling of materials is an increasing subject of materials research. This issue deals with the modeling of the hydration of hybrid concrete and of the conservation effects of a polymer resin in heterogeneous sandstone. Two fracture mechanics models are compared with respect to their applicability to concrete. Regarding design aspects of constructions, one paper discusses glued
laminated timber beams with rectangular holes. Finally, various subjects are dealt with such as the corrosion damage caused by magnesite floor screed, and one article is devoted to the wide research on concrete roads by Otto Graf.

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 (BMVBW), 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.fmpa.de
OTTO GRAF’S RESEARCH ON CONCRETE ROADS -
A RETROSPECTIVE VIEW

OTTO GRAF’S FORSCHUNG ÜBER BETONSTRASSENBAU -
EIN RÜCKBLICK

LA RECHERCHE D’OTTO GRAF SUR LES ROUTES EN BÉTON -
UNE RÉTROSPECTIVE

H.W. Reinhardt

KEYWORDS: Concrete, road, design, mix, abrasion, traffic safety

SUMMARY

Otto Graf has tested the components of concrete very extensively in the laboratory. Cement was most important which led to a small shrinkage and a high tensile strength. The fineness of the cement was one very important influencing quantity. He investigated the aggregates which should have a high compressive strength, high tensile strength and high resistance against freezing and abrasion. Grading curves were investigated and the compressive strength of concrete was related to the water-cement ratio. SN-curves have been received through tests in the laboratory and in-situ. Loading tests on concrete roads led to the optimum thickness of slabs which was 22 cm in 1939. Many questions of the time of Otto Graf are still relevant however with other boundary conditions.

ZUSAMMENFASSUNG

H. W. REINHARDT

Platten, die schließlich auf 22 cm festgesetzt wurde. Viele Fragestellungen von damals sind auch heute noch aktuell, allerdings mit neuen Randbedingungen.

RESUMÉ

Des essais de laboratoire ont été exécutés par Otto Graf dans une large mesure pour étudier les composants du béton. Une place importante tenait la recherche d’un ciment qui montre un petit retrait et une grande résistance à la tension, une fois durci. Une grandeur importante semblait la finesse de mouture du ciment. Grande attention était aussi donnée aux agrégats qui devraient avoir une grande résistance contre le gel et l’abrasion. La granulométrie du sable et des agrégats a été explorée et on a donné la résistance de compression comme fonction du rapport eau-ciment. Graf a déterminé des lignes Wöhler dans le laboratoire et in-situ. Des essais de chargement des dalles en béton menaient à l’épaisseur optimale qui a été fixée à 22 cm finalement. Beaucoup de questions posées à cette époque sont encore de l’actualité, pourtant avec des conditions nouvelles.

INTRODUCTION

Otto Graf was a pioneer in the field of concrete roads. He has published 53 papers on the subject. He was member of the relevant committees in Germany and has influenced the development of concrete roads very strongly. His work covers concrete technology, design, and traffic safety.

CONCRETE TECHNOLOGY

In 1927, he published a paper on the general requirements on road concrete. There were seven items mentioned: 1. high compressive and high impact strength, 2. high tensile strength to avoid cracks, 3. small shrinkage and swelling, 4. high abrasion resistance, 5. weather resistance, also with repeated freezing and thawing in saturated condition, 6. elasticity with reversible and irreversible deformation, 7. high resistance against chemical attack [1]. It follows that a concrete with crushed ductile rock as aggregate is suited for concrete. The concrete should have a minimum compressive strength of 30 MPa at the age of 28 days. The mortar content should not exceed 55% and the concrete should be cured for a long period.
He states also that concrete with a large amount of coarse aggregate does less shrink than another one with a high content of fine material. However, the workability of the concrete should be improved [2]. Fig. 1 shows a concrete cross-section with 26% by vol. sand in the aggregates [3].

![Concrete cross-section with 26% by vol. sand in the aggregates](image)

Fig. 1. Concrete cross-section with 26% by vol. sand in the aggregates [3]

The use of cement is a very important subject and Otto Graf has formulated seven questions at the beginning of a publication in 1935 [4]:

1. Which properties define the quality of a cement for concrete roads?
2. Are the results of standard tests on cement sufficient for the assessment of the suitability?
3. Which tests are necessary additionally to standard testing?
4. How do we find suitable cements for concrete roads?
5. How can the consumer judge whether a delivered cement is especially suitable for concrete roads?
6. How should the acceptance of cements be organised?
7. Is the development of road cements only important for the road construction?

He found that test results of standard tests are not sufficient to find out whether a cement is especially suited for concrete roads. He designed tests in which the specimens were stored in a cyclic moisture environment. Some cements were superior than others in the bending test. Obviously shrinkage influences the test results such that large shrinkage caused eigenstresses which low-
tered the bending strength. An important criterion is fineness of the cement. Fig. 2 shows the influence of the fineness of the cement on the compressive strength, the bending strength, and on shrinkage.

It is obvious that finer cement increases the compressive strength at 28 days as well as the bending strength. However, a finer cement causes also an increase of shrinkage. Graf requires that the compressive strength of a cement should be 45 MPa at the age of 28 days and the bending strength should be at least 5.5 MPa. In [6], the requirements were given more precisely and Graf points on the cracking, for instance due to capillary stresses during the summer. He states already in 1937 [7] that the production control in the cement factory is as important as the standard testing independent laboratories.

In 1935 [8] he published a paper on the most important parameters which determine the concrete strength. Fig. 3 shows the plot of the compressive strength of concrete as function of the water-cement ratio. It can be seen that the strength is mainly determined by the water-cement ratio. On the other hand, the scatter of results is important. He states also that the sand content of the concrete is important. Especially as the amount of sand exceeds 50%, the strength of concrete is much less.

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1 Throughout the paper the unit kg/cm² should be converted to 0.1 MPa.
Otto Graf carried out cyclic tests. He found that the concrete strength decreases due to cyclic testing, especially as the first million of cycles is concerned. The strength drops from the original value to about 80%. Fig. 4 shows the diagram which illustrates the reduction of compressive strength due to 5 million cycles. It is a Goodman-diagram with the upper stress at the top of the figure.

Shrinkage is of paramount importance for road concrete because it may cause large eigenstresses in a concrete slab. Graf investigated several cements and found in Fig. 5 that there are large differences as shrinkage of cement is concerned. Cement $a$ shows a shrinkage of about $0.8 \cdot 10^3$ whereas cement $e$ shows a shrinkage of $1.5 \cdot 10^3$. As Fig. 5 shows that the tests lasted 10 years.
He investigated also different aggregates in concrete and found that the shrinkage of concrete with furnace slag is small whereas with granite shows a larger shrinkage. After drying the specimens where stored in water and there the swelling of concrete with limestone aggregate is larger than with all other aggregates. In 1936, there appeared a paper on the general properties and requirements on concrete roads [9]. Graf states that the compressive strength should be at least 40 MPa at the age of 28 days, the bending strength should be 4.5 MPa. The slab thickness of concrete roads should be at least 20 cm [10]. He carried out many in-situ tests in order to establish the relation between the slab thickness and the loading capacity. Fig. 6 shows an illustration of the loading devices used at that time. Cars were used as reaction load for the pistons which were mounted under the loading bridge. By using this device he could apply also cyclic load on the concrete road.
DESIGN

As stated before, Graf introduced his papers very often by asking questions [11]. As the design rules are concerned, he formulated three questions:

a) What should be the thickness of concrete slabs for an axial load of 8 tons which occurs very often?

b) Which are the stresses which occur in 15, 20 and 25 cm thick slabs under moving loads? What are the thermal stresses due to climatic changes?

c) Which is the advisable width of joints between concrete slabs and what are the changes of joint width during a day and during a year?

To that end tests have been carried out on the expressways München-Holzkirchen and Stuttgart-Ulm. After testing he decides to design concrete slabs with a thickness of 20 cm. He refers to publications of Westergaard in 1926 [12]. He found that the strains which were predicted by Westergaard’s formulae were smaller than those which were measured in-situ. The measurement on joint widths showed that a mean value of 0.007 mm are most probable for a slab length of 1 m and a temperature difference of 1°C.
The question whether concrete roads should be reinforced with steel was not decided yet [13]. Graf states that reinforcement is only an auxiliary measure since reinforcement cannot suppress cracking. Reinforcement acts when the concrete cracks and the cracking cannot be prevented. So, in the guidelines for the construction of concrete roads (ABB) [14] it is recommended that reinforcement should be used only where the subbase is very weak, for instance on high dams or infills. In these situations the reinforcement ratio should be 6-8 cm$^2$ on the slab width of 3.75 m which is very low. In a later publication [15] the necessary steel cross-section is given with 0.5 to 1% of the concrete cross-section which is more realistic. There should be dowels in the joints. Otherwise, there will be steps on the concrete roads which impair the traffic safety [16]. In 1950, Graf presented a paper which summarises the main questions but also the experiences which were made with the concrete road construction in Germany [17].

**TRAFFIC SAFETY**

Tests were carried out on the friction resistance of concrete roads. To that end a measuring device was designed as a roundabout which is depicted in Fig. 7 [18].

![Fig. 7. Measuring device for the testing of friction between wheel and concrete [19]]
A dry road led to a friction coefficient of 0.63, a wet surface to 0.41 and a slippery road (= dirty) to 0.33. During the test duration the friction coefficient increased.

CONCLUSION

Otto Graf was an exceptional personality [20, 21, 22]. Besides the already mentioned 53 publications on concrete road he published 570 papers on other subjects like reinforced concrete, steel structures and timber structures. Besides his scientific achievements he was a very practical organiser whose testing and research was mainly funded by industry. He was a member of several committees in all areas. So it was possible to publish a guideline for the construction of concrete roads already in 1939 which was a milestone in concrete road construction. The quality of German expressways was very good and the durability was high. At that time it was a challenge to work in this area. To acknowledge his achievements the civil engineering division of the MPA got the name Otto-Graf-Institute in 1953.

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APPLICATION OF THE ULTRASONIC PHASE SPECTROSCOPY ON CONSTRUCTION MATERIALS

ANWENDUNG DER ULTRASCHALLPHASENSPEKTROSKOPIE AUF WERKSTOFFE IM BAUWESEN

APPLICATION DE LA PHASE D'ULTRA-SON SPECTROSCOPIE A DES MATERIAUX DE CONSTRUCTION

Hans-Jürgen Ruck

SUMMARY

There are different techniques for the evaluation of the quality and internal structure of a specimen. The most usual nondestructive methods at the IWB are the acoustic emission technique, the ultrasonic-transmission measurement and the impact-echo method. A more recent test method on construction materials is the ultrasonic phase spectroscopy. By transmitting continuous waves of varying frequency through a specimen, a phase-vs-frequency relation can be measured. Out of this relation one can determine the velocity, the acoustic impedance and the elastic constants of the specimen. Also one can measure the increasing damage during a compression test.

ZUSAMMENFASSUNG

Zur Bewertung der Qualität und inneren Struktur eines Prüfkörpers gibt es unterschiedliche Techniken. Die an unserem Institut am gebräuchlichsten eingesetzten zerstörungsfreien Prüfmethoden sind die Schallemissionsanalyse, die Ultraschall-Transmissionsmessung und das Impakt-Echo Verfahren. Ein neueres Prüfsystem an Werkstoffen im Bauwesen stellt die Ultraschall-Phasenspektroskopie (UPS) dar. Bei diesem Verfahren wird die Phase des Ausgangssignals mit der Phase eines durch den Prüfkörper transmittierten Signals verglichen. Trägt man diesen Phasenverlauf über die Frequenz auf, kann die Wellengeschwindigkeit in der Probe bestimmt werden, woraus sich die akustische Impedanz, sowie die elastischen Konstanten ermitteln lassen. Die Methode eignet sich auch dazu, Daten während eines Druckversuchs aufzuzeichnen und so den Schädigungsverlauf zu verfolgen.
RESUME

Il y a différentes techniques pour l'évaluation de la qualité et de la structure interne d'un spécimen. Les méthodes de nondestructiv les plus habituelles à l'IWB sont la technique acoustique d'émission, la mesure d'ultrasonique-transmission et la méthode d'impact-écho. Une méthode plus récente d'essai sur des matériaux de construction est la spectroscopie ultrasonique de phase. En transmettant les vagues continues de la fréquence variable par un spécimen, une relation de phase-contre-fréquence peut être mesurée. Hors de cette relation on peut déterminer la vitesse, l'impédance acoustique et les constantes élastiques du spécimen. En outre on peut mesurer les dommages croissants pendant un essai de compressibilité. compression test.

Keywords: ultrasonic phase spectroscopy, wave velocity, uniaxial compression test

INTRODUCTION

The wave velocity in materials can be determined by several measurement methods. One of them is the ultrasonic phase spectroscopy (UPS) also known as the π-phase-comparison method. At the beginning of the 70’s the method was described by Lynnworth et al. [1]. Their experimental equipment existed out a frequency oscillator, frequency counter, wideband amplifier and an oscilloscope. With such a lot of instruments the calibration was a delicate thing and take a long time. Today modern network analyzers allow a very simple measurement procedure [2] and enables an automatically calibration of the equipment. So the method can be used to follow the increasing damage during an uniaxial compression test [3].

This article describes basics of the UPS and also the analyzing methods of the detected data. Also UPS be compared with results of the ultrasonic pulse spectroscopy and acoustic emission tests.

ULTRASONIC PHASE SPECTROSCOPY

The equipment used for UPS is very compact. In addition to the network analyzer [4] one needs two ultrasonic sensors with a broadband frequency spectrum and a linear run of the phase curve. Now a wave with the frequency \( f \) transmitted through a specimen with the length \( L \) be compared with the input signal. Because the specimen has the same function as a “velocity-resistor”, a runtime difference between the input and output signal occurs.
An increasing of the frequency is finally leading to continuous phase shift $\Delta \Phi$. If the phase plotted against the frequency, one can determine the velocity $V$ out the gradient of the curve [5]:

$$V = -2\pi L \left( \frac{df}{d\Delta \Phi} \right).$$  \hspace{1cm} (1)

The programme UPAS [3] was developed to automate the data acquisition which controls the network analyzer Advantest R3754A. The program, which was written under LabVIEW 6i, calibrates the ultrasonic sensors to get a linear amplitude distribution against the frequency. Also it stores the results in the ASCII format which allows a data analysis in the most commonly used programmes.

During a compression test more than 200 individual events can arise. This is called an additional expenditure in the analysis time and data evaluation. Therefore a software was programmed, in order to determine the gradient of the curve automatically.

An other problem is the coupling of the sonensor on the specimen and so the choice of a suitable ultrasonic couplant [6]. It must have a high bandwith and a low damping factor. The following figure 2 demonstrates the different damping of some coupling agents, as wax, medical ultrasonic gel, Butyl seal tape and
hot-melt adhesive. The frequency quality and the low damping caused by the thin liquid layer of ultrasonic gel be opposed to the less adhesive property on materials like concrete or stone. These materials comparable with a sponge absorb the liquid part of the gel and make the coupling time-dependent. in the beginning hot-melt adhesive is liquid and enables a thin film between the coupling layers. The hardened glue has a high acoustic impedance what allows a good acoustic adaptation. It is the preferred couplant for rough surfaces. The hot glue clings very good on different materials, but it is so good that it is very difficult to separate the face to face together-glued sensors after the calibration because the coupling layer is too big. Wax is equal to hot-melting adhesive but it is softer in hardened condition, thus the separation-procedure of the sensors is less complex. Also the run of the curve in higher frequency areas is better. The handling of wax is uncomfortable because it hardened very fast. A compromise between these couplants agents is Butyl seal tape. Because Butyl is kneadable, its handling is simple and the adhesion characteristic is good, which can be improved by Butyl primers. To avoid the loss of the time dependent adhesive, one must provide a constant contact pressure.

![Damping curves of different coupling agents. The illustration on the right side shows a test specimen, which is coupled according to the new method with liquid plastic and ultrasonic gel.](image)

There are many different coupling agents with different qualities. The correct choice of the agent depends on the specimen surface quality and the sensor type. Especially for concrete there exists a new coupling method. The specimen
surface be pretreated with clear varnish or liquid plastic. Thereby a flat dust-free layer on which one can couple with ultrasonic gel results. The sensors must be fastened by a holder.

**ANALYSIS METHODS**

During a compression test the network analyzer scans every 10 seconds a frequency range and recorded both the phase and linear amplitude over the frequency. All data result from a uniaxial compression test on a concrete cube with side length 15 cm, an aggregate corn of 1,2/2 mm, a steel fiber (Dramix ZP 305) fraction of 0,4vol.-% and a water/cement ratio of 0,4. Out of these data a programme calculates the wave velocity out of the gradient of the phase curve (Figure 3 left). Therefor is only used the linear increasing part of the curve. The corners of the linear part signifies highest and lowest transmitted frequency. How one can see both the velocity and the maximum transmitted frequency decrease with increasing damage. The frequency range depends in the lower part on damping-effects of the specimen like interferences, side-wall reflections and complexe modes, in the higher part on the bandwidth of the sensors [7]. If the slope $\frac{df}{d\Delta \Phi}$ is constant, as in the frequency range of this experiment, dispersion is not significant.

![Figure 3: Phase as a function of the frequency and the velocity as a function of the deformation. The data are a result of an uniaxial compression test at a steel fiber reinforced concrete cube.](image)

The right side of figur 3 represents a result of the velocity analysis for a compression test. In the beginning the speed decreases slowly, until the cracks gain strongly. At this point the microcracks are connected. Because there is now
no direct way for the waves from sensor to sensor the velocity decreases strongly.

Also one can analyse the frequency spectrum over the time and the damage increasing. The network analyzer scans every 10 seconds a frequency range. This is like a fourier-spectrum of the transmitted signal. All spectra side by side form a flow chart and also a intensity diagramm by top view how figure 4 on the left side shows. Dark colours are low amplitude value. The specimen has a length $L$ and a material dependent wave velocity. Thus frequencies, which correspond to one of the multiples half wavelength, are preferred. They form standing waves which transmit more energy to the other side. This effect shows the 3D graphic on the right side in picture 4. Like in the mountains there are peaks and valleys whereby the standing waves correlate with the peaks.

![Frequency Spectrum](image)

\textit{Fig. 4: Frequency spectrum as a function of time. On the right side a zoomed 3D-graphic where the mountain structure clearly stand out.}

Higher frequencies are more strongly absorbed with increasing damage, since they are already disturbed by smallest cracks. This can be observed also at the transmitted amplitude of the signal. The picture on the left presents the change in amplitude versus the deformation for whole and different frequency ranges. The amplitudes react sensitiver to damages than the velocity. With a special developed programme one can separate special frequency ranges and calculate the amplitudes. As expected amplitudes of higher frequencies decrease faster with increasing deformation.
ULTRASONIC PHASE SPECTROSCOPY AND ULTRASONIC PULSE SPECTROSCOPY

With the ultrasonic pulse spectroscopy (UIS) one can determine the amplitude, frequency spectrum and the velocity of a specimen during a compression test too. But in contrast to the ultrasonic phase spectroscopy (UPS) a calibration is impossible. The advantage of the UIS is the high energy of the input signal so specimens with a bigger thickness can be measured. The frequency range be limited by the bandwidth of the sensor and the pulse length of the input signal. In this study the sensors (Vallen VS 30) mainly dictated this range (10-60 kHz). In this study both measurement methods applied at the same time to one specimen. A result of the good coupling of the UPS sensors with liquid plastik and ultrasonic gel as well as the insensibility of the UPS equipment against external interferences. For the UIS experimental setup (figure 6) an ultrasonic emitter and four receivers (for a better area cover) are used. The setup of UPS is the same as in the chapter before. Additionally there are two position encoders on the sides of the UPS sensors. Thus the specimen surface leaves no more place.

A problem is the detection of the amplitude of the transmitted pulse waves because they are with other oscillations superimpose. From there the onset time must be determined, so that the first vibration in the signal can be separated and the amplitude can be extracted. The peak of this amplitude corresponds with the longitudinal wave considered for the analysis [8].
To define the onset time of 1000 and more signals, a programme was developed that calculates that point of time automatically via the Hinkley-criterion with a dynamic threshold-control [9]. This algorithem makes a good job when the transmitted signals are low-noise. In the reality the detected signals are superimposed with noise from the measurement equipment and external interferences. Additionally the calculated onset times are filtered by a smooth-programme. With the smoothed dataset the point of time is searched in the original data when the longitudinal wave arrives. A comparison with the energy, the square of the signal amplitude, and the maximal transmitted amplitude (figure 7) shows that specimen intern activities during the compression have high influence on the results. It seem that in compressed areas the signal energy increases. If the internal structure finally gives way, the energy or maximally transmitted amplitude makes a jump downward, until it comes again to a compression. In contrast thereto the first amplitude of the longitudinal wave decrease constantly.

The next picture shows a comparison between the velocity determination by UPS and UIS. At the beginning both curves have the same gradient. The velocity of the UPS measurement has a higher value due to the not exact definition of the onset time and associated failure in the determination. With about 0,18 millimeters the UPS curve breaks in, because the absorption in the test specimen becomes too large due to internal damages.
The input signal of the UIS has sufficient energy, in order to cross smaller cracks. Therefore also a more homogeneous form. On the right side of figure 8 the frequency spectrum is pictured. The bandwidth of the sensors lies between 5 and 60 kHz. Because no calibration was made, the resonance frequencies of the sensors at 10 and 50 kHz are clearly identifiable. Between this range the resonances of the specimen are visible as with the UPS method but in the lower frequency range.
Fig. 8.: Velocity versus deformation determine with the pulse spectroscopy (black) and the UPS (grey). Right the frequency spectrum result from the IPS data.

ULTRASONIC PHASE SPECTROSCOPY AND ACOUSTIC EMISSION

Acoustic Emission Analysis (AEA) is another non-destructive testing method [10]. AE occurs if a crack growth or crack borders rub against. Therefor the test object must be compressed or stressed. The method detects the acoustic events and can identify, locate, and display the damage to the tested object [11]. In this study a concrete cube (aggregate 1,2/2 mm, steel fiber fraction 1,3 Vol.-%, w/z 0,4) was chosen on which a localisation of the acoustic events was realized. To create acoustic emissions the cube underwent an uniaxial compression test with different load levels. To determine the velocity and the maximal transmitted frequency the cube was scanned by the UPS after every test and the acoustic events were located. For the localization are used the velocity-results from the UPS analysis. The next picture shows the experimental setup of AE. Eight sensors recorded the emission signals, two sensors triggered the events. Additionally two position encoders were mounted. At first the specimen was scanned by UPS then loaded from 0 to 400 kN. AE measurements were taken during the loading. With the data the localization for the several events were calculated. Now the load was enlarged by 100 kN and the specimen was loaded again.

Figure 9 shows the damage development with increasing loading. The maximal transmitted frequency reacts sensitiver to little changes in structure. But it is sensitive with regard to the coupling too.
So the interpretation of the results is not easy and needs good experiences. The results of both measurements methods harmonize very good. The test specimen breaks at the right side, which at the lightening color distribution in the speed and frequency diagram becomes evident. This be emphasized by the increase of the acoustic emission on the right side. The white areas in the velocity and frequency diagram stand for a impossible measurement of transmission. A more exact examination of the specimen yielded that the cube was sloping.
Fig. 9: Result from the UPS, velocity $c$ and maximal transmitted frequency $f$, and from AEA in $y$-$z$ direction of the specimen.
CONCLUSION

A new non-destructive measurement method, the ultrasonic phase spectroscopy, was tested on construction materials. The experimental setup today is very simple with network analyzers which enables a calibration of the measurement equipment and an automatically operational sequence of the measurement by programs. The increasing damage can be observed during the compression test and several data can be calculated. So the velocity, the amplitude for the whole frequency range and special ranges, the maximal transmitted frequency and additionally the resonance frequencies of the specimen. The comparison with the ultrasonic pulse spectroscopy and the acoustic emission analysis yielded good agreements. So the measurement method can be adapted on all materials for non-destructive testing so long as the damping threshold is not reached. In the future improving of the coupling and a more exact deformation determination is planned.

ACKNOWLEDGEMENT

The author is grateful to the German Research Society (Deutsche Forschungs Gesellschaft) for the financial support in SFB 381, Project A9, under the scientific responsibility of Prof. Dr. H.-W. Reinhardt.

REFERENCES


INTRODUCTION OF A GROUND PENETRATING RADAR SYSTEM FOR INVESTIGATIONS ON CONCRETE STRUCTURES

VORSTELLUNG EINES BODENRADARSYSTEMS FÜR UNTERSUCHUNGEN AN BETONBAUTEILEN

PRESENTATION D'UNE SYSTEME RADAR POUR LA RECHERCHE DES CONSTRUCTIONS EN BETON

Finck, Florian

SUMMARY

Today, Ground Penetrating Radar (GPR) is a wide spread method for the investigation of building ground and engineering structures. The method allows for a rapid scanning of even long profiles and after some standard data processing, results can be evaluated on-site. Shallow investigations can be performed with high resolution, but an increasing target depth is at cost of resolution because of the attenuation of high frequencies. This year, the Institute of Construction Materials acquired a GSSI SIR-2 GPR system. Antennas with a centre frequency of 500, 900 and 1600 MHz are well-suited for various applications in civil engineering. In the following, some basic principles, the radar system and first measurements are presented.

ZUSAMMENFASSUNG

reich des Bauwesens. In diesem Beitrag werden einige Grundlagen, das Radar-
system und erste Messungen vorgestellt.

**RESUME**

La systeme radar aura aujourd'hui utilise pour la recherche des batiments et
de la terre. Avec ce procede il est possible de faire des grands mesurages vites et
apres une exploitation standard des donnees une evaluation est possible sur
place. On peut faire des mesurages pres de la surface avec une resolution tres
haute. Mais quand la profondeur d'exploration augmente a cause du assourdis-
sement des ondes hautes frequences electromagnetiques la resolution descende.
Depuis cette annee l'institut des materiaux pour le batiment possede une systeme
radar SIR-2 de GSSI. Les antennes ont des frequences de 500, 900 et 1600 MHz.
Elles permettent plusieurs utilisations en science des batiments. Dans ce article
des principes basiques, la systeme radar et des mesurages auraient presente.

**KEYWORDS:** ground penetrating radar, concrete, resolution

1. **INTRODUCTION**

Ground Penetrating Radar (GPR) is a high resolution electromagnetic tech-
nique that is designed to investigate the shallow subsurface of the earth, building
materials, and roads and bridges. Target objects are buried pipes, cables and re-
inforcement, caverns, flaws and cracks, as well as ground water and moisture,
etc. Resolution of GPR primarily is a function of antenna frequency and the di-
electric constant of the medium. With standard equipment, a resolution of much
better than 1 cm can be achieved. On the other hand, resolution is obtained at the
cost of penetration depth, since attenuation of electromagnetic waves in com-
mon building materials is rather high and increasing with frequency. Usually,
GPR is applied in reflection mode, which yields a cross section of the subsur-
face, where electromagnetic waves are scattered and reflected at the target ob-
jets. Depth and shape of the objects are calculated from the runtime of the re-
lected signals over a profile. Measurements can be performed in a rapid scan-
ning mode and the combination of various B-scans produces pseudo 3D images
of the subsurface. When the object is accessible from more than one side, inves-
tigations can be carried out also in transmission mode yielding a tomography.
The theory and application of GPR technique is similar to the one of the seismic
reflection method and programs for the data processing are commercially avail-
able.
2. **BASIC PRINCIPLES**

In the early 20th century, the reflection of electromagnetic waves on metallic and non-metallic surfaces was used to detect barriers and objects and to evaluate the distance. During the 2nd world war, *Radio detection and ranging* (radar) technology was enhanced for military applications. After the war, radar became soon an important tool for navigation, weather forecasting, etc. Since the 1950th, radar waves were used to estimate the thickness of arctic ice masses and glaciers. From the 1970th, also the structure of the shallow subsurface was investigated and first ground penetrating radar systems were commercially available.

A basic radar system consists of a control unit, an impulse generator, one or two antennas for the transmission and the receiving of the signals and a memory. The impulse generator sends short electric pulses (~1 ns) to the antenna with a rate of 50 kHz or more. For each of these pulses, the transmitter radiates an electromagnetic wave into the subject. The propagation and reflection is ruled by the electro magnetic parameters of the medium, which is discussed further down. However, reflected electromagnetic waves are detected by the receiving antenna. The acquisition and digitalization of these very high frequency signals is performed using a trick (fig.1).

![Diagram](https://example.com/diagram.png)

*Fig.1: Scheme of step wise sampling a high frequency radar signal.*

The trace of one point measurement is built by stringing together numerous single samples evaluated by the single impulses, which are sent with a rate of 50 KHz. The time delay between the samples is realized electronically. As an example, a point measurement with a trace of 512 samples needs approximately 0.01 seconds. This is fast enough to move the antenna over the profile and still achieve a good lateral resolution. In the single measurements, usually, the flash-over of the transmitter impulse and the reflection of the surface are inherent in
the beginning of the trace. The reflections of the subsurface are following. Fig. 2 represents an example of a single measurement.

Fig.2: Example of a signal from a point measurement.

A sequence of single measurements then yields a B-scan, or radargramm. For the visualization, which can be plotted on-line, amplitudes exceeding a threshold are plotted in black (wiggle method), or the amplitudes are coded by a colour. The traces are stored in the memory of the control unit. The processing is performed after the measurement.

The physical principles of electromagnetic waves are based on Maxwell’s equations [e.g. Gerthsen and Vogel, 1993]. For the electric field $\mathbf{E}$ the formulation,

$$\Delta \mathbf{E} = \varepsilon \varepsilon_0 \mu_0 \mathbf{E} + \mu_0 \sigma \dot{\mathbf{E}}$$

(1)

can be derived, where $\varepsilon$ is the relative dielectricity and $\sigma$ the conductivity of the medium. The formulation for the magnetic flux $\mathbf{B}$ is analogous. Eqn.1 is solved by the approach of an attenuated ($\delta$) plane wave, propagating in $x$-direction. $k = \omega / c$ is the wave number, where $\omega$ is the angular frequency and $c$ the vacuum velocity of light:

$$\mathbf{E} = E_0 \mathbf{e}^{i(\omega t - kx)} \mathbf{e}^{-\delta x}$$

(2)

leading to the complex expression
\[ (-\delta - ik)^2 = -\mu_0 \varepsilon_0 \omega^2 + \mu_0 \sigma i \omega, \]  

which can be decomposed into two real equations:

\[ k^2 - \delta^2 = \mu_0 \varepsilon_0 \omega^2 \]  

\[ 2\delta k = \mu_0 \sigma \omega. \]  

The first equation is the displacement term, governing at very high frequencies, and the second the conduction term, which is prominent at low frequencies. The frequency spectrum of GPR waves lies in between 10 MHz and 2.5 GHz and both terms have to be taken into account [e.g. DANIELS, 1996; REYNOLDS, 1997].

With the velocity \( v \) of the waves in the medium

\[ v = \frac{c}{\sqrt{\varepsilon}}, \]  

refraction of electromagnetic waves is formulated by Snell’s law:

\[ \frac{\sin \alpha_1}{v_1} = \frac{\sin \alpha_2}{v_2}. \]  

Electromagnetic waves are also reflected on material boundaries and inhomogeneities. For high frequencies and a normal incidence, reflectivity \( R \) is only dependent on the relative dielectricities of the media:

\[ R = \frac{\sqrt{\varepsilon_2} - \sqrt{\varepsilon_1}}{\sqrt{\varepsilon_2} + \sqrt{\varepsilon_1}}. \]  

Typically, reflections of inhomogeneities occur in the shape of hyperbola. The runtimes \( t_r \) for electromagnetic waves can be calculated by

\[ t_r = \frac{1}{v} \sqrt{x^2 + 4h^2}, \]  

where \( x \) is the lateral offset and \( h \) the depth.

Attenuation \( \delta \) of electromagnetic waves is given by

\[ \delta = \omega \sqrt{\left( \frac{\mu_0 \varepsilon_0}{2} \right) \left( 1 + \frac{\sigma^2}{\omega^2 (\varepsilon_0 \varepsilon)^2} \right)} - 1. \]
With eqn.9, the skin depth $\Delta = 1/\delta$ can be calculated, where the electromagnetic field has decayed with $1/e$. This parameter is often used to describe the investigation depth.

An important role plays the resolution of the electromagnetic waves. As a rule of thumb, the Rayleigh resolution limit is defined by $\lambda/4$, where $\lambda$ is the wavelength. Of course the signal to noise ratio also governs the significance of the measurements and various processing tools and filters are available.

3. **RADAR SYSTEM SIR-2**

The SIR-2 GPR System, distributed by Geophysical Survey Systems Inc. (GSSI), was one of the most popular radar systems in the 1990’s. It is very robust and user friendly and can be adapted to numerous applications. SIR-2 runs under a DOS operating system. In fig.3, the SIR-2 control unit is presented (down left) with the power supply and an IDS (Ingegneria Dei Sistemi S.p.A., Italy) 1.6 GHz antenna with survey wheel for high resolution investigations.

![Control unit SIR-2 with power supply and 1.6 GHz antenna by IDS.](image)

A 500 MHz antenna with handle bar and a 900 MHz antenna are presented in fig.4. A frame with a survey wheel will soon be developed for these antennas as well.

Fig. 4: 500 MHz antenna with handle bar and 900 MHz antenna (both by GSSI).

The system requires a voltage of 12 V with 3 A, which can be provided by a car battery in the field or by an adaptor via the net.

In the set-up menu all parameters about the antennas, known material constants and the measuring procedure are defined. The measurements can be triggered manually, or better by the survey wheel. The latter provides equidistant scans along the profile. The collected data is plotted on the display on-line, and saved on a hard drive. By that, a first evaluation of the results can be performed on-site. After the measurements, the data is transferred to a personal computer via a serial data bus [SIR System-2, Operation Manual, 1996].

For the final processing of the data, the software packet ReflexW by Sandmeier [2002] offers a lot of options. The data can be filtered and migrated, to reduce geometrical effects and enhance the significance of the results. Pseudo 3D images can be generated for suitable data sets.

4. EXAMPLE OF MEASUREMENTS

Investigations were performed on a cascaded specimen (fig. 5) with faults in various depths. This specimen was built for ultra sonic and impact echo measurements [Ruck and Beutel, 2001]. Geometrical effects of the numerous edges of the specimen caused problems in ultra sonic investigations. One profile ran over the center of the specimen, starting on the shallow step towards the deepest step. The measures of the specimen are given in fig. 5, where \( d \) is the thickness of the various steps and \( t \) the depth of the faults in cm, respectively.
Results of these investigations are presented in fig.6 and 7. The pseudo depth axis on the right is adapted to the measured data via a calibration measurement on the specimen, which revealed a electromagnetic wave velocity of approximately 0.1 m/ns, or an $\varepsilon_r=9$.

The raw data in fig.6 show four prominent reflection hyperbola, representing the faults. The second reflection (from the left) is weaker than the others and appears slightly under the connecting line of the other reflections. No information
of the signals can be assigned to back wall reflections. The data from the shallowest step is relatively noisy. To enhance the quality of the image and its significance, a Kirchhoff-migration [e.g. Conyers and Goodman, 1997] was performed on the data. This migration corrects the data for geometrical side-effects. The prism shaped faults become clearer and the hyperbola vanish. Also the back wall reflections of the shallowest and the second step are clearly visible.

Fig. 7: Migration of the section presented in fig. 6.

The reflection of the fault in the second step is weak again. This fault is not placed in the center of the specimen, so the reflections, which are visible in the data, are generated by the side of the fault. This is also the reason, why this fault appears deeper than it is.

This example shows, that the radar system is very well-suited for high resolution investigations. After the migration, also geometrical effects can be minimized. The positions and depths of the reflectors do very well correlate with the real dimensions of the specimen. On the other hand, the limited depth of investigation is obvious. The back wall of the third step is expected in a depth of 45 cm, but there is no clear reflection, only a weak shadow. For these target
depths, additional measurements with an antenna with a lower frequency would be necessary.

5. CONCLUSION

The SIR-2 GPR system allows for small and mid-sized investigations of concrete and stone constructions, building ground, tunnels, etc. The antennas with frequencies of 500, 900 and 1600 MHz offer a wide range of resolution and penetration depth. Within the frame of a diploma thesis, studies on the resolution of the system under varying conditions are performed.

REFERENCES


ABSTRACT

Tests with high strength stainless steel strand justify the use of strength class S 1100 spiral strands made from the material 1.4436 under moderate chloride and sulfur dioxide load. The findings gained from tension members made of high strength stainless steel wires seem to indicate that strength class S 1100 strands made from the material 1.4401 should only be used in constructions of negligible chloride and sulfur dioxide contents. This assessment takes into account the behaviour compared with the most important types of corrosion such as pitting and crevice corrosion as well as stress corrosion cracking under the known marginal conditions for tension members in accordance with DIN 18800 part 1.

ZUSAMMENFASSUNG

RESUME

Des essais sur des torons en acier inox justifient l’utilisation de câbles de classe de résistance s 1100 en 1.4436 sous action modérée de chlorures et dioxyde de soufre. Les résultats obtenus pour les membres de tension acier inox de haute résistance indiquent que les câbles en 1.4401 de classe de résistance S 1100 ne doivent être utilisés uniquement dans des constructions sans teneur notable en chlorures et dioxyde de soufre. Cette estimation prend en compte le comportement face aux principaux types de corrosion tels que la corrosion par piqûres, la corrosion par fissuration et la corrosion fissurante sous tension sous les conditions définies dans la 1ère partie de la DIN 18800.

KEYWORDS: High strength steel, austenitic stainless steel, tension members, spiral strands, mechanical properties, corrosion behaviour, pitting corrosion, fretting corrosion, stress corrosion cracking, corrosion fatigue.

1. INTRODUCTION

In 1998, the German Institut für Bautechnik in Berlin, responsible for construction supervision, awarded the general certification for stainless steels [1] which regulates the use of these materials and their products under typical corrosion conditions when used in the building and construction industry. The certification relates to typical product items for steels with and without work hardening up to yield or tensile strengths of 690/850 N/mm². The listed types of steel meet the requirements for certain corrosion exposures and are broken down into resistance classes against corrosion.

High strength tension members in accordance with DIN 18800-1 (steel structures, dimensioning and construction) are not the subject matter of the general certification for stainless steels. Cold deformed steel wires with yield strengths of > 1000 N/mm² are not covered by the empirical range of the certification as laid down by the strength spectrum and with regard to known corrosion properties. Since in the meantime high strength steels on the one hand have become important alternative materials for load-bearing structural elements and on the other some innovation developments have taken place in this field some of the implications shall be mentioned here. The high strength tension member made of stainless steel will be highlighted as a good example of enhanced engi-
neering. Meanwhile the German Institut für Bautechnik has awarded a certification for this construction product.

2. THE FREE TENSIONED TENSION MEMBER IN STRUCTURAL ENGINEERING [2, 3]

The tension elements consisting of steel wires for use in structural engineering are so-called "static ropes" which in contrast to "running ropes" in the hauling and hoisting will not be moved nor ridden upon. As opposed to other construction methods (steel structures, concrete constructions) ropes offer advantages. At relatively low cross-sectional dimensions they can transfer high tension forces and bridge large span lengths. Considerably higher strengths as opposed to construction steels can be utilized due to high tensile loads. Low wire dimensions and the type of rope construction have the effect that tension members can practically be considered as resilient (flexible).

Ropes consist of a great number of cold worked high strength steel wires which lie on top of each other in layers. Conventional ropes are manufactured from unalloyed carbon steel; the wire strength lies at 1700 N/mm$^2$. In tension members made of stainless steel wires the strength exceeds 1450 N/mm$^2$.

With regard to structural tension members a distinction is made between different types of construction (Fig. 1):

**Spiral strands** are of a design which are wrapped counterrotating in the individual layers of wire (single stranding) whereby the wires of one layer all form the same spiral shape. Open spiral strands (Fig. 1 a) are manufactured exclusively of round wires mainly of identical diameter. In the case of up to 3 layers applied to a core wire ($1 + 6 + 12 + 18 = 37$ wires) they are known as steel strand. Fully locked spiral strands (Fig. 1 b) receive further layers of preformed Z-profile wires over a core with several layers of round wires.

In general, open spiral strands made of round wires will rather be employed to bear minor loads at lower requirements to stiffness (as net ropes for wide-span structures or for guying in the event of pylons or in bridge construction). In contrast, the main operational range of fully locked spiral ropes is to be found with structures under higher stress such as main cables in bridges, edge ropes of wide-span structures. Whilst the application of profiled wires and their anchoring in sealing bodies (metal grouting or epoxy grouting plus steel balls) hampers the ingress of corrosion promoting aqueous media into the rope interior of
locked spiral strands since the joints between the outer wires will close under load, crevices and capillary cavities can hardly be prevented in open spiral strands and also in their cable clip joints, so that humidity can penetrate in between the single wires. In a corrosive environment this can result in crevice corrosion.

Apart from the ropes, stranded ropes (Fig. 1 c) are frequently used in constructional engineering consisting of one or several layers of steels strand, wound in spirals around a core. The multiple stranding applied in this case leads to a flexible tension member. However, the ensuing low metallic cross-sectional area factor of the rope, its low stiffness and its necessarily very small diameter increase the so-called spinning loss factor and impede corrosion protection.

Very big loads can be beared by bundles of parallel-arranged wires of steel strands (Fig. 1 d).

This paper deals with open spiral strands made of stainless steel wires representing 90 % of the application of tension members made of stainless steel wires and which are used almost exclusively under corrosion promoting conditions. In this case round wires sized 0.6 to 3.5 mm are processed to ropes of a diameter of 3 to 38 mm.

Up to now in the building and construction industry only austenitic stainless steel wires mad from the materials 1.4401 (X 5 Cr Ni Mo 17-12-2) and
1.4436 (X 4 Cr Ni Mo 17-13-3) with characteristic tensile strengths of $\geq 1450$ N/mm$^2$ and 0.2 %-proof stress $\geq 1100$ N/mm$^2$ (strength class S 1100) are in use. The elongation at break $A_{10}$ of these ropes is of 6 % and the uniform elongation of $\geq 2$ %. A maximum creep strain of $2.5 \cdot 10^{-2}$ % must be expected for load of 40 % of the tensile strength at ambient temperature after 1000 hrs.

The wires a the straight wire rope under tensile load are mainly strained by tensile stresses. Apart from that and as a result of the stranding, anchoring and deflection additional stresses occur which can have an influence on the bearing capacity, the working line, the creeping, the dynamic behaviour and also on the corrosion behaviour of the wires in the rope.

Under fatigue loading and under transverse stress a friction of the wires among themselves as well as at the anchorages, cable clips and deflections occurs. This has an adverse influence on the fatigue behaviour of the ropes.

3. ROPES MADE OF STAINLESS STEEL WIRES

3.1 Reasons for an application in structural engineering

Tensioned ropes made of high strength steel wires are subjected to static and dynamic stresses as well as various corrosive influences. With regard to a high durability they must therefore be sufficiently dimensioned and also protected against environmental impact when using corrosion susceptible wires made from unalloyed steels. So far such rope wires or ropes were protected by means of metal or polymer coatings. Presently the state of the art is a combination of both methods (metal plus paint-system). Such corrosion protection systems, however, may fail under extreme and mostly unforeseeable mechanical and/or corrosive stresses [4, 5-7] as a consequence of non-uniform corrosion, stress corrosion cracking, fretting and fatigue corrosion. Particularly in areas of load application at anchorages, cable clips diversions (saddle) it is possible that these protection measures are not sufficient even under "normal" environmental influences.

Hence it was apparent that by using high strength wires made of sufficiently high alloyed stainless steel one could dispense with additional corrosion protective measures and at the same time make allowances for esthetical aspects. Cold-drawn wires made from stainless steel have been manufactured for some
20 years with comparable mechanical properties and fatigue strength just like high strength wires made of unalloyed steels [8-10] and also further processing to ropes is not difficult. In practical use the ropes have stood the test also from a corrosion point of view (Section 2.3) and have given proof of their high resistance in laboratory and nature tests.

### 3.2 Manufacture of wires

Because of the required increased resistance to corrosion mainly austenitic steels of the chrome-nickel type are suitable for manufacturing the wires in the course of which with regard to a higher chloride resistance molybdenum is added in the alloy. The manufacture of high strength stainless steel wires for ropes takes into consideration the type of product "rolled wire quenched". Following that and in order to obtain the desired strength of at least $R_m = 1450 \text{ N/ mm}^2$ depending on the strengthening effect the steels are cold drawn by approx. 50 to 70 %. Cold working leads to a considerable rise of the tensile strength and most of all of the 0.2 %-proof stress with rolled wire as a basis and whereby the deformation characteristics indeed decline, however, they remain sufficiently high (Fig. 2). Compared with unalloyed steel austenitic material show a relatively higher rise of the strength during the cold working due to strengthening (compare in Fig. 2 a and b the behaviour of the unalloyed steel C 70 with the high-alloy steel). This is based on:

- the distinct strengthening capabilities of the austenitic,
- the precipitation behaviour of the nitrogen in the case of nitrogen-alloy materials,
- the formation of deformation martensite (hard martensite needles in an austenitic texture) in materials which are unstable in their texture.
Fig. 2: Influence of cold deforming on mechanical properties [10]
The stability of the austenite against the formation of deformation martensite depends most of all on the contents of chromium, manganese, molybdenum and nitrogen in the steels [12,13]. Increasing contents of the elements hamper the formation of martensite. The publication [10] quotes an indicator for the austenitic stability:

\[ M_{d30} = 413 - 462 (\% C + \% N) - 9,2 \% Si - 8,1 \% Mn - 13,7 \% Cr - 9,5 \% Ni - 18,5 \% Mo \]

With a falling indicator M the stability of the austenite will rise. For the steel types shown in Fig. 2 as well as for a material 1.4436 used for open spiral strands the following series of an increasing austenitic stability will emerge (the values are based on real analyses):

<table>
<thead>
<tr>
<th>Steel Type</th>
<th>Composition</th>
<th>M Value</th>
<th>Stability</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.4310 X 12 Cr Ni 17-7</td>
<td>M = - 1 °C</td>
<td></td>
<td>1 °C</td>
</tr>
<tr>
<td>1.4401 X 5 Cr Ni Mo 17-12-2</td>
<td>M = - 11 °C</td>
<td></td>
<td>11 °C</td>
</tr>
<tr>
<td>1.4436 X 4 Cr Ni Mo 17-13-3</td>
<td>M = - 27 °C</td>
<td></td>
<td>27 °C</td>
</tr>
<tr>
<td>1.4439 X 2 Cr Ni Mo N 17-13-5</td>
<td>M = - 127 °C</td>
<td></td>
<td>127 °C</td>
</tr>
<tr>
<td>1.3974 X 2 Cr Ni Mn Mo N Nb 23-17-6-3</td>
<td>M = - 388 °C</td>
<td></td>
<td>388 °C</td>
</tr>
</tbody>
</table>

Accordingly, the steel 1.4310 has a lower, 1.4401 a sufficient, 1.4436 a considerably improved and 1.4439 as well as 1.3974 a very high austenitic stability. Under [11] permeability measurements yielded that cold worked strand wires consisting of 1.4401 show portions of deformation martensite as opposed to those consisting of 1.4439 and 1.3974.

In Fig. 2 the strength values of the steel 1.4310 rise particularly after cold working. Here, the effects of strain hardening and the formation of martensite overlie. The stable austenitic steel 1.3974 has a very strong strengthening effect; here, precipitations of the nitrogen (0.4 % N) in particular contribute to the increase in strength. The most negligible strengthening of the steel 1.4401 occurs during the deformation which hardly contains any martensite and is not nitrogen alloyed. During cold working the material 1.4436 would show a similar behaviour.

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\(^{1)}\) Temperature in degrees C at which an originally austenitic steel shows 50 % of martensite in the texture after a 30 % deformation.
Hence the formation of deformation martensite has a positive effect on the strengthening behaviour of the wires. Nevertheless nowadays no steels unstable in their texture are used in manufacturing these products since the martensite portions in the austenitic texture have an adverse effect on the corrosion behaviour (Section 2.3).

3.3 Corrosion behaviour of the ropes

In rope construction some particularities must be observed which also concern their durability and in particular their corrosion behaviour:

− Mostly open air conditions will prevail. As a consequence ropes are weathered where they are exposed to aggressive effects such as acid and/or chloride salt containing aqueous media.

− The high strength cold worked steel wire in the rope is particularly susceptible to mechanical and chemical effects and therefore an durable corrosion protection (or, as in this case) a selection of sufficiently corrosion-resisting stainless steels must be taken into consideration.

− The wires in the rope are constantly tensioned with up to 42 % of their tensile strength and, in addition to that, they are transversely pressed in the anchorage areas and the saddle.

− For ropes dynamic loads can occur as a consequence of oscillations under load or from outside forces, such as wind. This can lead to remarkable amplitudes and stress cycles in the event of a longitudinal tensile force. In the concurrence of environmental influences and corrosion wire fractures can occur due to fretting and corrosion fatigue cracking. In normally light and wide span supporting structures rhythmic vortex shedding on ropes in laminar wind can lead to lateral vibrations which results in bending load strains and additional fretting at the anchorages. Rhythmic stresses caused by gusts of wind or from a series of heavy vehicles on bridges can entail lateral rope vibration. For instance in cable-stayed bridges the traffic and vortex shedding due to aeolian vibration can cause amplitudes up to appr. 100 N/mm² [14].
The constructive design of the tension member, their fittings and other details (e.g. crevices) have an influence on corrosion and demand particular care.

The existing knowledge on corrosion of austenitic stainless steels, paying particular attention to the aspects already mentioned, is compiled in [4,15]. The particular behaviour of ropes made from cold-worked austenitic steel wires is dealt with in [4,11]. Some partial aspects as well as the latest findings are summarized in the following parts. It is assumed that high strength ropes made of stainless steel are most likely to be subjected to the danger of pitting and crevice corrosion, stress corrosion cracking and crack corrosion under dynamic load.

### 3.3.1 Pitting and crevice corrosion

Electrolytic, material and design influences are responsible for pitting and crevice corrosion to happen.

The initiation of pitting corrosion requires the presence of chloride ions. The susceptibility to corrosion decreases with falling chloride content, falling temperature and rising pH-value. Chloride enriched acid media are particularly critical. In pitting corrosion inducing aggressive agents not sufficiently high alloyed stainless steels can be particularly endangered by crevice corrosion.

On the material side the alloying elements chromium, but most of all molybdenum, but also nitrogen have a favourable effect on the pitting corrosion behaviour. Hence the corrosion resistance of the material 1.4436 in pitting corrosion inducing media is higher than that of 1.4401.

The corrosion resistance in pitting corrosion inducing media also depends on the surface quality and can thus be impaired by a surface treatment and also by cold working. In principle, the resistance against pitting corrosion of stainless steels is the better, the smoother and more homogenous the surfaces are. From a corrosion point of view the quality of the surface will increase somewhat in the order of oxidized - rough grinding - blasted - finish grinding - pickled - polished [16]. The corrosion resistance also increases in this sequence. Cold drawn round wires for ropes, comparable with finish grinding materials, show relatively smooth surfaces which has a positive effect with regard to hampering local attack.

The results concerning the effects of a stronger working on the pitting and crevice corrosion behaviour are not uniform as can be seen in the literature on
the subject [15]. In some tests rather an increase of the susceptibility at rising cold working was found. Other tests, however, showed no influence of the cold working up to forming degrees of appr. 60%.

The deformation martensite which is formed during deformation process of structural unstable stainless steels can most likely have an unfavourable influence on corrosion behaviour but will, however, lead to a higher susceptibility only under pitting corrosion conditions. In chloride enriched aqueous media a preferred selective corrosion of the martensite takes place which intensifies the pitting corrosion [17] and as a consequence can also lead to cracking. In an inert damp environment, oxygen free and oxygen containing water, in an SO2-containing atmosphere and even at (low) chloride contents of 50 mg/l of an aqueous phase the resistance of austenitic types will not be impaired by deformation martensite [18]. Therefore it is recommended that as is the case with the materials 1.4401 and 1.4436, to form such steels to become high strength wires which due to their alloying content show a sufficiently high structural stability in the cold deformation process.

Under [11] a series of specific tests was carried out on pitting and crevice corrosion of steel strand (the simplest type of open spiral strands) made from high strength austenitic steel wires, i.e. the materials 1.4401, 1.4439 and 1.3974:

A storage of specimens took place in artificial climates with particular emphasis on extreme seawater and deicing salt conditions and conditions in an industrial atmosphere. Strand wires were stored up to 3 years under 100% relative humidity and a temperature of 45°C. By means of an atomizer a 0.5% NaCl solution and/or SO2 water of pH 3 were sprayed daily on the strand wire. Within 3 years all materials sustained without any visible signs of rust and also in the crevices between the wires there was no corrosion attack.

Furthermore steel strand sections with attached bodies (crevices) were exposed to a 3 year outdoor weathering in the maritime splash zone and in sea atmosphere on the isle of Heligoland and in an industrial climate on the roof of a metallurgical plant in Duisburg. Apart from the crevices the same favourable corrosion behaviour was found as previously described in the artificial climates. In the area of crevices only wires made from the steel 1.4401 and only after exposure in the splash zone showed a weak pitting corrosion (pitting depths < 50 µm).
Crevice corrosion lab tests in a NaCl solution and SO\(_2\) water of pH 3 were carried out at ambient over a period of 4 years. In the test steel strand sections were immersed vertically to about one half in a cylindrical sampler containing a 3.5 \% NaCl solution or SO\(_2\) water of pH 3 respectively. At the top the sampler was sealed with a chlorinated rubber plug pushed over the steel strand. During the test, due to capillarity, the test solution rose within the strand and evaporated above the top side plug. This way the steel strand in the case of the saline was always surrounded by a highly over saturated saline in the crevice of the plug. In the test in SO\(_2\) water there was no proof of corrosion effects in all 3 strand materials. After the test in a NaCl solution, however, the strand wires showed preferably local corrosion attack only in the upper contact area with the top-side plug. These attacks as a result of crevice corrosion under a prevailing extreme corrosion load due to upgraded chloride salines are only distinct on the material 1.4401 (pitting depth of up to 100\(\mu\)m). They were negligible low (< 10 \(\mu\)m) in the case of 1.4439 and non-existing on the material 1.3974.

### 3.3.2 Anodic stress-corrosion cracking

In structures under tensile load transcrystalline stress-corrosion cracking must be taken into consideration mainly for relevant sensitive chrome-nickel steels in chloride containing media. Rising chloride contents and temperatures as well as falling pH-values of the attacking electrolytes reinforce the attack. The nickel content has a considerable influence on the stress-corrosion cracking of stainless steels. The austenitic steels 1.4401 and 1.4436 with nickel contents of 12 or 13 \% respectively used for ropes are considered to be sensitive to stress-corrosion cracking. Additions of molybdenum to the steel have a positive effect since frequently the stress-corrosion cracking acts at the initial state of pitting corrosion (pitting induced stress-corrosion cracking) whereas increasing additions of molybdenum hamper pitting corrosion.

In [15] a critical examination of the bibliography on the influence of cold working on the stress-corrosion cracking behaviour of stainless steels was carried out. Altogether, weaker degrees of deformation do not seem to exert a relevant influence on the construction. However, as in higher deformations, residual tensile stresses can occur in areas close to the surface which will suffice to initiate stress-corrosion cracking in the event of critical material or electrolyte parameters.
In accordance with tests [19] the stress-corrosion cracking behaviour of austenitic steels will be improved after cold working whereby rather a positive effect is adjudged to the deformation martensite being generated at higher degrees of deformation. But then a stable austenitic steel showed rather a decreasing resistance at a higher degree of deformation [20]. Other tests [21], however proved that a selective corrosion at martensite accumulations also may lead to cracking. Several damages that occurred in an indoor swimming pool atmosphere on stainless steel as a result of stress-corrosion cracking portions of martensite were found in the texture [22].

A stronger cold working as is required for the manufacture of high strength wires may altogether improve the stress-corrosion cracking behaviour of stainless steels by raising the threshold stress for the stress-corrosion cracking resistance. If, however, an assessment is made at a tension which shows a constant relation to the yield strength an unfavourable impression may possible ensue concerning the influence of cold working [23, 24].

As a result of these connexions austenitic steels are considered to be susceptible to chloride induced stress-corrosion cracking whereby the environmental influences must be given particular attention:

– So far no grave damages on austenitic steels due to stress-corrosion cracking under atmospheric corrosion load have been made known. Also the data collected at the hanging spiral strands of older projects (see below) did not show any relevant indications.

– Tests [11] with concentrated chloride solutions on cold worked wire of steel strand made of the material 1.4401 pitting induced stress-corrosion cracking under crevice corrosion conditions was found (Fig. 3) which was not on the wires made from 1.4439 and 1.3974. High concentrations of chloride may also occur in the building trade if a local upgrading happens as a result of evaporation processes. Under atmospheric conditions extremely high chloride concentrations due to the deposits of hygroscopic salts can be obtained. Dry salts vapour from the air which is liberated as liquid water near the saturation humidity which is characteristic of any salt [4]. During a weathering exposure (rain test) in open air the above-mentioned salt upgradings cannot happen.
Indoor swimming pools offer a particular hazard where apart from the aforementioned salt upgradings also acidizing of aqueous fluid films can occur e. g. as a result of water treatment using the chlorine gas method. The collected data under [25] also noted damages due to stress-corrosion cracking on high strength austenitic steels.

The tests carried out and the data collected on crevice and stress-corrosion cracking thus were introduced into the new certification of ropes made from high strength stainless steel wires to the effect that ropes made of the material 1.4401 must be used only in structures without noticeable contents of chlorides and sulfur dioxide. The material 1.4436 is mainly reserved for ropes in environments with moderate chloride and sulfur dioxide contaminations.
3.3.3 Crack corrosion under dynamic load

Firstly, the dynamic behaviour of a rope is influenced by the properties of the wire (material, surface). Defects in the surface of a high strength wire severely degrade the fatigue strength. Fretting corrosion has a negative influence on the fatigue strength of the wires within the strand/rope. Fretting movements under pressure diminish the fatigue strength behaviour also at diversions, anchorages and cable clips.

In a natural environment fractures in the wires can occur under the effects of load changes and fatigue loads of the tension members as a result of
- fatigue (in dry air),
- fretting corrosion (in dry air and when attacked by aqueous media),
- corrosion fatigue (when attacked by aqueous media).

Fretting corrosion. Due to multiple influences the constantly high fatigue strength of the stainless steel individual wire [9] cannot be transferred to the final product "anchored strand/rope" [11]. Strands/ropes always have a much lower fatigue strength than the individual wires. This is attributable to additional loads on the wires after stranding and also to fretting movements among the wires under simultaneous transversal pressure and fatigue load. The surrounding air causes the fretting areas to oxidize and due to fretting corrosion and under fatigue load additional tensile loads will come up which constantly change their direction [26]. Furthermore at the fretting areas the metal texture will be locally disrupted by the interaction of mechanical and corrosive operations leading to initial cracking which, as sharp-edged notches, promote the generation of fatigue failures. There are a number of constructional influences such as anchorages and cable clips which permit to further reduce the bearable range of stress of tension members due to fretting corrosion. Since the force is gradually led into these components vibrations cause relative displacements with pressures between the strand/rope and the anchorage for instance (fitting on the open spiral strand). For this reason the inlet locations of the wires are in particular danger with regard to fretting corrosion and the initiation of fatigue failures.
Corrosion fatigue. The use of ropes in open air always requires an evaluation of corrosion fatigue under load [4]. The simultaneous interaction of a mechanical fatigue stress and an electrochemical corrosion stress reduces the number of cycles to failure found in air, i.e. without any influence of corrosion. An initiation of corrosion fatigue neither requires the specific sensitivity of an alloy nor the specific properties of an electrolyte, and yet specifically effective ions in the aqueous attack medium such as Cl\(^-\) and/or H\(^+\) will intensify the attack. The intensity of corrosion fatigue increases with a falling frequency of the vibration.

For the material it is true to say that with rising strength surface notches have an increasing negative effect. Alloying elements in the steel which intensify the passivity increase the corrosion fatigue strength under otherwise identical conditions. Therefore, as a rule, the fatigue strength in a corrosive medium is the higher the more corrosion resistant the material proves to be.

For an assessment of the corrosion fatigue behaviour of steel strand (single open spiral strands) made of high strength stainless steel wires under unfavourable atmospheric conditions in [11] and continuous evaluations under pulsating loads tests were carried out in chloride containing and/or acid atmospheric water (Fig. 4). As expected, the reaction was clearly better than that of steel strand in a comparative test made of high strength unalloyed steel wires. However, the strand did not react as favourably as can be expected of single wires. This has to do with the fact that under a dynamic load of the steel strand the corrosion fatigue is superimposed by a fretting fatigue due to fretting corrosion. With regard to fretting corrosion stainless steels react rather more sensitively than unalloyed steels so that the extremely positive behaviour of the single wire can only be carried over to the strand to a lesser extent. In high alloyed steels the initiation of active over regions on an otherwise passive surface is stimulated by fretting under transversal pressure.

The overall test results proved that even under most unfavourable environmental conditions and at a non-restive load the tolerable range of stress of a strand wire (i.e. an open spiral strand) made of high alloy steel can be set at a minimum of 100 N/mm\(^2\).
Fig. 4: Results of fatigue tests on high strength strands in air and corrosive media (frequency 0.5 and 5.0 s⁻¹) [11]

left: material 1.4401

right: material 1.4439
3.4. Pendant rope tests on older rope projects

Four up to 23 years old suspension bridges with suspended reinforced concrete footbridges were investigated in the Stuttgart urban area (Fig. 5 above). The main carrying ropes are fully locked-spiral ropes. The corrosion protection consists of a hot-dip galvanizing of the wires and a duplex coating of the rope. The attached pendant ropes for fixing the reinforced concrete slabs are spiral strands made from the materials 1.4401 or 1.4436. The anchorages and other fittings are cast steels made from the material 1.4462. Three of the footbridges span highly frequented federal highways undergoing deicing salt treatment in wintertime. One bridge crosses the Neckar river. The investigated strand segments and fittings are installed just as a few metres above the road surface and thus in the entrance region of deicing salt spray.

In all cases, apart from so-called "bleedings" on open pores of the cast fittings, no essential corrosion effects were found. The strands look virtually new (Fig. 5 below). This even applies to strands made from 1.4401 the 23 years-old "Rosensteigsteg" which spans the highly frequented federal highway B 14. Also in the entrance region of strands in fittings no signs of crevice corrosion were found.

4. EVALUATION

The tests conducted justify the use of strength class S 1100 spiral strands made from the material 1.4436 under moderate chloride and sulfur dioxide load. The findings gained from tension members made of high strength stainless steel wires seem to indicate that strength class S 1100 strands made from the material 1.4401 should only be used in constructions of negligible chloride and sulfur dioxide contents. This assessment takes into account the behaviour compared with the most important types of corrosion such as pitting and crevice corrosion as well as stress corrosion cracking under the known marginal conditions for tension members in accordance with DIN 18800 part 1. The more conservative rating of the material 1.4401 compared to the material 1.4436 is due to the fact that the material 1.4401 in the strength class S 1100 in the presence of chloride salts and their upgrading in cracks is not sufficiently safe against crevice corrosion and stress corrosion cracking.
Fig. 5: Open spiral strands of a suspension bridge with suspended reinforced concrete footbridge

The load on the strands is preferably static. In rare cases of a non-static stress amplitudes clearly under 100 N/mm² must be expected. Ropes made of the materials 1.4401 and 1.4436 will reach this stress value even under a high corrosion load due to chlorides and sulfur dioxide.
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FIBER REINFORCED DRAINAGE CONCRETE


FASERVERSTÄRKTER DRÄNBETON

BETON DE DRAINAGE ARME DE FIBRES

Michael Stegmaier

SUMMARY

Drainage concrete used as finishing coat on roads offers due to its open porous structure a large potential for reducing the noise of road traffic. As a consequence this open-porous structure leads to a very brittle material behaviour. To increase the ductility of this concrete polypropylene fibers were added and the influence of the fibers on the mechanical properties was investigated. Additionally tests on the freeze–thaw behaviour of fiber reinforced drainage concrete were accomplished. With the addition of the fibers a noticeable improvement of the ductility was achieved compared to drainage concrete without fibers. Furthermore fiber reinforced drainage concrete without other admixtures or additives shows a satisfying freeze–thaw resistance. This freeze–thaw resistance however can be improved by the use of polimeric dispersions.

ZUSAMMENFASSUNG

Lerdings durch Verwendung einer Polymerdispersion weiter verbessert werden kann.

**RESUME**

Le béton de drainage offre, grâce à sa porosité ouverte, un grand potentiel pour son utilisation comme couche de roulement. Cette porosité mène à un comportement très fragile. Pour augmenter la ductilité du matériau, des fibres de polypropylène ont été ajoutées et leur influence sur le comportement mécanique a été étudié. Le comportement au gel et dégel a également été testé. Une nette amélioration de la ductilité a été obtenue par rapport au béton de drainage sans fibres. Nous avons également constaté que le béton de drainage armé de fibres sans addition d'adjuvants ou d'additifs présente une résistance au gel et dégel satisfaisante. Celle-ci peut être encore améliorée par une addition d'une dispersion de polymères.

**KEYWORDS:** drainage concrete, polypropylene fiber, fracture energy, freeze thaw resistance

1. **INTRODUCTION**

The reduction of noise of road traffic is nowadays with a steady increasing number of vehicles of large interest. The road traffic noise is composed of the noise of the engine and the noise of rolling. Investigations have shown that noise of rolling is dominating at velocities higher than 70 km/h [8]. Here now the possibility exists to reduce the noise with a suitable road surface layer. With the concrete road building method different surface textures were already examined regarding the noise reduction. The treatment of the surface turned out suitably by means of broom line and/or jute cloth [8]. A clearly higher noise reduction is however possible by the employment of a porous material for the surface layer. Concrete can be manufactured purposefully in such a way that it has a freely accessible pore system which is able to absorb sound. Measurements of a test track from this open - porous concrete (drainage concrete) on a federal motorway showed that with passenger car at a speed of 120 km/h the sound energy can be reduced by 63 % and for a truck at 80 km/h by 80 % [3, 7]. Due to this open - porous structure this concrete shows however an extremely brittle material behaviour. Moreover doubts exist that due to the abrasion of the tyres single grains could loosen and be hurled around by the vehicles. In order to reduce
these dangers and to increase the ductility of drainage concrete polypropylene fibers were added and the fiber content was varied. In a further step investigations on the freeze and deicing salt resistance have been accomplished. Drainage concrete containing fibers shows a sufficient freeze – thaw resistance.

2. STATE OF THE ART

Drainage concrete is a concrete with a raw ore structure that contains only a single fraction of aggregates. As bonding agent portland cement CEM I 32,5 R is used, whereby the amount of cement paste is designed in that way that the aggregate grains are totally coated, however the gaps between the aggregates remain empty. The DIN 1045 can not be applied for this type of concrete. Only the strength classification is adopted [4, 5, 11]. In well known investigations [1, 2, 3, 6, 7] always similar mixtures were used. Usually 300 - 350 kg/m³ cement CEM I 32,5 R and about 1500 kg/m³ moraine crushed stone with a particle size distribution between 5 - 8 mm was used. The water - cement ratios varied between 0,24 and 0,30 and with the mix composition a void content of 15 – 25 volume -% was planned. This void content is necessary for an appropriate sound absorbion of this concrete. Moreover a polymeric dispersion was added to improve the freeze - thaw resistance of the drainage concrete. To get a sufficient freeze – thaw resistance additionally silica fume is necessary according to [1, 2].

3. RANGE OF INVESTIGATIONS

The determination of the compressive strength and the splitting tensile strength was after DIN 1045 T. 5. The compressive strength of all manufactured mixtures was determined by means of 3 cubes with a length of the edges of 150 mm at an age of 28 days. The bending tensile strenght of the concretes was determined according to the ZTV Beton StB (2001) [12] with beams with the dimensions 700 mm x 150 mm x 100 mm (l x w x h). The standard plans for this measurement that this is accomplished strength - steered. Since at the same time in addition, the load - deflection line should be measured for the evaluation of the ductility, the tests had to be performed deformation controlled. Therefore the results of the bending tensile measurements can’t be compared directly with results from other investigations. The displacement rate at these tests was 1 mm/min. The deflection of the specimens was determined by the displacement of the plunger of the testing machine. As the evaluation and comparison of the
ductility of the individual mixtures among themselves exclusive on the basis of the load - deflection lines is difficult, additionally the fracture energy using the following equation (1) for the individual mixtures was determined:

\[ G_f = \frac{1}{A} \int_0^f P(f) df \]  

with:

- \( G_f \) = fracture energy [N/mm]
- \( A \) = cross sectional area [mm²]
- \( P \) = load [N]
- \( f \) = deflection [mm]

The splitting tensile strength was determined with the oddments of the bending tests. 4 measurements were performed for each mixture.

The freeze – thaw resistance of the concretes was tested according to the CDF – test. The dimensions of the specimens were 150 mm x 150 mm x 75 mm. Deviating from the CDF – test demountable plastic formwork with dimensions of 200 mm x 200 mm x 200 mm have been used. They were not treated with a separating agent. The cubes manufactured with this formwork were subsequently cut to the necessary specimen size with a stone saw. As inspection surface the sides resting against the formwork were used. The further procedure corresponded to the recommendations of the CDF - procedure [13].

4. CONCRETE MIX PROPORTIONS

The cement used for these investigations was a portland cement CEM I 32,5 R. As aggregates moraine crushed stone with a grain distribution from 5 – 8 mm was employed. For the investigations on the mechanical properties a polymeric dispersion (furthermore called dispersion 1) was added. The dispersion has a density of 1,04 kg/dm³ and contains about 50 % water and 50 % solids. The effective components are a copolimerisat on styrene basis and acrylic acid ester. For the investigations on the freeze – thaw resistance another dispersion was used. It was an acrylate resin dispersion with cement - reactive mineral substances and active substances for the modification of cement - bound mortars and bonding layers (called in further dispersion 2). The density of this dispersion amounts to 1,25 kg/dm³. The fraction of solids is about 56 %. All mix-
tures were designed with a total voids content of 20 volume-%. The detailed mix proportions are shown in table 1.

Table 1: Composition of the concretes for the investigations on the mechanical properties.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Fiber content [Vol.-%]</th>
<th>Cement [kg/m³]</th>
<th>w/c [-]</th>
<th>Aggregates [kg/m³]</th>
<th>Dispersion content [% of cement]</th>
</tr>
</thead>
<tbody>
<tr>
<td>R3</td>
<td>0,0</td>
<td>300</td>
<td>0,29</td>
<td>1554</td>
<td>20</td>
</tr>
<tr>
<td>F23</td>
<td>1,0</td>
<td>300</td>
<td>0,29</td>
<td>1527</td>
<td>20</td>
</tr>
<tr>
<td>F24</td>
<td>2,5</td>
<td>300</td>
<td>0,29</td>
<td>1488</td>
<td>20</td>
</tr>
<tr>
<td>OZ1</td>
<td>1,0</td>
<td>400</td>
<td>0,29</td>
<td>1341</td>
<td>20</td>
</tr>
<tr>
<td>OZ2</td>
<td>2,5</td>
<td>400</td>
<td>0,29</td>
<td>1302</td>
<td>20</td>
</tr>
<tr>
<td>OZ3</td>
<td>1,0</td>
<td>350</td>
<td>0,29</td>
<td>1434</td>
<td>20</td>
</tr>
<tr>
<td>OZ4</td>
<td>2,5</td>
<td>350</td>
<td>0,29</td>
<td>1395</td>
<td>20</td>
</tr>
<tr>
<td>OF1</td>
<td>1,5</td>
<td>400</td>
<td>0,29</td>
<td>1328</td>
<td>20</td>
</tr>
<tr>
<td>OF2</td>
<td>2,0</td>
<td>400</td>
<td>0,29</td>
<td>1315</td>
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<td>400</td>
<td>0,29</td>
<td>1354</td>
<td>20</td>
</tr>
</tbody>
</table>

For the investigations on the freeze – thaw resistance different additives and admixtures were added to a basic mixture containing 400 kg/m³ cement CEM I 32,5 R, a water - cement ratio of 0,29 and a fiber content of 1,5 volume-%. The purpose was to test the influence of these additives and admixtures on the freeze – thaw resistance of the drainage concrete. The designed void content of these mixtures was also 20 volume-%.

The mix proportions for the CDF – tests are shown in table 2.

Table 2: Composition of the concretes for the investigations on the freeze – thaw resistance.

<table>
<thead>
<tr>
<th></th>
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<th></th>
<th></th>
<th></th>
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<tbody>
<tr>
<td>CF1</td>
<td>1328</td>
<td>1</td>
<td>20</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>CF2</td>
<td>1426</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>CF3</td>
<td>1362</td>
<td>2</td>
<td>20</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>CF4</td>
<td>1425</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>Sperrpulver (8g/kg cement)</td>
</tr>
<tr>
<td>CF5</td>
<td>1425</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>Innenversiegler (15ml/kg cement)</td>
</tr>
<tr>
<td>CF6</td>
<td>1425</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>Luftporenbildner (1,5 ml/kg cement)</td>
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<td>0</td>
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</tr>
<tr>
<td>CF8</td>
<td>1403</td>
<td>-</td>
<td>10</td>
<td>0</td>
<td>0</td>
<td>Microhohlkugeln (1kg/m³)</td>
</tr>
<tr>
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<td>1311</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>25</td>
<td></td>
</tr>
<tr>
<td>CF10</td>
<td>1288</td>
<td>-</td>
<td>0</td>
<td>10</td>
<td>25</td>
<td></td>
</tr>
</tbody>
</table>

Fiber reinforced drainage concrete
5. TEST RESULTS AND DISCUSSION

5.1 Influence of cement content on the mechanical properties.

The results of the compressive strength against the cement content of the mixtures is shown in figure 1.

![Graph showing compressive strength against cement content.](image)

*Fig. 1: Compressive strength against the cement content.*

It can be seen that both with a fiber content of 1 volume-%, and with 2.5 volume-% the compressive strength rises with increasing cement content. The increase of the strength can be explained by two different causes. On the one hand the voids content of the mixtures becomes smaller with rising cement content and on the other hand the aggregate content decreases with increasing cement content. This leads to a smaller specific surface with rising cement paste content and thus to a better connection of the aggregates and the fibers.

Also with the splitting tensile strength and the bending tensile strength an increase can be recognised with increasing cement content (figure 2). The reasons for the improved strength are the same as mentioned for the compressive strength.
Fig. 2: Bending tensile strength and splitting tensile strength against the cement content.

With both fibre contents an increase of the fracture energy is connected with the increasing amount of cement (see figure 3). With a fiber content of 2.5 volume-% the fracture energy is approximately doubled by the increase of the cement content, while with 1 volume-% fiber content the increase amounts approx. to 50 %. Due to the increasing cement paste content the fibers are embodied better in the matrix and can therefore participate for a longer time period in bearing the load.

Fig. 3: Fracture energy against the cement content.
5.2 INFLUENCE OF FIBER CONTENT ON THE MECHANICAL PROPERTIES

With increasing fiber content a reduction of the compressive strength can be recognised (see figure 4). The compressive strength of 20 N/mm² of the mixture with a fiber content of 2,5 volume-% is about 30 % less than the compressive strength of the concrete with 0,5 volume-% fiber content. The increase of the fiber content within the mix design leads to a reduced fraction of aggregates. The specific surface of the fibers is however much bigger than the specific surface of the aggregates so that the replacement of aggregates with fibers induces a great enlargement of surface that has to be connected by the cement paste.

The specific surface of 0,5 volume-% fibers is about 40 times bigger than the specific surface of the corresponding aggregates. Since the amount of cement paste was held constant a weakening of the matrix occurred and therefore the compressive strength decreases.

As can be seen in figure 5 the splitting tensile strength as well as the bending tensile strength also decreases with the fiber content.

The reason for the reduction of strength is a weakened matrix due to the bigger specific surface at a constant cement paste content.
Fig. 5: Bending tensile strength and splitting tensile strength against the fiber content.

The increasing fiber content produces a noticeable gain in fracture energy. The extension of the fiber content from 0,5 volume-% to 2,5 volume-% leads to a 3 times bigger fracture energy (see figure 6). The difference in fracture energy between a fiber content of 2,0 volume-% and 2,5 volume-% is however small.

Fig. 6: Fracture energy against the fiber content.

The increase of the fracture energy with fiber content is possibly a consequence of a better distribution of the load in the fracture cross section due to the growing number of fibers. The fibers are obviously pulled out of the matrix slower in spite of the weakened cement paste matrix.
6. RESULTS OF THE CDF – TESTS

Figure 7 shows the results of the CDF – tests after 28 freeze – thaw cycles. There are great differences in the behaviour of the different concretes. The best freeze – thaw resistance have the mixtures with both 20 volume-% polymeric dispersion whereby the mix CF3 with dispersion 2 is the best one. Remarkable is the result of the mixture CF2. This concrete contained no admixtures and/or additives and however showed a good freeze – thaw behaviour.

![Fig. 7: Results of the CDF – tests.](image)

This proves that the addition of a polymeric dispersion is not absolutely necessary to get a sufficient freeze - thaw resistance. The notably good behaviour of the mixtures with the polymeric dispersion is possibly a consequence of a denser transition zone between the aggregates and the matrix. For this purpose the dispersion 2 seems to be more suitable due to the reactive ingredients.

7. CONCLUSIONS

The results of the investigations show that the ductility of drainage concrete can be improved noticeably by the addition of fibers. But with increasing fiber content the compressive strength and the tensile strength decreases. This can be counteracted by increasing the amount of cement. In the end one has to find a compromise between the ductility and the compressive and tensile strength. A reasonable fiber content seems to be 1,5 volume-% with an amount of cement of 400 kg/m³.
CDF – tests concerning the freeze – thaw behaviour were conducted with concretes composed like this. Different additives and admixtures were added to investigate their influence on the freeze – thaw resistance. The mixtures with a polymeric dispersion produced the best results. However the fiber reinforced drainage concrete without any admixtures or additives and an average loss of weight of 503 g/m² after 28 freeze – thaw cycles has a satisfying freeze – thaw resistance.

With a further addition of the cement content the compressive and tensile strength can possibly be increased at a fiber content of 1,5 volume -%. Simultaneously the water – cement ratio should be further reduced.

REFERENCES


SUMMARY

This article is the continuation of the preceding Otto-Graf-journals essay over DuCOM [1]. The program DuCOM discussed there was extended by the possibility of modelling high-strength concrete containing normal and lightweight aggregates. The obtained results are compared with experimentally determined data.

ZUSAMMENFASSUNG


RESUME

Cet article est la continuation de l'essai du DuCOM qui a été publié dans le Otto-Graf-journaux précédent [1]. Le programme DuCOM, discuté dans ce journaux, a été ajouté à la possibilité de modeler un béton de haute résistance contenant les agrégats normaux et légers. Les résultats obtenus sont comparés aux informations expérimentalement déterminées.

KEYWORDS: DuCOM, simulation of concrete hydration, high strength concrete
1. INTRODUCTION

High performance concretes are concretes fulfilling highest claims in regard to their properties. The best known high performance concrete is the high strength concrete. A crucial characteristic of high strength concrete is its low water-paste ratio. The workability of concrete with low water-paste ratio became possible due to the invention of concrete fluidisers and fluxing agents. It was proven theoretically that an ordinary Portland cement concrete with a water-paste ratio of 0.36 that was stored under water can completely hydrate and no capillary pores will develop. The water-paste ratio of a high strength concrete amounts up to 0.30. This is less than the value that is necessary to provide complete hydration. Hence the danger of self-desiccation exists with developing hydration if no additional water will be provided. Furthermore high strength concrete develops within the first days such a dense structure that no water from the outside can infiltrate into the structure. Because of this effect all common after treatment methods of concrete are ineffective and the self-desiccation occurs resulting in early age shrinkage and cracks inside the structure do emerge. The strength development stagnates due to the crack initiation. An inside water reserve that is provided by water saturated light weight aggregates can help to avoid these effects. During a testing program of the IWB concretes were produced with normal and light weight aggregates [3;4]. It was found that water saturated light weight aggregates were functioning as additional water reservoir inside the concrete and that they were fulfilling the function as an after treatment for concrete. Without any outside after treatment the concretes with 25% light weight aggregates content reached a 20% higher strength after one year compared to concretes with solely normal weight aggregates. In comparison to normal weight concrete there weren’t any visible cracks found at specimens with light weight aggregates and the slump of the 180 days strength of normal weight concrete failed to appear. These attributes of high strength concrete should be simulated and attributes relevant for construction sites like compressive strength should be predicted by the computer model. The used model is based on the DuCOM [2].
2. PRINCIPALS

The Liapor F8 grains have a grading curve where a 88% fraction remains between the sieve sizes of 4 and 8 millimetres. Hence the average diameter was assumed with six millimetres. This assumption results in a globe volume of $113.09 \text{ mm}^3$. This volume was converted to an equivalent sized cuboid having an edge length of about 4.8 mm. It was necessary to use cuboids because DuCOM is only able to calculate with eight nodes elements. In dependence of the fraction of Liapor of the concrete mixture the size of the surrounding concrete matrix was calculated. The names of the simulated cubes and mixtures refer to the proportion of light weight aggregates in the mixture. The used mixtures are given in Table 1.

<table>
<thead>
<tr>
<th></th>
<th>HB 0</th>
<th>HB 15</th>
<th>HB 25</th>
<th>HB 30</th>
</tr>
</thead>
<tbody>
<tr>
<td>light weight aggregate</td>
<td>[vol.-%]</td>
<td>0</td>
<td>15</td>
<td>25</td>
</tr>
<tr>
<td>w/c-value</td>
<td></td>
<td>0,33</td>
<td>0,33</td>
<td>0,33</td>
</tr>
<tr>
<td>cement</td>
<td>[kg/m$^3$]</td>
<td>450</td>
<td>450</td>
<td>450</td>
</tr>
<tr>
<td>fine aggregates</td>
<td>[kg/m$^3$]</td>
<td>815</td>
<td>737</td>
<td>691</td>
</tr>
<tr>
<td>fraction ≤ 4/8</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>coarse aggregates</td>
<td>[kg/m$^3$]</td>
<td>934</td>
<td>800</td>
<td>524</td>
</tr>
<tr>
<td>fraction ≥ 4/8 without light weight aggregates</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>light weight aggregates 4/8</td>
<td>[kg/m$^3$]</td>
<td>0</td>
<td>212</td>
<td>534</td>
</tr>
<tr>
<td>17% water content of the light weight aggregates</td>
<td>[kg/m$^3$]</td>
<td>0</td>
<td>36,04</td>
<td>90,78</td>
</tr>
</tbody>
</table>

During all simulations the used boundary conditions were for the first three days constant. The temperature was 20 degrees Celsius and humidity 100%. After this time the humidity was reduced to 65%. The temperature remained constant. The Liapor grain was modelled as a concrete mixture that has a very high water content, a very low cement content and no other supplements. Thereby a sufficient water content and a relatively low strength were achieved.
This approach was chosen because DuCOM can’t simulate any other materials except of concrete.

3. IMPLEMENTATION OF LIGHTWEIGHT AGGREGATES

The theoretical description of light weight aggregates assumes an ideal dispersion of light weight aggregates within the concrete mixture [2]. If the aggregate content is below a certain fraction there won’t be a continuous network with a porosity that is high enough to build up continuous pores. Because of this the problems of moisture exchange can be reduced to the interactions of grain and surrounding matrix. Light weight aggregates can be simply described as a water reservoir with certain attributes in regard to water content and strength. Therefore it can be described as:

\[
\frac{q^a_{fl}}{t} = R_{cap.-agg.} + R_{matrix.-agg.}
\]

\( q^a_{fl} \): water content of the light weight aggregates

\( R_{cap.-agg.} \): speed of water delivery to the capillary pores

\( R_{matrix.-agg.} \): speed of water delivery to the concrete matrix

The water content of aggregates can be calculated by considering the saturation of the pores with any fluid. The water vapour content can be neglected.

\[
q^a_{fl} = r_{fl} f_{Agg.} S_{fl}
\]

\( r_{fl} \): density of a fluid; \( f_{Agg.} \): porosity of aggregates;

\( S_{fl} \): saturation of aggregates

The results of the performed simulations have been achieved by a different approach. The light weight aggregates have not been part of the mathematical description of the concrete matrix. They were modelled as an element with the attributes of water saturated aggregates. Two types of models were created for the simulations. One with a dense mesh for simulations of the dispersal of moisture within the concrete (figure 3) and another one for the calculation of compressive strength and degree of hydration. Therefore it was necessary to use a coarse mesh (figure 2) otherwise computing time would have been very high. The size of the light weight aggregate has been constant while the thickness of the concrete matrix differed in dependence of the proportion of light weight aggregate (figure 1). For each model three simulations were performed. One to
simulate an adiabatical system; another one with a system where one side of the cube had an exchange of temperature and humidity and the last one where two sides had an exchange with the environment. These boundary conditions were chosen to simulate the inside, a cube placed on the surface of the column and one in the corner having two surfaces.

light weight aggregate:  

concrete matrix:

with all mixtures:  

dimensions:

\[
\begin{align*}
\text{HB 0} & : a = 9.09 \text{ mm} \quad V = 751.2 \text{ mm}^3 \\
\text{HB 15} & : a = 9.09 \text{ mm} \quad V = 751.2 \text{ mm}^3 \\
\text{HB 25} & : a = 7.67 \text{ mm} \quad V = 450.7 \text{ mm}^3 \\
\text{HB 30} & : a = 7.22 \text{ mm} \quad V = 375.6 \text{ mm}^3
\end{align*}
\]

**Figure 1: Geometry of the simulated cubes**

**Figure 2: Model with eight Liapor grains**

**Figure 3: Model with one Liapor grain**
4. RESULTS OF THE SIMULATIONS

Degree of hydration

The in figure 4 shown curve-progressions are the results of the simulations of the hydration degree resulting after 365 days of curing. The adiabatical curves are shown as an example for the simulations.

Because the heat loss is circumvented within an adiabatical system the heat increases rapidly. After one day the water loss reduces the reaction speed significantly and the curve growth of the mixture without an additional water reservoir (HB 0) decreases. For the mixtures with water saturated light weight aggregates this effect is postponed. This results in a higher degree of hydration. It can be shown that the hydration continues approximately up to the age of 80 days. Between the 2. and the 80. day the remaining capillary water diffused into the grain through the CSH needles and reacts with the cement. The difference between mixtures with and without light weight aggregates is growing until the 9. day. After this time the gap between HB 0 and the other curves decreases again. This can be explained by the resulting slower reaction because of the bigger grain boundary surface of the higher hydrated mixtures. The mixture HB 25 reaches the highest level of hydration of all mixtures.

To show the influence of the light weight aggregates and the boundary conditions on the hydration degree it was best to provide these information in a single plot. At the age of 14 days the difference between the curves are the biggest. Figure 5 shows the hydration degrees at this age.
The influence of the content of water saturated light weigh concrete is clearly visible. With increasing aggregate content the hydration degrees reached are increasing too. The linear development line shows for the simulations with an exchange with the environment a fast ascending slope. The lines for one and two open sides develop approximately parallel to each other. The line for an adiabatical system has a very flat progression. This can be explained by the non existent moisture loss to the environment. All the water of the mixture reacts with the cement and this continues until all the water has been used. One reason why the additional water content has little effect on the hydration degree might be that the water transport mechanism is controlled by Darcy’s law which connects local pressure differences with permeability. Hence within an adiabtical system no transport of vapour and water exists. The influence of the additional water is reduced to nodes of elements that are connected with light weight aggregates and the cement matrix. Since the compressive strength is dependent on the hydration degree the curve progressions do resemble each other.

**Dispersion of moisture**

This picture should provide an impression of the possibilities of the simulations. It shows a plot of the results of the capillary water after 4.2 days. The water content is given in kg/m$^3$. The cube had one open side that permitted an exchange of temperature and moisture. It was the side the z-axis is directed to.
The water content in the center of the Liapor core is reaching 113 kg/m$^3$. This is significantly higher than the moisture content of the surrounding areas. At the free open side the free water content is 99.2 kg/m$^3$. The remaining concrete matrix has a water content of 100.9 kg/m$^3$ while the transition area between the core and the matrix has an average water content of 102.9 kg/m$^3$. These numbers were measured after 4.2 days. The amount of chemically bound water reached after 365 days 80 to 90 kg/m$^3$ (Table 2). The water reservoir of the Liapor grain reaches 20 kg/m$^3$ after one year. The calculated autogenous unrestrained drying shrinkage shows a significant decrease for specimens with water saturated light weight aggregates. Especially if two cubes, one with and one without water saturated light weight aggregate, are compared. The numbers are not corresponding with the measured ones but they do show the same tendency like the in the laboratory gathered results.

### Table 2: chemically not bound water of the concrete matrix

<table>
<thead>
<tr>
<th>mixture</th>
<th>simulation</th>
<th>t=0 d</th>
<th>t=0,15 d</th>
<th>t=360 d</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>max [kg/m$^3$]</td>
<td>max [kg/m$^3$]</td>
<td>max [kg/m$^3$]</td>
<td></td>
</tr>
<tr>
<td>HB 0</td>
<td>adibatical</td>
<td>150</td>
<td>142</td>
<td>57</td>
</tr>
<tr>
<td></td>
<td>1 open side</td>
<td>150</td>
<td>142</td>
<td>65</td>
</tr>
<tr>
<td>HB 15</td>
<td>adiabatical</td>
<td>150</td>
<td>142</td>
<td>72.5</td>
</tr>
<tr>
<td></td>
<td>1 open side</td>
<td>150</td>
<td>142</td>
<td>65</td>
</tr>
</tbody>
</table>
5. DISCUSSION

The calculated results were compared with experimentally obtained numbers. These laboratory experiments have been performed by Weber [3;4]. During the appraisal of the results it should not be forgotten that the size of the simulated cubes was small compared to the ones experimentally tested. This may cause a difference between experimental and calculated results. While adiabatical simulations are not affected by the size of the concrete spheres the simulations with an open side are. Thus the results should be seen as a first approximation in order to show the general tendencies. The size of the models was limited by the number of elements with whom DuCOM can compute.

Hydration

Weber has shown that the hydration degree after 180 days reached 0.67 and after 360 days 0.82 [3]. The used mixture was comparable to the HB 15. The temperature faltered during the experiments between 15 and 25°C and the humidity between 40 and 50%. This boundary condition was comparable to the situation “2 open sides”. After 360 days the calculated hydration degrees of the mixture HB 15 were ranging between 0.76 for 2 open sides, 0.75 for 1 open side and 0.84 for the adiabatical system. For the mixture HB 0 the measured hydration degree was 0.67 after 360 days while the calculated value was 0.73. The difference may be explained by the size effect or simply by the higher humidity during the calculation. Nevertheless the model provides a good estimation for the hydration degree for concretes with light weight aggregates.

Strength development

The compressive strength development is following the curve development of the hydration degree. Table 3 shows the calculated 28 days strength.

| t = 28,0 d | fraction of light weight aggregates [%] |
| --- | --- | --- | --- | --- |
| cube | 0 | 15 | 25 | 30 |
| adiabatical | 139.17 | 144.59 | 144.48 | 144.62 |
| 1 open side | 57.83 | 82.70 | 83.31 | 105.21 |
| 2 open sides | 56.66 | 73.25 | 72.81 | 100.73 |
Laboratory experiments [3] have been performed with the mixture HB 15. The 365 days experimental results are presented in table 4 and compared with the computed ones.

<table>
<thead>
<tr>
<th></th>
<th>experimentally determined compressive strength [N/mm²]</th>
<th>calculated compressive strength [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>adiabatical</td>
<td>122</td>
<td>144.5</td>
</tr>
<tr>
<td>1 open side</td>
<td>120</td>
<td>128</td>
</tr>
<tr>
<td>2 open sides</td>
<td>122</td>
<td>125</td>
</tr>
</tbody>
</table>

On the one hand the results for the calculations with one or two open sides seem to be accurate, on the other the value for the adiabatical calculation is overestimated. The experiments performed found that the concrete matrix is dense enough to circumvent any moisture transport from the outside. The simulation did not reproduce this situation exactly. The porosity was during all time steps high enough to simulate a water loss/gain by evaporation/condensation. The adiabatical simulation nearly reached the maximum compressive strength of 145.5 N/mm². Hence it can be assumed that the amount of available water was sufficient for complete hydration. So the size-effect, the water loss by evaporation and the fact that the maximum compressive strength was reached are reasons why it can be concluded that the strength development is overestimated but it is accurate enough to provide a first approximation.

**Dispersion of moisture**

The mixture HB 15 had 150 kg/m³ water content and additionally 47 kg/m³ were provided by the saturated aggregates. After 360 days the cube with 2 open sides contained 155 kg/m³ and the adiabatically stored cube had 175 kg/m³ water content. The water was found partly chemically bound and partly as liquid in the pores. The calculated results are slightly different. The adiabatical system reached a water content of 170 to 180 kg/m³ and the cube with an exchange with the environment reached 180 to 190 kg/m³. The higher water content of the simulated cube with an exchange with the environment can be explained by the
higher humidity during the simulation, compared with the experimental conditions.

6. SUMMARY

The modified Chaube model proves to be able to simulate high performance concrete. Some of the results seem to be in compliance with the found laboratory results. Although the experimentally obtained amount of data was very humble and the sizes of the models were smaller than the cubes used for the performed experiments, it can be summarized that:

- the 360 days computed hydration degrees were close to the measured ones. Even though due to the lack of data it was only possible to compare concrete at the age of 180 days and 360 days.

- it was found that the porosity and the early age compressive strength is slightly overestimated by DuCOM. The results for the cubes with an exchange with the environment have been very good. The accuracy of the adibatical simulations have been less reliable.

- the influence of the additionally provided water is measurable. The amount of physically and chemically bound water is corresponding with experimentally found numbers.

- the future development of this model where the water saturated light weight aggregates will be part of the governing equations will improve the accuracy of calculated material parameters.

REFERENCES

CORROSION DAMAGES CAUSED BY CAST MAGNESITE FLOOR SCREED

KORROSIONSSCHÄDEN DURCH MAGNESIAESTRICH

DÉGÂTS DE CORROSION DUS AUX CHAPES EN CIMENT MAGNÉSIEN

Willibald Beul und Klaus Menzel

SUMMARY

The use of cast magnesite floor screed demands special protective measures for steel and reinforced concrete because of the release of chloride. Considering a case of damage, the consequences of neglecting corrosion protection are demonstrated. Evolution of the damage is followed up for three years.

ZUSAMMENFASSUNG


RESUME

L'utilisation de chapes en ciment magnésien nécessite des précautions spéciales de protection contre la corrosion pour l'acier et le béton armé, ceci en raison de la libération de chlorures par la chape. A l'aide d'un exemple d'un dégât, les conséquences de mesures de protection insuffisantes sont montrées. L'évolution des dégâts est suivie sur une période de trois ans.

KEYWORDS: Magnesite Screed, Corrosion, Chloride, Concrete, Reinforcement, Headed Studs
1 MAGNESITE SCREED

Magnesite screed is used mainly in industrial buildings and exclusively in interiors, where the following characteristics are required:

- high abrasion resistance
- high resistance against impact
- dustlessness (with appropriate care)
- low heat conductivity
- stability against mineral oils, solvents and fuels
- pleasant appearance
- easy cleaning and repair

It is made of caustic magnesia, additions (fillers), an aqueous salt solution of bivalent metals – generally magnesium chloride – and additives (like colouring materials). Magnesium chloride is an acid, hygroscopic and therefore corrosive salt.

After hardening, the pores are filled with magnesium chloride solution. Magnesium chloride does not dry within relative humidities >32%. Consequently magnesite screed affects corrosively usual building metals and alloys (steel, galvanised steel, some stainless steels). In this connection already very early special preventive measures were prescribed: The paper A50 of “Industrial Building e. V.” [1] demands 1962 for example:

- “The concrete must be and remain dry” and
- “All metal parts coming into contact with the lining are to be protected adequately”

With reference to fittings made of metal, DIN 18560, part 3 [2] states: “metal fittings must be, if necessary, provided with a corrosion protection adapted to the bonding agent of the screed”. Further, DIN 18560, part 3 claims in table 2: “On reinforced concrete, a barrier layer is to be planned”.

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As several cases of damage from the last years show, these regulations are not sufficiently considered. This publication would like to demonstrate the importance of corrosion protection in connection with magnesite.

2 CASE OF DAMAGE

2.1 GENERAL

An industrial building, built in 1997 showed local bulging of the screed (ME 60 and ME 80) in the proximity of columns (fig. 1), where headed studs with relatively small concrete cover were cast in the floor slab. Since the damage arose systematically and was found exclusively within areas with magnesite screed, a connection with the screed works was assumed. The Otto-Graf-Institut was charged with the investigation of the damage.

Fig. 1: Bulges in the floor near column
2.2 INVESTIGATIONS AND RESULTS

In a first visual investigation some of the bulged spots (diameter approx. 25 cm) were opened and examined. Bulging always started from headed studs with small concrete cover (< 2 cm), corroded on top (fig. 2).

Fig. 2: Bulge (bottom side) with rust stains from headed stud

For further investigation 3 cores were drilled. At the boundary screed/concrete no apparently recognisable intermediate layer was found (fig. 3). The reinforcing steel with concrete cover of $\geq 4$ cm was not yet corroded. From the drill cores samples were prepared for chloride analysis. The resulting chloride profile is given in fig. 4. In a depth of 2 cm (at the head of the studs) a chloride content of 0,3 to 0,9 % (related to cement weight) was found, in a depth of 4 cm (at the reinforcement) the concentration decreases to 0,15 to 0,2%.
Corrosion damages caused by cast magnesite floor screed

Concrete

Screed

Fig. 3: Drill core (no visible barrier layer)

Chloride content (concrete below screed)

M% / cement weight

1
0.8
0.6
0.4
0.2
0

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30 - 45

Depth in mm
00000000

00000000

00000000
00 00 00 00 00 00 00 00
left column00 00 00 00 00 00 00 00 2000 right column
0 0 0 0 0 0 0 0 2003

Fig. 4: Chloride profile in the years 2000 and 2003

Closer investigation of the boundary layer magnesite screed/concrete by
means of differential thermal analysis (DTA) and thermogravimetry (TGA)
showed that on the concrete an organic layer had been applied. However did the
95



chloride profiles point clearly to the absence of “sealing”- or “barrier”-characteristics of this layer.

To detect early stages of corrosion, potential measurements were performed. The measurements resulted in potentials within a relatively narrow range from -140 to -50 mV against the saturated calomel electrode. Evaluation of the results by means of statistic methods [3] gave a critical potential value of -80 mV.

2.3 CAUSE OF THE DAMAGE

The systematic uniformity of the apparent damage on all the areas covered with magnesite screed and the agreement of the chloride profiles in different places of the building permitted the conclusion that magnesium chloride solution had penetrated into the concrete and depassivated inserted steel with small concrete cover. The bulging and spalling was the effect of pressure, produced by corrosion products. The majority of the reinforcement of the floor slab did not seem concerned yet, as the results of potential measurements and the low chloride content in this depth (around 0.2 %) suggest.

The cause for the penetration of chloride was the absence of the barrier layer between magnesite screed and reinforced concrete according to DIN 18560 part of 3, table 2. The applied bonding layer was no barrier layer in the sense of DIN 18560.

2.4 PROGNOSIS, REPAIR

At the time of the investigations a chloride profile with decreasing concentration was found. Critical chloride contents had to be assumed only in a depth less than 3 to 3.5 cm. I the course of time, due to the concentration gradient, chloride transport to deeper layers had to be expected. At the same time potentials were expected to increase (due to drying) and critical chloride contents to decrease (because of the higher potential). Chloride diffusion substantially depends on the water content of the concrete and on the kind of the cation. For magnesium chloride, diffusion coefficients three to four times higher than for the more usual sodium chloride were found in laboratory experiments [4].

Because of the above mentioned complications, prognosis regarding chloride ingress and corrosion was not possible with reasonable accuracy from the available data. Therefore it was suggested to repeat potential measurements and chloride analysis at a later time. In a first step, repair was restricted to visibly
Corrosion damages caused by cast magnesite floor screed

bulged, hollow spots, were concrete was removed to a depth of 4 cm and replaced by a special repair mortar after cleaning the studs by means of sand-blasting.

2.5 FOLLOW-UP INVESTIGATIONS

In the years 2002 and 2003 the potential measurements were repeated at the same spots. Chloride profiles were also acquired again in 2003. At this time, additional bulging around six of the columns was found. Potentials are summarised in tab.1. An example of potential readings (2000 and 2003) is given in fig 5. The majority of potentials decreased in the course of time, but the critical value also was found to be lower by 25 mV. Anyway, active corrosion was still indicated at a remarkable number of measuring points, even in areas without headed studs.

Table 1: Results of potential measurements 2000 to 2003

<table>
<thead>
<tr>
<th>Screed</th>
<th>Potential in mV (GKE)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2000</td>
</tr>
<tr>
<td></td>
<td>min.</td>
</tr>
<tr>
<td>ME 60</td>
<td>-140</td>
</tr>
<tr>
<td>ME 80</td>
<td>-</td>
</tr>
</tbody>
</table>

1) Number of measuring points: 194
Fig. 5: Example of Potential Readings 2000 and 2003
Mean values for the chloride profiles 2000 and 2003 are given in fig.4. Unexpectedly, the concentration changes with time were found to be very small.

3 CONCLUSIONS

Neglection of protective measures, especially regarding the “barrier-layer” on top of reinforced concrete floors leads to serious corrosion damages, caused by the ingress of chloride from the cast magnesite screed. Corroding spots can be detected by means of potential measurements (potential mapping). Chloride ingress will be very slow after drying of excess-water in indoor conditions. Nevertheless, active corrosion is still progressing after 3 years, implicating a series of repair measures in the course of time.

REFERENCES

DETERMINATION OF CONCRETE ADMIXTURES IN CONCRETE BY NMR SPECTROSCOPY

UNTERSUCHUNGEN ZUM NACHWEIS VON BETONZUSATZMITTELN IN BETONEN MIT NMR-SPEKTROSKOPIE

DETERMINATION D’ADDITIFS DE BETON PAR SPECTROSCOPIE DE RESONANCE MAGNETIQUE NUCLEAIRE

Uwe Herterich, Gerhard Volland, Günter Krause, Dagmar Hansen

SUMMARY

Based on investigations of typical components - active components and impurities - of common concrete admixtures like water reducers, retarders, plasticizers, mobile components and mobile decomposition products of active components resulting from reactions of concrete admixtures with concentrated alkaline solutions in porous water were determined. Cement mortar and concrete samples with admixtures (admixture concentration 0.5 and 2 % of cement content) were extracted/leached with different organic solvents and aqueous solutions at higher temperatures. The identification and quantification of relevant compounds in initial products and extracts were carried out by nuclear magnetic resonance spectroscopy (¹H NMR) as well as by gas chromatography/mass spectrometry. The obtained data prove on one hand, that concrete admixtures are detectable in concrete with the methods chosen. On the other hand the data proves, that even under “worst-case”-conditions (grounded cement bound building material and extraction/leaching at higher temperatures for more than 5 d) only minor portions of active components and impurities are mobile in water. More than 70 % of the added admixture is irreversible bound to the concrete matrix. Besides small portions of active components mainly formiate and acetate can be detected in aqueous solutions. Of the listed dangerous substances only phosphoric acid tributyl ester can be detected in aqueous extracts of concrete and cement mortar in minor traces.
ZUSAMMENFASSUNG


RESUME

Au cours des études de composants typiques d'additifs de béton commercials – des agents et des matériaux auxiliaires (moyens)- la fraction des additives mobiles ainsi que celle des produits de décomposition mobiles formés dans la réaction avec ciment ont été déterminées.

Des échantillons de béton et de mortier à ciment contenant 0,5 et/ou 2% d’additif (par rapport à la teneur en ciment) ont été extraits avec des différents solvants organiques ainsi qu'avec d'eau pure à températures élevées. L'identification et la quantification des composés soluble pertinents dans les extraits et/ou dans les additifs (produits originaux) ont été accomplies par la spectroscopie de résonance magnétique nucléaire (1H NMR) ainsi que par la chromatographie de gaz/spectrométrie de masse (GC/MS).
Determination of concrete admixtures by NMR spectroscopy

D’une part les résultats prouvent que les additifs de béton appliqués peuvent être déterminés certainement par les méthodes choisies. D’autre part les résultats montrent que même à conditions très dures (mortier à ciment pulvérisé et extraction à températures élevées pendant plus que 5 jours) il y a seulement peu d’agents et/ou de moyens qui peuvent être mobilisés. Plus que 70 % des additifs appliqués sont absorbés irréversiblement par le ciment. Des composés dangereux il n’y a que le tributylphosphate qui est déterminé en quantité minimale.

KEYWORDS: concrete admixtures, determination, NMR, leaching, environmental risks

1. INTRODUCTION

Concrete admixtures are added to concrete and mortar in concentrations of 0.5 – 2%, in order to influence defined characteristics of fresh and/or hardened concrete. In the following only active components and supplies of concrete admixtures used as water reducers, superplasticizers and retarders were regarded. The assigned raw materials are not considered as biologically easily degradable and may not directly discharged in discharge systems, ground- and surface water [1]. The presented investigation serves to identify mobile substances contained in different commercially available concrete admixtures for concrete in contact with ground water and offers a possibility to determine concrete admixtures in concrete. The experimental conditions of the chosen extraction methods in water and organic solvents allow to detect even minor mobile portions of the admixtures. The results of the presented study do give an outline of possible mobile components and/or degradation products of concrete admixtures leached out of concrete or cement mortar. The results cannot be directly transferred to conditions of leaching monolithic concrete bodies in contact with ground water.
2. INVESTIGATED CONCRETE ADMIXTURES

For the investigations 5 common concrete admixtures were selected. Manufacturer’s data of active components of these concrete admixtures are given in the following table.

<table>
<thead>
<tr>
<th>Concrete admixture</th>
<th>Active component (manufacturer’s data)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Acrylate copolymer + naphthaline sulfonate</td>
</tr>
<tr>
<td>B</td>
<td>Acrylate + lignosulfonate + naphthaline sulfonate</td>
</tr>
<tr>
<td>C</td>
<td>Naphthaline sulfonate</td>
</tr>
<tr>
<td>D</td>
<td>Polycarboxylate ester</td>
</tr>
<tr>
<td>E</td>
<td>Lignosulfonate + naphthaline sulfonate</td>
</tr>
</tbody>
</table>

The concrete admixtures were added to cement mortar and concrete (w/c = 0.5) in two different concentrations (0.5 and 2 % relative to the cement content). The maximum permissible dosages for these admixtures to cement are about 10 ml/kg (densities of the admixtures are approx. 1.1 g/cm³). A dosage of 2 % (relative to cement) corresponds thus to an overdosing of approx. factor 2 for concrete plasticizers and represents on the other hand the maximum concentration for superplasticizers. Moreover the selected different concentrations give information on the concentration dependent emission potential in the different polar extraction media.

3. EXPERIMENTAL RESULTS

3.1 Admixtures based on naphthaline sulfonate

Concrete admixtures based on naphthaline sulfonate can be identified by characteristic NMR fingerprints (δ = 7.5 – 8.3; solvent D₂O) after extraction with organic solvents and aqueous solutions. The naphthaline sulfonates (derivatives of the naphthaline sulfonates) determined in water-, methanol- and acetone/H₂O-extracts of cement bound building material are modified compared
to the naphthaline sulfonate of the initial products and differ also (chemically) from monomeric naphthaline sulfonate described in the literature which can be mobilized via the water path (comparison of spectra with Aldrich Library of $^{13}$C and $^1$H FT NMR Spectra, volume 2) (s. also [2]). A structure determination of these compounds could not be made with the selected methods. In extracts of dichloromethane (DCM) of hardened cement paste samples an additional non-polar compound on basis of naphtaline sulfonate can be determined. This compound is definitely not naphthaline. It can be determined in all naphthaline sulfonate containing admixtures investigated and differs chemically (proven by NMR spectra) from naphthaline sulfonate derivatives found in aqueous (polar) extracts. Compared to the total amount of naphthaline sulfonate in aqueous extracts the fraction soluble in DCM is about < 10 to 20 % of the total naphthaline sulfonate content. Naphthaline can neither be determined in DCM-extracts of initial products nor in cement mortar added with naphthaline sulfonate. Yet, naphthaline can be determined in small quantities (< 10 mg/kg concrete admixtures), if admixtures based on naphthaline sulfonate are treated with a strongly alkaline solution at higher temperatures for longer periods of time. This shows that naphthaline sulfonate containing concrete admixtures are no relevant source for naphthaline in ground water and leachate. All concrete admixtures (initial products) based on naphthaline sulfonate are contaminated with different organic solvents like cis and trans decahydronaphthaline (decalin) (admixtures B and C), different n- and cycloalkanes (admixtures A and E) or xylene as well as t-butyl-xylene (admixture A). The concentrations of these impurities are below 5 g/kg admixture.

3.2 Admixtures based on polycarboxylates

Concrete admixtures based on polycarboxylate esters consist of an acid component (polyacrylate backbone) esterified with different polyols. For admixtures A and D the polyacrylate backbone is esterified with polyethylene glycols (resp. methylether derivatives of polyethylene glycol) of different chain length. With the methods chosen it cannot be differentiated whether besides esterified polyethylene glycol (derivatives) non esterified "free" polyethylene glycol is contained in the initial products. Both components, the polyacrylate backbone and the polyethylene glycol side chains are detectable in the tested initial products.
Admixtures which contain polyacrylate/polycarboxylate show characteristic ethoxy-compounds in water-, methanol- and acetone/H$_2$O (1:1) extracts (NMR-fingerprint at $\delta = 3.45 - 3.65$; solvent D$_2$O). These ethoxy compounds are essentially methylethers of polyethylene glycol and possibly nonsubstituted polyethylene glycols. This mixture on basis of polyethylene glycol will be described as polyethylene glycol (derivative) in the following. This class of compounds can be determined in the initial products and represents the main part of active components in concrete admixture D. The polycarboxylate/polyacrylate backbone is not detectable in aqueous eluats and DCM-extracts. In concrete with admixtures based on polycarboxylate/polyacrylate only these characteristic ethoxy compounds (polyethylene glycols) are mobile. This portion of the admixtures can be originated either from an alkaline cleavage of the esters or it consists of "free" non esterified polyethylene glycols, which are already part of the original product. The substantially different results of the mobile quantities of polyethylene glycols for concrete admixtures B and D, regained in aqueous solutions (Fig. 2) clearly point out, that the chemical composition of the products (active components) is essential for the release of polyethylene glycols. Besides those polyethylene glycol derivatives p-toluenesulfonic acid can be determined in aqueous eluates of cement mortar treated with admixture D. The amount of mobile p-toluenesulfonic acid is less than 10 mg/kg cement.

3.3 Admixtures based on lignosulfonates

$^1$H NMR spectra of lignosulfonate are not very characteristic. Lignosulfonate of initial products B and E is detectable by $^1$H NMR (NMR-fingerprint at $\delta = 6.6 - 7.1$; solvent D$_2$O). In aqueous and organic extracts of cement mortar treated with concrete admixtures B and E no significant hint for lignosulfonates can be determined. There are 3 possible reasons for this:

- the sensitivity of the applied method for compounds of this type is so low, that even larger mobile quantities cannot be detected

- the concentration of lignosulfonate in the initial products (B and E), which is relatively low compared to the concentrations of other active components in concrete admixtures, is so small that in fact no lignosulfonate is leached out

- or the solubility of the active component lignosulfonate in general is so low that lignosulfonate is practically not mobilizable via the water path.
3.4 Processing agents and impurities

Besides the active components concrete admixtures contain product-depending various processing agents (see table 2 and 3).

Table 2: Identified components of concrete admixtures (initial products) – results according to nuclear magnetic resonance spectroscopy (1H NMR)

<table>
<thead>
<tr>
<th>Concrete admixture – manufacturer’s data</th>
<th>Content of active components (% after drying)</th>
<th>Content of active components – fraction of naphthaline sulfonate (according to 1H NMR)</th>
<th>Components of concrete admixture (detected by 1H NMR)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>A</strong> Acrylate copolymer + Naphthaline sulfonate</td>
<td>26 approx. 100</td>
<td>Naphthaline sulfonate Polyacrylate Formiate Acetate Methanol Isopropanol</td>
<td></td>
</tr>
<tr>
<td><strong>B</strong> Acrylate + Lignosulfonate + Naphthaline sulfonate</td>
<td>29 approx.65</td>
<td>Naphthaline sulfonate Lignosulfonat Polyacrylate Polyethylene glycol (derivative) Formiate, Acetate, Methanol Isopropanol</td>
<td></td>
</tr>
<tr>
<td><strong>C</strong> Naphthaline sulfonate</td>
<td>21 approx.100</td>
<td>Naphthaline sulfonate Formiate, Acetate, Methanol Isopropanol</td>
<td></td>
</tr>
<tr>
<td><strong>D</strong> Polycarboxylate ester</td>
<td>34 - -</td>
<td>Polycarboxylate Polyethylene glycol (derivative) p-Toluenesulfonic acid</td>
<td></td>
</tr>
<tr>
<td><strong>E</strong> Lignosulfonate + Naphthaline sulfonate</td>
<td>42 approx.105</td>
<td>Naphthaline sulfonate Formiate, Acetate, Methanol Isopropanol Lignosulfonate (side component)</td>
<td></td>
</tr>
</tbody>
</table>

Besides ubiquitous compounds such as formiate and acetate, organic solvents like n- and cycloalkanes, methanol and isopropanol, aromatic compounds
such as xylene and t-butyl-xylene can be determined in the initial products. Most of these solvents are impurities within the trace range (< 100 mg/kg concrete admixtures). Exceptions are t-butyl-o-xylene and decahydronaphthaline (decalin). The ubiquitous compounds formiate and acetate are part of almost all admixtures (except admixture D) and can be determined in all aqueous extracts/leachates, including concrete or cement mortar without admixtures, in differing concentrations. The determined concentrations of formiate and acetate leached out of cement mortar range from < 5 mg/kg for cement mortar without admixtures up to 30 mg/kg for cement mortar treated with admixtures based on naphthaline sulfonate. Besides concrete admixture D, a superplasticizer of the new generation, phosphoric acid tributylester is detectable in all aqueous eluats as well as in all in all examined admixtures. This admixture (D) contains p-toluenesulfonic acid as processing agent, which can be detected in aqueous leachates. Due to the sample preparation easily volatile compounds such as methanol, isopropanol, n- and cycloalkanes cannot be detected in the leachates. Impurities of cement like mineral oil hydrocarbons can be detected in all DCM extracts.

*Table 3: Volatile components of concrete admixtures (Headspace gas chromatography/ mass spectrometry)*

<table>
<thead>
<tr>
<th>Concrete admixture</th>
<th>Volatile components (headspace technique)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Methylcyclopentane, ethylcyclopentane, cyclohexane, methylcyclohexane, dimethylpentane, xylene, t-butyl-o-xylene</td>
</tr>
<tr>
<td>B</td>
<td>cis- trans decahydronaphthaline (decalin)</td>
</tr>
<tr>
<td>C</td>
<td>cis-, trans decahydronaphthaline (decalin)</td>
</tr>
<tr>
<td>D</td>
<td>No volatile components detected</td>
</tr>
<tr>
<td>E</td>
<td>Methylcyclopentane, ethylcyclopentane, cyclohexane, methylcyclohexane, dimethylpentane</td>
</tr>
<tr>
<td>Pure active compo-</td>
<td></td>
</tr>
<tr>
<td>nents</td>
<td></td>
</tr>
<tr>
<td>Lignosulfonate (pure active components)</td>
<td>No volatile components detected</td>
</tr>
<tr>
<td>Naphthaline sulfonate (pure active components)</td>
<td>Decane, cis-, trans decahydronaphthaline (decalin)</td>
</tr>
</tbody>
</table>
4. DISCUSSION

A comparison of solvents of different polarity used for extraction/leaching shows that modified solvents like methanol or acetone/water do not improve the mobility of characteristic compounds. All characteristic compounds found in extracts/leachates with modified solvents can also be detected in (pure) aqueous solutions. Hence, pure water and dichloromethane as solvents are sufficient for the determination of characteristic mobile compounds in concrete with admixtures. Despite limitations regarding the accuracy of $^1$H NMR spectra on water basis the following conclusions can be made:

- About 5 – 30 % of the added amount of active components are in principle mobile when leaching grounded cement mortar added with different admixtures with water for a longer period of time at higher temperatures (5 d at 60 °C). The amounts of leached admixtures are in general less than 100 mg active components per kg cement. Transferred to concrete and an average dosage of admixtures of 0.5 % about 3 – 8 mg of active components and/or their derivatives per kg concrete are mobile under the “worst-case” conditions chosen.

Figure 1: Amount of applied naphthaline sulfonate (NFS) and corresponding amount of NFS found in aqueous solutions of hardened cement paste (w/c = 0.5) with 0.5 and 2 % admixtures.
• An increase of the dosage of the concrete admixture (factor 4) correlates for nearly all mobile compounds (active components) with an increase of the mobilized amounts (factor 2 – 6) (Fig. 1).

• Besides the active components of the concrete admixtures and their degradation products (derivatives of naphtaline sulfonate or derivatives of polyethylene glycol) only phosphoric acid tributylester used as antifoaming agent, p-toluenesulfonic acid and ubiquitous formiate, acetate and mineral oil hydrocarbons are mobile in aqueous media. Other organic compounds cannot be detected neither with polar (water) nor nonpolar (dichloromethane) solvents. Mobilizable amounts are generally < 5 mg/kg for tributylphosphate and 20 – 30 mg/kg for formiate and acetate, respectively (Table 4).

Figure 2: Amount of applied polyethylene glycol (derivative) and corresponding amount of polyethylene glycol (derivative) found in aqueous solutions of hardened cement paste (w/c = 0.5) with 0.5 and 2 % admixtures.
Table 4: Active components of concrete admixtures in the fresh cement mixtures and in aqueous solutions of hardened cement paste.

<table>
<thead>
<tr>
<th>Concrete admixture added 1)</th>
<th>Content of active component - fraction of naphthaline sulfonate (derivative) 2)</th>
<th>Content of active component - fraction of polyethylene glycol (derivative) 2</th>
<th>Amounts of water-soluble active components and supplies in mg/kg hardened cement paste (w/c = 0.5)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>in % relative to cement</td>
<td>in mg/kg cement</td>
<td>Naphthaline sulfonate (derivative)</td>
</tr>
<tr>
<td>A (0.5)</td>
<td>150</td>
<td>n.d.</td>
<td>25</td>
</tr>
<tr>
<td>A (2)</td>
<td>600</td>
<td>n.d.</td>
<td>40</td>
</tr>
<tr>
<td>B (0.5)</td>
<td>100</td>
<td>50</td>
<td>20</td>
</tr>
<tr>
<td>B (2)</td>
<td>400</td>
<td>200</td>
<td>80</td>
</tr>
<tr>
<td>C (0.5)</td>
<td>100</td>
<td>n.d.</td>
<td>30</td>
</tr>
<tr>
<td>C (2)</td>
<td>400</td>
<td>n.d.</td>
<td>80</td>
</tr>
<tr>
<td>D (0.5)</td>
<td>n.d.</td>
<td>1500</td>
<td>n.d. *</td>
</tr>
<tr>
<td>E (0.5)</td>
<td>200</td>
<td>n.d.</td>
<td>50</td>
</tr>
<tr>
<td>E (2)</td>
<td>800</td>
<td>n.d.</td>
<td>100</td>
</tr>
</tbody>
</table>

n.d. = not detectable

* not contained in the initial product

1) addition of initial product

2) calculated of 1H NMR

Altogether, for admixtures based on naphthaline sulfonate the maximum amounts of active component detected in aqueous solutions by NMR, are about 20–30 % of the originally applied amounts of active component. These amounts exceed values given by other authors [2,3,4] by a factor of approx. 2. These differences can be attributed to higher temperatures and the substantially longer times of leaching used in this investigation. In our opinion the values presented here give an upper limit for the amounts of active components which can be leached out of cement bound building materials with admixtures.
For concrete admixtures based on polycarboxylate as active components only polyethylene glycol (resp. methylether derivatives of polyethylene glycol) is mobile in aqueous solutions. The corresponding regaining rates differ however substantially depending on the specific concrete admixture used and vary between 50 and 5 %, whereby the maximum amount of mobile polyethylene glycol reaches 350 mg/kg hardened cement paste (Fig. 2).

5. CONCLUSION

Mobilizable fractions of admixtures of cement-bound building materials and the amount of mobile reaction products were determined. Cement mortar and concrete were extracted with different organic solvents and aqueous solutions. The identification and quantification of the relevant soluble compounds in admixtures and extracts were carried out by nuclear magnetic resonance spectroscopy ($^1$H NMR) as well as by gas chromatography/ mass spectrometry.

In the following some essential results are compiled briefly:

- The components of concrete admixtures can be characterized by $^1$H NMR and GC/MS: Active components as well as side components can be identified and quantified.

- Active components of concrete admixtures applied in concrete and/or cement mortar samples can be determined by characteristic fingerprints in the $^1$H NMR spectra of aqueous solutions and/or organic extracts.

- An increase of the dosage of the concrete admixtures correlates with an increase of the mobilized amounts.

- For admixtures based on naphthaline sulfonate the naphthaline sulfonates determined in aqueous extracts of cement bound building materials are modified compared to the naphthaline sulfonates of the initial products.

- For admixtures based on polycarboxylate/polyacrylate only polyethylene glycol is mobile in aqueous solution. The other part of the active components – the polycarboxylate backbone – is not detectable in aqueous extracts of concrete or cement mortar.

- Besides the active components of the concrete admixtures only tributyl phosphate as well as ubiquitous formiate, acetate and mineral oil hydrocarbons are mobile in aqueous media. Other organic compounds are not detected.
Data indicate that environmental risks of cement-bound building materials with admixtures can be considered as low.

REFERENCES


CONCEPTS OF TRANSIENT RECORDER DEVELOPMENT FOR ACOUSTIC EMISSION ANALYSIS

KONZEPTE DER TRANSIENTENREKORDER ENTWICKLUNG FÜR DIE SCHALLEMISSIONSANALYSE

DES CONCEPTES DU DEVELOPMENT D’UN LECTEUR POUR ANALYSER DES EMISSIONS ACOUSTIQUES

J.H. Kurz, V. Wolter, G. Bahr und M. Motz

SUMMARY

The large number of acoustic emissions occurring during an experiment require a fast and high resolution transient recorder system. A standard personal computer with a RAID system and two data acquisition cards, each has got 4 input channels, 5 MHz sampling rate and 12 bit amplitude resolution, build the hardware platform. Using this hardware two transient recorder software concepts were realized. The first one is an event controlled system where each event is triggered and stored immediately with a rate of up to 50 events per second. The second one is a continuously recording system where the data flow is recorded and stored continuously and the events are extracted afterwards. The advantage here is that really all events are recorded but really large datasets need to be handled.

ZUSAMMENFASSUNG

Die große Anzahl während eines Versuches auftretenden Schallemissionen bedarf eines schnellen und hochauflösenden Transientenrekorders. Ein Standard PC mit einem RAID System und zwei Messkarten, jede mit 4 Kanälen, 5 MHz Sampling Rate und 12-Bit Amplitudenauflösung, bildet die Hardware Plattform. Auf Basis dieses Equipments wurden zwei Transientenrekorder Systeme umgesetzt. Das eine System zeichnet kontinuierlich den Datenfluss auf, aus dem hinterher die einzelnen Schallemissionsereignisse extrahiert werden. Der Vorteil dieses Prinzips ist, dass wirklich alle Schallemissionen erfasst werden. Aller-
Acoustic emissions are defined as the spontaneous release of localised 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 material during the entire load history. It is obvious that the recording of damage processes from the microscopic to the macroscopic scale produces a large number of events during relative short time spans. The number of events can be about several thousands during one test. Due to the large number of occurring acoustic emissions within short periods, it is obvious that a fast recording system is needed. Furthermore, the events' frequencies are in the ultrasonic frequency range. Therefore, the transient recorder needs a high sampling rate, too. The kernel of such a system is the data acquisition part. Here, a fast and high resolution A/D converter in form of a measurement card is generally used. In the case of that more than one card is used, they need to be synchronised. The data then
is stored by a computer system which must be fast during the process of data storage and which must have sufficient disk space. All these processes are controlled by the corresponding software.

The rate of acoustic emissions during experiments especially in concrete can be about 30 or more events per second. Commercial data acquisition systems including the software detect between 2 and more than 100 events per second depending on the time delay of the system during the process of storage. The sampling rate for concrete specimens should be greater than 1 MHz with a high amplitude resolution. The required disk space should be about several gigabyte. The more sensors are used the better the data analysis works. Therefore, in general more than one data acquisition card is needed. The cards then need to be synchronised. These are the benchmarks a good system for acoustic emission analysis on concrete should achieve.

REALISATION OF THE REQUIRED HARDWARE STANDARDS

The most limiting factor concerning the chosen hardware and not mentioned yet, is the constricted budget. With an infinite budget it would be possible to go far beyond the required hardware standards. But with the chosen concept a relatively cheap solution was found which fulfils all mentioned benchmarks.

The chosen hardware is a standard personal computer with an AMD Athlon 1800+ processor and a raid controller onboard. The system has got 1 GB RAM and 3 hard disks, each with about 60 GB memory. The first hard disk contains the operating system while the two other hard disks are the memory for the transient recorder. They are configured as a RAID 0. RAID means Redundant Arrays of Inexpensive Disks. That means a compound of hard disks which complement each other. RAID 0 especially also called striping arranges the data equally on both disks. The data transfer rate is so twice as high as with one disk. With respect to fast data processing on a second system the two RAID disks were set into two carrier bodies.

Data acquisition is realised with two measurement cards of type PCI-6110 from National Instruments implemented in the personal computer. The connection between sensors and data acquisition system is realised via a self designed panel. Each measurement card contains 4 analog inputs, 2 analog outputs and 8 digital in-/outputs as well as two 24 bit counter/timer. Concerning the transient recorder only the analog inputs are needed. Each analog input has got an own ADC with a resolution of 12 bit, differential input mode, input coupling
switchable between DC/AC and 8 bipolar ranges from +/- 0.2 V to +/- 42 V. The sampling rate can be chosen between 1 kS/s and 5 MS/s.

The onboard trigger is a bit limited. It is only possible to trigger on one channel and logical connections between different channels are not possible. A slewrate trigger is not realised, too. Using the analog trigger mode one input channel or an external signal is the trigger source. The trigger level has got a resolution of 8 bit. Within the digital trigger mode an external TTL signal is used. Furthermore, triggering by a software signal is also possible and pre- and posttrigger are supplied, too.

A flexible interface which can be used for generating a timing signal is also implemented on the measuring card. This interface can be used for routing the timing signal to other data acquisition systems or to an external output. The routing possibility is essential for a transient recorder system with two or more cards because there is a need of sending the clock pulse as well as the trigger pulse from the trigger channel to the other measurement cards. This is realised via the RTSI (Real Time System Integration) bus which is a simple connection between the data acquisition cards [PCI-6110/6111, 2000].

The up to now short description of the chosen hardware will be completed by a brief description of the two software systems in the following which control the whole recording implemented on the data acquisition unit. The first one is an event controlled program while the second one is a continuous recording system.

DEVELOPMENT OF AN EVENT CONTROLLED TRANSIENT RECORDER SOFTWARE

The event controlled transient recorder software was developed with LabVIEW 6.1 using the Programming language G which is widespread in the field of data acquisition and measurement systems (LabVIEW, 2002). Using LabVIEW the coding is realised in a graphical way which is different to e.g. Fortran or C# but easy to learn. Furthermore, the program flow in LabVIEW is not sequential but related to the data flow. This simplifies process parallelization. Other programming features like loops and sequences are also available as well as subroutines called SubVI (VI: virtual instrument).

Each LabVIEW program is called a VI and consists of a front panel and a diagram. In the following a simple example shell demonstrate the programming procedures within LabVIEW (Fig. 1).
Two numbers shall be entered on a user interface, then multiplied with each other and finally the result displayed on a moving coil instrument (Fig. 1, left). Pressing the stop button the program is terminated. The buttons, instruments and controls can be placed on the front panel which is the user interface by drag and drop operations. The example shows two controls (factor 1 and 2), a boolean control button (STOP) and a numerical display in form of a moving coil instrument. Each control element or display creates automatically a terminal on the diagram surface (Fig. 1, right). Within the diagram functions, operators and structures are used which can also be placed by drag and drop operations. The diagram's elements are then connected by virtual wires. Fig. 1 (right) shows a while loop (grey border line) and within the loop the two input elements factor 1 and factor 2 and the multiplication operator (cross sign). The multiplication operator is connected to the two input elements and the output display (product) and the stop terminal is connected to the conditional terminal of the while loop. After starting the program two number can be multiplied with each other until the STOP button is pressed which cancels the while loop.

This relative simple example shows the general procedure of programming in G. The transient recorder software is a much more complex problem but it is in principal the same basic system. The software's front panel (Fig. 2) has got a menu bar including the menus File, Settings, Measure, Display and Help.

The menu Settings contains dialogs for the data acquisition settings and for the definition of the auto-sequence. The menu Measure is for starting and stopping the measurement as well as the auto-sequence. The menu Display contains the possible display settings and the Help menu calls the help functions.
Below the menu bar is the symbol bar situated containing the most important functions from the menu bar. The screen's largest part is acquired for displaying the signals of each channel. The display settings of each channel can be chosen by using the change button on the left side of each channel or via the Display menu in the menu bar. At the bottom of the front panel a table for chosen parameters is situated.

The control panel of the data acquisition card is shown in Fig. 3. All essential interfaces and displays can be found in the four index cards Timebase, Input Amplifier, Trigger and Physical Unit. The auto-sequence panel is shown in Fig. 4. The available functions can be chosen from the corresponding interface and are copied into the auto-sequence list. The commands listed in there define the current auto-sequence which is started by pressing the Auto Record button in the symbol bar.

The front panel's diagram structure is the code behind the above shown graphical user interface (Fig. 2). Its architecture will be shown in the following: The VI “Main Panel” is the top level VI (Fig. 5). It consists of 2 while loops
which are executed parallel. The upper loop shows the event control. The user activates an event e.g. by pressing a button on the front panel or choosing a menu entry from the menu bar. The programmer defines which control elements are activated by a certain event using the so called event structure (inner frame in the upper loop of Fig. 5). Each event has got an own case in the event structure whose subdiagram will executed if the corresponding event occurs. As long as no event appears the event structure is in a sleeping mode which saves processor capacity.

Fig. 3: Control panel of the data acquisition card

Fig. 4: Auto sequence panel
Fig. 5: Diagram of the front panel showing 2 parallel while loops

Fig. 5 shows the event case which will be executed if the record button on the front panel (Fig. 2) is pressed. Then an array consisting of symbolic constants with the elements “Config and Route”, “Start and Read Data”, “Unroute” and “Display Data” will be created. This array leaves the events structure using a so called tube with destination to the “First Notifier” (lower loop in Fig. 5). A “Notifier” enables the communication between 2 independent parts of a block diagram. Its principal is comparable to a mailbox which also sends and receives data. The “Notifier's” data output is connected to the input tube of a for loop whose auto-indexing is activated. The tube's output is connected to the selector of a case structure which is situated in the for loop. The auto-indexing affects that the elements of an array which is connected to the input tube of the for loop are passed one after the other to the loop. The loop counter (n-terminal) which normally controls the number of loop executions, does not need to be connected to the loop. The loop is executed with respect to the dimension of the array. The mode of operation works as follows: If the user initiates an event, an array consisting of symbolic constants is situated at the input tube of the for loop. During each execution of the for loop one symbolic constant of the array is read, i.e. During the firts loop execution the “Config and Route” constant which is passed to the selector of the case structure that executes the corresponding “Config and
Route” subdiagram (Fig. 5, lower loop). Having read all constants the “Notifier” lapses into a sleeping mode until a new event is initiated. Other events create arrays consisting of different symbolic constants which call certain case structures. Such a programming architecture is called state machine.

The data transfer of subdiagrams of the state machine is realised by so-called shift registers. A shift register is a pair of connections which are situated on the vertical sides of the loop frame (triangle signs in the lower loop of Fig. 5). The connection on the right hand side (triangle upwards) stores the data at the end of one loop execution, that it can be used in the next run as input by the other connection (triangle downwards). A shift register can be used with different data types. The advantage of such an architecture is the simple way of extending the program. If further functions are needed the corresponding case structures are implemented in the state machine and if needed also in the event control.

Until now the used programming environment (LabVIEW) was introduced and the main features of the graphical user interface of the transient recorder software were shown. In the following a few important details of the coding are presented.

Using 8 sensors two data acquisition cards are needed. Herefore, the synchronisation of the cards is important. Each card can be used separately for signal recording with up to 4 sensors. But for an acoustic emission analysis it is important that the cards have got the same timebase, i.e. they need to be synchronised. Therefore, the SubVI “Config & Route” was developed (Fig. 6). Further SubVIs are implemented in this routine and connected by the “Daisy Chain Procedure” [ObjectVIEW, 2002]. That means the output parameters of one VI represent the input parameters for another one. This leads to a sequential data flow which is useful here.

The left side of Fig. 6 contains the controls “Master device” and “Slave device” which represent the two data acquisition cards. The decision which trigger channel is chosen in the control panel determines which card will be master and which slave. The parameters “Master device”, “Master channel scan list” (list of channels used for measurement), “coupling & input config” (input configuration and input coupling) and “Blocklength” are set in the control panel and passed to the SubVI “AI Config” which is executed first. The sampling rate is set in the
SubVI “Clock config”. Trigger channel, trigger slope, trigger level and hysteresis are set in the SubVI “Trigger config”.

Fig. 6: SubVI “Config & Route” for configuration and routing

The trigger procedure is very important for the whole process of data acquisition. Therefore, a more detailed description will be given. If the trigger condition is fulfilled the analog trigger on the data acquisition card generates an internal trigger signal. This internal digital trigger signal is send by the SubVI “Route signal” to the RTSI bus (connection RTSI 0) where it can be received by the second data acquisition card. Within the post-trigger mode or if the pre-trigger samples are set to zero, this internal signal is the starting point for the data acquisition. During the pre-trigger mode the internal signal stops the acquisition because the data acquisition has already started and the pre-trigger samples must be seized, before the trigger condition is fulfilled.

Each data acquisition card has got a 20 MHz time base from where the needed timing signals are send e.g. to the A/D converter. The board clock can also be routed by the RTSI bus, though other data acquisition cards can use the same time base. This is performed by the second call of the SubVI “Route signal” where the signal “Board clock” is send to the “RTSI bus” (Fig. 6). The next two SubVIs configure the slave device in the same manner as described for the master device. Configuring the trigger of the slave device is different from the master device trigger configuration. Herefore, a digital trigger is used and the trigger channel is not an analog external signal but the RTSI 0 connection which uses the master device trigger signal. Though the both cards are triggered si-
multaneously. Finally the board clock of the slave device is set to the RTSI clock, that the sampling is synchronised in phase and frequency.

A further important point is the error trapping. Most of the SubVIs of the diagram shown in Fig. 6 contain an error input and an error output which is connected to the error input of the following VI. Error in- and output are in form of a cluster which is comparable to a dataset or a structure in text based programming languages. This cluster contains a boolean, a numerical and a string control or indicator. If an error occurs the boolean element is set true, the numerical element contains the error number and the string the error message. The next SubVI which receives the error cluster has then the possibility to decide how to handle the further proceeding. Fig. 7 shows an example for such an error handling in the SubVI “Clock config”. If this SubVI receives an error cluster the case “Fehler” is selected from which the task ID and the error cluster are passed to the following VI (Fig. 7, right). If no error occurs the subdiagram of this case called “Kein Fehler” is executed. If an error occurs during the execution of this case the error cluster is set to error and the error number and the error message “AI Clock Config” is passed to the error output (Fig. 7, left).

The described part of the program containing configuration and routing must only be executed once if the configuration is not changed. The next parts of the software which control the start of the data acquisition or the recording of the data can be repeated without re-configuring and re-routing.

![Image](image_url)

*Fig. 7: Error handling within the SubVI “Clock config”*

After configuration and start of the recording system, the data acquisition cards wait for a trigger condition for data storage. All available VIs concerning
the idle running time before an event occurs blocked the program, i.e. no precast VI could be used. Using a timeout function to stop the data acquisition if no event occurs, did not produce satisfying results, too. Therefore, the SubVI “Read Data” was developed (Fig. 8, left).

![SubVI “Read Data” and Subdiagram “Waiting”](image)

Fig. 8: SubVI “Read Data” (left) and Subdiagram “Waiting” (right) which corresponds to the Subdiagram “Data Available” on the left side for idle running time handling

The SubVi “Read Data” (Fig. 8, left) imports zero datapoints from the master device in a while loop by using the SubVI “AI Read”, i.e. the loop cannot be blocked. The parameter “Scan Backlog” which is returned by this VI says how many datapoints are staying in the buffer after the SubVI’s execution. As long as no trigger condition is fulfilled the “Scan Backlog” value is zero and the subdiagramm “Waiting” (Fig. 8, right) of the following case structure is passing only the master and slave IDs and the error cluster to the following loop execution where again “Scan Backlog” is read out. The break switch within the subdiagramm which can be used via the VI-server function by other VIs enables to stop the data acquisition if the corresponding control button is used on the main panel. If the trigger condition is fulfilled and all data is in the buffer the “Scan Backlog” value is the value of the blocklength. Then the subdiagram “Data Available” is executed where the data readout of the master and the slave device from the buffer is realised. Activating the termination condition of the while-loop within this subdiagramm the SubVI “Read Data” is ceased. The data is now available in the terminals master device data and slave device data for storage or display.
DEVELOPMENT OF A CONTINUOUSLY RECORDING TRANSIENT RECORDER SOFTWARE

The continuously recording transient recorder system is based on the same hardware platform as the event controlled one. The software DevTRec is written in Visual C++. The data acquisition cards are approached by the C-libraries of National Instruments (NI-DAQ) [LabVIEW, 2002].

The data acquisition is organised via a ringbuffer which enables measurements with up to 8 channels and 2 megasamples continuously (Fig. 9).

![Fig. 9: Ring buffer organisation](image)

Though, the experiment is recorded completely. The data analysis is performed afterwards. Due to the buffer arrangement the data is organised block by block in a binary file and needs to be sorted afterwards. Therefore, the optimal performance is guaranteed for data acquisition.

The data acquisition cards write the data block by block into the ringbuffer. That means at first card number 1 writes a block of data into the ringbuffer, then card number 2, then number 1 again and so on. Within each block the data of each channel is lined up in succession. Due to the knowledge of the blocklength it is then possible to put the data in the correct order. This ordered data is written to the so called “Basefile”. The data is still in a binary 12-bit format. The format conversion uses the following formula:

\[
\text{measured value} \times \frac{10}{(2048 \times \text{gain})} = \text{real value}
\]
Finally each event is extracted from the continuous data file. Herefore, a simple trigger level can be chosen which defines the occurring of an event (Fig. 10).

The measured values are compared in succession to the trigger level value. It is possible to decide if a definite number of samples is extracted after the trigger level is reached or if the extraction of the event is finished if the values sink again below the trigger level. The events are written separately to a file whereas an information file is also created for every event which contains the relative trigger time. Due to the fact that the “Basefile” is not erased, the extraction can be repeated with different trigger values.

![Fig. 10: Screenshot of DevTRec](image)

The data visualisation is performed using the above shown conversion formula. Using the menu bar it is possible to decide how many values are shown and how many values per redrawing will be shown. That means it is possible to view even huge signal in a kind of film.

**PERFORMANCE OF THE DEVELOPED SOFTWARE AND CONCEPTS FOR THE DATA ANALYSIS**

The two described transient recorder systems on one hardware platform were developed to be able to store as much events per second as possible. The
The continuous recording system records all occurring acoustic emissions. But they need to be extracted from the continuous data stream. That means the user has to deal with large data sets. The point of concern using this system is not the storing rate but the effective handling of the data.

Each data acquisition card is able to use a sampling rate of 5 MHz. If the two cards are used coupled and synchronised the sampling rate for the event controlled system is reduced to 2.5 MHz. The reason herefore seems to be that the onboard FIFO (First In First Out) memory which has got a capacity of 8 kB, is not able to pass the data fast enough via the PCI bus. However, the continuously recording system is only able to sample with 2 MHz using all 8 channels. The reason is not completely clear. If it is a software error the problem may be searched within the National Instruments software (NI-DAQ), i.e. within the C-libraries. The problem might also be based within the buffer organisation and monitoring routine. It is also possible that the data flow is around the limit of what the system bus of a standard personal computer is able to pass to the hard disk, i.e. the problem is a hardware problem.

The disc space of 120 GB makes recording times of more than 30 minutes possible but the user has to take into consideration that for the acoustic emission extraction further disc space is needed. The extraction is performed by a thresholding algorithm. The event extraction works automatically but however it is relatively slow. Therefore, removable hard discs are used in the transient recorder, so the data analysis can be realised on another computer. A further possibility would be to divide the raw data file, containing the continuous data, into several pieces and apply the extraction procedure on them. Until now there is not much experience in handling the huge data sets gained with the continuous recording system.

The event controlled recording system is not able to record all acoustic emissions occurring during one experiment. But is has got a much better performance than the until now used event controlled system which was only able to record 1 event per second with 5 MHz sampling rate (amplitude resolution 12 bit). A performance test revealed that the new transient recorder system is able to trigger and store about 50 events per second with a sampling rate of 2.5 MHz and 12 bit amplitude resolution. This transient recorder system produces large data sets, too, but much less disc space is needed than with the continuously recording system. The larger data sets enable a better analysis e.g. an automatic onset detection algorithm with a rate of success of about 40 % leads generally to
more significant results with 3000 recorded events than having only recorded 300. Concerning the system's hardware performance better data acquisition cards are available (16 bit amplitude resolution, 12 channels, 10 MHz sampling rate, 6 GB onboard memory) but the price of one of the best systems available now is factor 10 higher than of the one we developed.

ACKNOWLEDGEMENTS

These investigations are part of our work in the collaborative research center SFB 381 at the University of Stuttgart which is financially supported by the Deutsche Forschungsgemeinschaft (DFG). We gratefully acknowledge this support.

REFERENCES


COMPARISON BETWEEN THE DOUBLE-K FRACTURE MODEL AND THE TWO PARAMETER FRACTURE MODEL

VERGLEICH ZWISCHEN DEM DOPPEL-K-BRUCHMODELL UND DEM ZWEIPARAMETERBRUCHMODELL

COMPARAISON ENTRE LE MODELE DE FRACTURE DOUBLE-K ET LE MODELE DE RUPTURE DEUX PARAMETRES

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ABSTRACT

Concrete fracture experiments on both the three-point bending notched beams and the wedge splitting specimens with different relative initial crack length ($a_0/D$) were carried out according to the experimental requirements for determining the fracture parameters introduced in the Double-K Fracture Model (DKFM) proposed by Xu and Reinhardt in recent years and the Two Parameter Fracture Model (TPFM) proposed by Jenq and Shah in 1985. The results of the comparison showed that the critical crack length $a_c$ determined using the two different models are hardly different. The values of $K_{lc}^{un}$ and $CTOD_c$ measured for the DKFM are in good agreement with $K_{lc}^s$ and $CTOD_c$ measured for the TPFM.

However, the testing procedure of the TPFM needs a closed-loop testing machine whereas the DKFM needs only monotonic loading. In the TPFM a high order nonlinear equation has to be solved in order to get the relevant parameters whereas in the DKFM the parameters can be determined analytically on a pocket calculator. Furthermore, the DKFM supplies more information on the fracture process.
ZUSAMMENFASSUNG

Betonbruchversuche an gekerbten Dreipunktbiegebalken und Keilspaltenproben mit unterschiedlicher Anfangsrisslänge ($a_0/D$) wurden durchgeführt, wobei die Versuchsbedingungen zur Bestimmung der Bruchparameter entsprechend dem Doppel-K-Bruchmodell (DKFM), kürzlich vorgeschlagen von Xu und Reinhardt, und dem Zweiparameterbruchmodell (TPFM), 1985 vorgeschlagen von Jeng und Shah, eingehalten wurden. Die Ergebnisse des Vergleichs zeigten, dass sich die kritische Risslänge $a_c$ nach den zwei Modellen kaum unterscheidet. Die Werte von $K_{lc}^{un}$ und $CTOD_c$, gemessen für das TPFM, sind in guter Übereinstimmung mit $K_{lc}^s$ und $CTOD_c$ nach dem DKFM.

Die Versuchsdurchführung des TPFM benötigt indessen eine verformungsgesteuerte Prüfmaschine, während beim DKFM monotone Belastung ausreicht. Im TPFM muss eine nichtlineare Gleichung höherer Ordnung gelöst werden, um die maßgeblichen Bruchparameter zu bekommen, während diese im DKFM analytisch mit einem Taschenrechner bestimmt werden können. Außerdem enthält das DKFM mehr Information über den Bruchprozess.

RESUME

Des essais de rupture de béton ont été réalisés par flexion trois points sur des éprouvettes entaillées et par partage en biseau. Ces essais ont été réalisés pour des longueurs de fentes initiales ($a_0/D$) variables selon les exigences expérimentales pour la détermination des paramètres de rupture introduits dans le modèle de rupture double-K (DKFM) proposé récemment par Xu et Reinhardt et le modèle de rupture deux paramètres (TPFM) proposé par Jenq et Shah en 1985. Les résultats montrent que les longueurs de la fente critique $a_c$ déterminées pour les deux modèles ne diffèrent guère. Les valeurs de $K_{lc}^{un}$ et $CTOD_c$ mesurées pour le DKFM correspond bien avec les valeurs de $K_{lc}^s$ et $CTOD_c$ mesurées pour le TPFM.

Cependant la procédure d’essai du TPFM requiert une machine à contrôle en boucle fermée, tandis qu’un chargement monotone suffit pour le DKFM. Pour le TPFM, une équation non-linéaire de haut ordre doit être résolue afin de déterminer les paramètres centraux, tandis que les paramètres du DKFM peuvent être déterminés analytiquement sur une calculatrice de poche. En outre, le DKFM livre plus d’informations sur le processus de rupture.
1 INTRODUCTION

Past attempts at describing the fracture behavior of concrete from the standpoint of conventional linear elastic fracture mechanic (LEFM) have not been very successful, because of the existence of the fracture process zone (FPZ) and the cohesive force ahead of a traction-free crack. In order to predict the crack propagation and to reflect the effluence of the FPZ on the fracture characteristic of materials, several fracture models, like the fictitious crack model (FCM) by Hillerborg et al. (1976), the crack band model (CBM) by Bazant and Oh (1983), the two parameter fracture model (TPFM) by Jenq and Shah (1985), the effective crack model (ECM) by Karihaloo and Nallathambi (1990) and Swartz and Refai (1987) as well as the size effect model (SEM) by Bazant, Kim and Pfeiffer (1986) have been presented. Based on different hypothesis and explanation for the phenomenon of non-linearity observed in tests, many of these models introduced the modified fracture parameters to predict the fracture behavior of concrete structures still by applying the conventional LEFM.

Typical among aforementioned models is TPFM. In the TPFM, two fracture parameters are proposed, namely the critical stress intensity factor $K_{lc}$ defined as the stress intensity factor calculated at the critical effective crack tip and the critical crack tip opening displacement $CTOD_c$ defined as the crack tip opening displacement calculated at the original notch tip of the specimen. For determining them, an unloading and reloading procedure is needed to be performed in tests so that an unloading compliance $c_u$ can be used to evaluated the effective crack length $a_c$. Then the measured value of the peak load $P_{max}$ and the evaluated value of the effective crack length $a_c$ are inserted into a formula of LEFM to determine $K_{lc}$ and $CTOD_c$.

In recent decades, more and more experimental investigations have showed that the fracture process in concrete structures includes three manifest stages: crack initiation, stable crack propagation and unstable fracture (or failure). So it is hoped that any fracture model could be depict these three stages in crack propagation. While all the above-mentioned fracture models can only be used to predict the unstable fracture of concrete structures without considering the crack initiation. For a normal structure, it may be sufficient only to predict its failure or unstable fracture under given loading or displacement conditions accurately. But for some special and important structures, for example, for a concrete pressure vessel or a high concrete dam, accurate prediction of both failure and crack
initiation are imperative. In some cases, accurate prediction of the crack initiation is more important. In engineering practice, one may expect a fracture model is not only accurate for predicting the behavior of cracked structures, but also simple for evaluating the corresponding fracture parameters introduced in the model. Therefore an analytical fracture model that can contain these three stages and also easy to be conducted in tests is required for practical purpose.

In order to reflect the different stages in concrete fracture, a double-$K$ fracture criterion is proposed by Shilang Xu and Reinhardt (1999a). In the double-$K$ fracture criterion, the two fracture parameters ($K_{lc}^{ini}$ and $K_{lc}^{un}$) are introduced, both of them are given in terms of stress intensity factor. $K_{lc}^{ini}$ is called the initiation toughness and its value is determined by inserting the initial cracking load $P_{ini}$ and the initial crack length $a_0$ into a formula of LEFM. $K_{lc}^{un}$ is termed the unstable fracture toughness or the critical stress intensity factor and its value is determined by inserting the measured maximum load $P_{max}$ and the measured critical effective crack length $a_c$ into the same formula of LEFM. It is found that $K_{lc}^{ini}$ and $K_{lc}^{un}$ are size-independent for the tested specimens. Also, for determining the double-$K$ parameters, there is no need to unloading and reloading procedure, and a closed-loop system is not necessary.

In this report, a detailed comparison between the double-$K$ model and the TPFM is made to clearly see the main difference between them and well understanding the effect of FPZ and cohesive force on the fracture characteristic of concrete material.

2 THE COMPARISON IN THEORY AND EXPERIMENT METHOD BETWEEN THE DOUBLE-$K$ CRITERION AND TPFM

A load-$CMOD$ (crack mouth opening displacement) plot of a typical pre-notched beam tested in three-point bending is shown in Fig. 1. The nonlinear displacement can be attributed to the slow but stable crack growth preceding the attainment of the peak load. The effective crack length $a_c$, the sum of the initial crack length $a_0$ plus the stable crack growth $a_c \Delta a_c$ is corresponding to the maximum load $P_{max}$. While in the reasonable evaluation of the effective crack length $a_c$, the TPFM and the double-$K$ are based on the different hypothesis and theory concept. In the TPFM it is considered that the nonlinearity segment on the $P$-$CMOD$ is mainly due to the elastic $CMOD_{n}^{e}$ and only the elastic part $CMOD^{e}$ or the compliance $C_{e}$ measured on the unloading line $AA^{1}$ is taking into account to
calculate the effective crack length $a_c$. While it may be lead to an underestimate of the $a_c$, because the nonlinear behavior in the $P$-$CMOD$ plot results from both the residual $CMOD^p$ that the plastic-frictional energy dissipated on it cannot be neglected (Bazant, 1996) and the $CMOD^e_n$, the difference between the initial elastic compliance $C_i$ and the unloading elastic compliance $C_u$. So in the double-$K$, based on the linear asymptotic superposition, the scant compliance $e_s$ as illustrated in Fig. 1, or $CMOD_c$ (including the elastic part $CMOD^e$ and the unrecoverable deformation $CMOD^p$), is used to calculate the effective crack length $a_c$, which consists of an equivalent-elastic stress-free crack and an equivalent-elastic fictitious crack extension.

![Fig. 1 A load-CMOD curve tested on the three-point bending beam](image)

Another difference between these two models lies in the selection of fracture parameters. As above-mentioned, the TPFM choose the critical stress intensity factor $K_{lc}$, similar to unstable fracture toughness $K^{un}_{lc}$ in double-$K$, it can predict the unstable fracture of concrete structures. But, this model cannot be used to depict the crack initiation which has been observed by many researchers with different investigating methods. Therefore, for some special cases the applications of this model are somewhat restricted. Yet, in the double-$K$, the initiation fracture toughness $K^{ini}_{lc}$ is also introduced to represent the onset of stable crack propagation. Besides, these two fracture toughness are not isolated, the
difference is the cohesion toughness $\Delta K_c$ due to cohesive forces distributed on the fictitious crack during crack propagation. Their relationship is as follows (Shilang Xu and Reinhardt, 1999b):

$$\Delta K_c = K_{lc}^{ini} - K_{lc}^{un}$$

The dissimilarity in the explanation of cause of non linear feature in $P$-$CMOD$ curve leads to the difference in test methods. In the TPFM, unloading compliance $C_u$ is needed to calculate the effective crack length $a_c$, so at least one unloading and reloading procedure should be carried out, and for achieving the stable unloading after the maximum load, a closed-loop testing system is necessary. However some advantages in the TPFM are favourable. For example, only a single size of three-point bend beams is needed in the tests, all of fracture parameters, like $K'_{lc}$, $CTOD_c$, $a_c$ can be directly measured. So it is possible that the properties of size-independence of $K'_{lc}$ and $CTOD_c$ claimed by Jenq and Shah (1985) which are evaluated by the method described in (Jenq and Shah, 1985: RILEM, 1990) could be further justified by the results that are directly measured. In double-$K$, for the mensuration of the double-$K$ fracture parameters, $K_{lc}^{ini}$ and $K_{lc}^{un}$, tests on a single size of three-point bending notched beams are needed. The testing procedure is rather simple without unloading and reloading procedures. It only needs to apply monotonously a load on a beam until the maximum load is gained and to measure the rising branch of a $P$-$CMOD$ curve. For achieving such an aim to measure the initial compliance $c_i$ and the $c_s$ in tests, a closed-loop testing system is not necessary.

3 TEST RESULTS AND CALCULATIONS

Because the main difference is the evaluation of effective crack length $a_c$ owning to the different explanation of non-linear segment in a $P$-$CMOD$ plot, so the emphasis is focused on the calculation of $a_c$. Tests to determine the fracture parameters are performed on two groups specimens, standard three-point bending notched beams denoted with serials B and the wedging splitting specimens represented by serials WS. For these two groups, the cubical compressive strength $f_{cu}$ is 47.96 MPa, and the maximum size of the coarse aggregate is 20 mm, and the static dead load for specimens is 24 KN/m³, and the self weight of loading facilities is 0.23 KN.
The configuration of three-point bending notched beam (BM) and wedging splitting specimens (WS) are illustrated in fig. 2. The dimensions for series BM are 800×200×200 (span×depth×thickness) with ratio of the initial crack length $a_0$ against $D$ as the variable, and series WS with dimensions 200×200×200 ($D_1×H×B$). During tests, load is measured through a sensor and a clip gauge is employed to record the crack mouth opening displacement $CMOD$, and these data are picked continuously through “GRAB” picking system. The test results in terms of $P$-$CMOD$ for BM (or $P_v$-$CMOD$ for WS) curve is shown in fig. 3 and fig. 4, and the measured results like the peak load $P_{max}$ for BM (or $P_{vmax}$ for WS) the $CMOD_c$, the initial compliance $C_i$, the scant compliance $C_s$, as well as unloading $C_u$ are listed in Table 1 and Table 2. With these directly measured data, fracture parameters such as the critical stress intensity factor $K_{lc}$, the critical crack tip opening displacement $CTOD_c$ in TPFM, $K_{lc}^{ini}$ and $K_{lc}^{fin}$ in the double-$K$, and the effective crack length $a_c$ can all be determined.

Fig. 2 (a) the configuration of three-point bending beam (BM)
Fig. 2 (b) the configuration of wedge splitting specimen (WS)

Fig. 3 (a) \( a_0/D = 0.2 \)
Comparison between the Double-K Fracture Model and the Two Parameter Fracture Model

Fig. 3 (b) $a_0/D=0.3$

Fig. 3 (c) $a_0/D=0.5$
Fig. 3 (d) $a_o/D=0.6$

Fig. 4 (a) $a_o/D=0.353$
Comparison between the Double-K Fracture Model and the Two Parameter Fracture Model

Fig. 4 (b) $a_0/D=0.471$

Fig. 4 (c) $a_0/D=0.588$
Table 1  The measured results of series BM specimens (S × D × B = 800 × 200 × 200mm, $H_0 = 1$mm, $f_{cu} = 47.96$MPa)

<table>
<thead>
<tr>
<th>Nos. of Specs.</th>
<th>$a_0/D$</th>
<th>$P_{\text{max}}$ (KN)</th>
<th>CMOD$_c$ (mm)</th>
<th>$C_c \times 10^{-3}$ (mm/KN)</th>
<th>$C_s \times 10^{-3}$ (mm/KN)</th>
<th>$C_u \times 10^{-3}$ (mm/KN)</th>
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<tbody>
<tr>
<td>BM102</td>
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<td>1.872</td>
<td>4.054</td>
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<td>0.0498</td>
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</tr>
<tr>
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<td>4.177</td>
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<tr>
<td>BM2803</td>
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<tr>
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</tbody>
</table>
Comparison between the Double-K Fracture Model and the Two Parameter Fracture Model

Table 2: The measured results of series WS specimens \((D_1 \times H \times B = 200 \times 200 \times 200\text{mm}, \ H_0 = 1\text{mm}, \ f_{cu} = 47.96\text{MPa})\)

<table>
<thead>
<tr>
<th>Nos. of Specs.</th>
<th>(a_0/D)</th>
<th>(P_{\text{max}}) (KN)</th>
<th>CMOD (_c) (mm)</th>
<th>(C_{ac10^{-3}}) (mm/KN)</th>
<th>(C_{ic10^{-3}}) (mm/KN)</th>
<th>(C_{uc10^{-3}}) (mm/KN)</th>
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</thead>
<tbody>
<tr>
<td>WS102</td>
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<td>0.0648</td>
<td>6.613</td>
<td>2.796</td>
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<td>WS2902</td>
<td>0.235</td>
<td>6.847</td>
<td>0.0459</td>
<td>6.704</td>
<td>2.832</td>
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<td>WS1803</td>
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<td>4.878</td>
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<tr>
<td>WS2103</td>
<td>0.353</td>
<td>6.806</td>
<td>0.0675</td>
<td>9.918</td>
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<td>8.212</td>
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<td>WS2203</td>
<td>0.353</td>
<td>5.74</td>
<td>0.0663</td>
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<td>4.760</td>
<td>10.591</td>
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<td>5.125</td>
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<td>10.771</td>
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<td>WS2403</td>
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<tr>
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<td>WS1205</td>
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<td>0.0576</td>
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<tr>
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<tr>
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<td>0.0804</td>
<td>37.711</td>
<td>15.284</td>
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<td>32.558</td>
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<tr>
<td>WS2005</td>
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<td>2.583</td>
<td>0.0675</td>
<td>26.132</td>
<td>15.512</td>
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<td>0.1044</td>
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<td>53.078</td>
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<td>0.0699</td>
<td>54.123</td>
<td>33.592</td>
<td>42.759</td>
</tr>
</tbody>
</table>

Then, the necessary equations for the calculation of fracture parameters for series BM and WS specimens will be detailed subsequently.
THE CALCULATIONS EQUATIONS FOR BM

First, the Young’s modulus $E$ can be calculated from the measured initial compliance $c_i$ of $P$-$CMOD$ curve as follows (RILEM, 1990):

$$E = 6S a_o V_1(\alpha_o) / \left[ C_i D^2 B \right]$$

(2)

for $S/D=4$, the function $V_1(\alpha_o)$ is given by

$$V_1(\alpha_o) = 0.76 - 2.28\alpha_o + 3.87\alpha_o^2 - 2.04\alpha_o^3 + 0.66/(1 - \alpha_o)^2$$

(3)

where

- $a_0$ = initial crack length;
- $\alpha_0 = (a_0 + H_0) / (D + H_0)$;
- $H_0$ = thickness of clip gauge holder;
- $S$ = specimen loading span;
- $D$ = beam depth;
- $B$ = beam width;
- $C_i$ = the initial compliance from $P$-$CMOD$ curve.

In this step, the TPFM and the double-$K$ is the same, the main difference lies in the determination of the effective crack length. Based on the linear asymptotic superposition assumption, the double-$K$ solves the effective crack length denoted by $a_{ck}$ by LEFM as follows (Tada, 1985):

$$E = 6S a_c V_1(\alpha_c) / \left[ C_s D^2 B \right]$$

(4)

where

- $a_c$ = critical effective crack length to be determined;
- $\alpha_c = (a_c + H_0) / (D + H_0)$;
- $C_s$ = the scant compliance from $P$-$CMOD$ curve, equal to $CMOD_c / P_{\text{max}}$.

While, in the TPFM, the effective crack length denoted by $a_{cp}$ is calculated from the unloading compliance $C_u$ at 95% of peak load, so substitute $C_u$ for $C_s$ in equation (4), the $a_{cp}$ described in the TPFM can be got. The comparison of $a_{ck}$ and $a_{cp}$ is listed in table 3.
### Table 3: The comparison of the TPFM and the double-K model in terms of effective crack length for BM specimens

<table>
<thead>
<tr>
<th>No.</th>
<th>$a_0/D$</th>
<th>$C_x \times 10^{-3}$ (mm/KN)</th>
<th>$C_s \times 10^{-3}$ (mm/KN)</th>
<th>$C_u \times 10^{-3}$ (mm/KN)</th>
<th>$E$(Mpa)</th>
<th>$a_{ck}/D$</th>
<th>$a_{cq}/D$</th>
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<td>0.2</td>
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<td>29.869</td>
<td>39270</td>
<td>0.688</td>
<td>0.695</td>
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</table>

After the effective crack length is known, the fracture toughness $K_{lc}^{ini}$ and $K_{lc}^{un}$ can be decided. As previous stated, the unstable fracture toughness $K_{lc}^{un}$ is corresponding to $(P_{max}, a_{ck})$, so it can be evaluated by inserting the maximum load $P_{max}$ and the critical crack length $a_{ck}$ into the following expression (Tada et al., 1985):

$$K_{lc}^{un} = 3(P_{max} + 0.5W)SF_1(a_{ck})/(2D^2B)$$  (5)

in which

$$F_1(a_{ck}) = \frac{1.99 - a_{ck}(1-a_{ck})(2.15 - 3.93a_{ck} + 2.7a_{ck}^2)}{(1 + 2a_{ck})(1-a_{ck})^{3/2}}$$  (6)

where

$$a_{ck} = a_{ck}/D$$

$$W = W_0S/L$$, and $W_0$ is the self weight of the beam.

As for the initial fracture toughness $K_{lc}^{ini}$, it will be calculated based on the equation (1), and the details of analytical solution for $K_{lc}^{ini}$ caused by the cohesive force is presented in (Shilang Xu and Reinhardt, 2000), herein it is briefed for the completeness of the report.
As shown in Fig. 5, the distributed cohesive force $\sigma(x)$ is replaced by a concentrated load $P_e$, acting on the centroid of the cohesive force $\sigma(x)$, and the equation for $K^c_{ic}$ can be written as (let $\beta = \sigma_s(CTOD_c)/f_t$, $V_0 = a_0/D$, $V_c = a_ck/D$, $U_c = x_c/a_c$, $x_c$ is the distance of the acting point of $P_e$ from the bottom of the beam):

$$K^c_{ic}/f_t\sqrt{D} = \frac{2P_e}{f_t\sqrt{\pi}a_c}Z(U_e, V_0/V_c)F(U_e, V_c) = (1 + \beta)\sqrt{V_c/\pi}(1 - V_0/V_c)$$

where

$$U_e = x_c/a_c = \frac{(2 + \beta + (1 + 2\beta)V_0/V_c)}{3/(1 + \beta)}$$

$$F(U_e, V_c) = \frac{3.52(1 - U_e)}{(1 - V_c)^{3/2}} - \frac{4.35 - 5.28U_e}{(1 - V_c)^{1/2}} + \frac{1.30 - 0.30U_e^{3/2}}{[1 - U_e^2]^{1/2}} + 0.83 - 1.76U_e \{1 - (1 - U_e)V_c\}$$

$$Z(U_e, V_0/V_c) = \frac{6(1.025 - 0.1\beta)}{1 + 1.83(V_0 - 0.2)}\left(\frac{V_0}{V_c}\right)^p \sqrt{\frac{V_c}{\pi}} \frac{U_c^{0.2}}{0.2 \leq V_0 \leq 0.8}$$

in which, $p = 1.5(V_0 - 0.2) + 0.8$, when $0.2 \leq V_0 \leq 0.6$; $p = 3(V_0 - 0.6) + 1.4$, when $0.6 \leq V_0 \leq 0.7$; and $p = 6(V_0 - 0.7) + 1.7$, when $0.7 \leq V_0 \leq 0.8$.

During determining the $\sigma_s(CTOD_c)$, the bilinear softening traction-separation law is adopted as sketched in fig. 6.
The area under the $\sigma$-$w$ in fig. 2 is defined as the fracture energy $G_F$ of concrete material (Hillerborg, 1976). The bilinear softening-traction separation law can be listed as:

$$
\begin{align*}
\sigma &= f_i - (f_i - \sigma_s)w/w_s && 0 \leq w \leq w_s \\
\sigma &= \sigma_s (w_0 - w)/(w_0 - w_s) && w_s \leq w \leq w_0 \\
\sigma &= 0 && w \geq w_0
\end{align*}
$$

For determining value of the break point $(w_s, \sigma_s)$ and the crack width $w_0$, Xu proposed a formulized method based on concrete material’s physical meaning (Shilang Xu, 1999)

$$
\begin{align*}
w_s &= 0.4\sqrt{\alpha_F G_F/f_i} \\
\sigma_s &= \left(2 - 0.4\sqrt{\alpha_F}ight)f_i/\alpha_F \\
w_0 &= \alpha_F G_F/f_i \\
\alpha_F &= \lambda - d_{\text{max}}^{0.9}/8 \\
G_F &= (0.0204 + 0.0053d_{\text{max}}^{0.95}/8)(f_{\text{c}}/f_{\text{ck}})^{0.7} \\
\lambda &= 10 - \left[f_{\text{ck}}/(2f_{\text{ck}})\right]^{0.7}
\end{align*}
$$
where $f_t$, $f_c$ are the tensile and compressive strength in MPa; $f_{c0}=10$MPa; $G_F$ is the fracture energy in N/mm; $d_{\text{max}}$ is the maximum size of aggregate in mm; $f_{ck}$ is the characteristic strength representing the concrete grade in MPa; $f_{ck0}=10$MPa. According to CEB-FIP Model Code 1990, there is a relation of $f_c = f_{ck} + 8$ MPa. Now it can be seen if concrete grades and the maximum size of aggregate in the concrete are known, all parameters needed in the bilinear softening traction-separation curve can be certainly determined according to equations (12).

In the previous expression (11), one needs to know the $CTOD_c$ corresponding to the peak load to give the correct evaluation $\sigma_s(CTOD_c)$ needed in equation (7). The following expression is used to determine $CTOD_c$ (Jenq and Shah, 1985):

$$CTOD_c = CMOD\left[1 - \frac{a_o}{a_c}\right]^2 + \left(1.018 - 1.149\frac{a_c}{D}\left[\frac{a_o}{a_c} - \left(\frac{a_o}{a_c}\right)^2\right]\right)^{0.2}$$ \hspace{1cm} (13)

Up to now, the whole procedure for determine the double-$K$ parameters has been completed. The test results from series BM is illustrated in Table 4, also their values are visualized in Fig. 7.
### Table 4 The results of double-K parameters from series BM

<table>
<thead>
<tr>
<th>Nos. of specs</th>
<th>( \alpha_0/D )</th>
<th>( \alpha_0/D )</th>
<th>( CTOD_c ) (mm)</th>
<th>( E ) (Mpa)</th>
<th>( K_{ic}^c ) (Mpa m(^{1/2}))</th>
<th>( K_{ic}^{ini} ) (Mpa m(^{1/2}))</th>
<th>( K_{ic}^{un} ) (Mpa m(^{1/2}))</th>
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<td>37950</td>
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<td>0.016</td>
<td>36760</td>
<td>0.492</td>
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</tr>
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<td>BM3402</td>
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<td>0.035</td>
<td>34255</td>
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<td>0.987</td>
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</tr>
<tr>
<td>BM3502</td>
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<tr>
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<td></td>
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<td>0.1680</td>
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<tr>
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<td>0.565</td>
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<tr>
<td>BM2903</td>
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<tr>
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<td>0.567</td>
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<td>29322</td>
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</tr>
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</table>
THE CALCULATIONS EQUATIONS FOR WS

WS specimens are gaining more and more attentions because of its nonconsideration of the specimen weight during the determination of fracture parameters. They are widely used now to measure the fracture parameter of concrete materials (Xu et al., 1991; Brühwiler and Wittmann, 1990).

For WS specimens, the same procedure is conducted to get the comparison of the effective crack length \( a_{ck} \) in double- and \( a_{cp} \) in the TPFM, and the double-\( K \) fracture toughness \( K_{lc}^{ini} \) and \( K_{lc}^{un} \) can also be obtained. The following briefed the expression used in the calculation.

The formula to determine the fracture parameters of wedge splitting specimens are the same to the CT (compact tension) due to the geometry and the loading condition like each other. During the test, the vertical load \( P_v \) and the \( CMOD \) are recorded, this is not like the CT in which the \( P_h \) and \( CMOD \) on the load line are directly recorded. But certain relation is existed when taking the wedge angle \( \alpha \) (as illustrated in Fig. 2(b)) into account:

\[
P_h = P_v / (2tg\alpha)
\]  

(14)

in this report, \( \alpha \) is equal to \( 15^0 \).
For the geometry of test specimens WS used in this report basically satisfy the standard CT-specimen recommended by ASTM standard E-399-72 (1972), the formula for determining the Young’s modulus can be listed as (Murakami, 1987):

\[ E = V_2(\alpha_0)/BC_{ih} \]  

(15)

where

\[ V_2(\alpha_0) = (1+\alpha_0)^2(2.163 + 12.219\alpha_0 - 20.065\alpha_0^2 - 0.9925\alpha_0^3 + 20.609\alpha_0^4 - 9.9314\alpha_0^5) \]  

(16)

in which

\[ \alpha_0 = (a_0+H_0)/(D+H_0); \]

\[ C_{ih} = \text{the initial compliance of } P_h-CMOD, \text{ equal to } 2tg\alpha C_{iv}, C_{iv} \text{ is the initial compliance of } P_v-CMOD \]

\[ B = \text{the thickness of WS specimens}; \]

\[ H_0 = \text{thickness of clip gauge holder}. \]

This step for calculating the Young’s modulus is the same for both the TPFM and the double-K criterion.

While for the determination of the effective crack length, attentions may be paid to employment of the compliance of \( P_h-CMOD \) curve or \( P_v-CMOD \) curve

\[ E = V_2(\alpha_{ck})/BC_{sh} \]  

(17)

where

\[ \alpha_{ck} = (a_{ck}+H_0)/(D+H_0); \]

\[ C_{sh} = \text{the scant compliance from } P_h-CMOD \text{ curve }, \text{ equal to } CMOD/P_{maxsh}, \]

or equal to \( 2tg\alpha C_{sv}, C_{sv} \text{ is the scant compliance from } P_v-CMOD \text{ curve}. \)

For the TPFM, the effective crack length \( a_{cp} \) is using the same expression except the use of different compliance, replace the \( C_{sh} \) in equation (17) with \( C_{uh} \), or \( C_{sv} \) with \( C_{uv} \) which is the unloading compliance of \( P_v-CMOD \) curve of 95% peak load. The calculation results are listed in Table 5.
Table 5  The comparison of the TPFM and the double-K in terms of effective crack length for WS specimens

<table>
<thead>
<tr>
<th>No.</th>
<th>a/D</th>
<th>Cn x 10^-3 (mm/KN)</th>
<th>Cs x 10^-3 (mm/KN)</th>
<th>Cuv x 10^-3 (mm/KN)</th>
<th>E(Mpa)</th>
<th>a/k/D</th>
<th>a/k/D</th>
</tr>
</thead>
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<td>6.613</td>
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<td>35315</td>
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<td>5.618</td>
<td>34860</td>
<td>0.409</td>
<td>0.378</td>
</tr>
<tr>
<td>3</td>
<td>WS2103</td>
<td>4.823</td>
<td>9.918</td>
<td>8.212</td>
<td>36150</td>
<td>0.498</td>
<td>0.466</td>
</tr>
<tr>
<td>4</td>
<td>WS2203</td>
<td>4.760</td>
<td>11.551</td>
<td>10.591</td>
<td>36625</td>
<td>0.530</td>
<td>0.520</td>
</tr>
<tr>
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While for the double-K fracture toughness $K_{lc}^{ini}$ and $K_{lc}^{ini}$ in WS specimens, the same procedure is samely carried out as series BM as presented above, and the calculated results are listed in Table 6 and Fig. 8.
### Table 6 The results of double-K parameters from series WS

<table>
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<th>Nos.of specs.</th>
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<th>$a_{33}/D$</th>
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<th>$E$(MPa)</th>
<th>$K_{IC}$ (MPam$^{1/2}$)</th>
<th>$K_{IC}^{ini}$ (MPam$^{1/2}$)</th>
<th>$K_{IC}^{un}$ (MPam$^{1/2}$)</th>
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<td>0.353</td>
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Continuation of Table 6

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![Fig. 8 The values of $K_{lc}^{\text{ini}}$ and $K_{lc}^{\text{un}}$ measured from series WS](image)

4 CONCLUSION

For decades, the nonlinearity behavior of $P$-CMOD curve observed in testing three-point beams has been one of the study focuses in the concrete fracture mechanics. Based on the different explanation and hypothesis about this phenomenon, many models depicting the concrete fracture characteristics have been presented. In this report, a detailed comparison is made between the Two Parameter Fracture Model (TPFM), typical in the existing literature, and the double-$K$ fracture criterion proposed in recent years.

There is a growing recognition that the fracture process in the concrete structures consists of three apparent stages: the crack initiation, stable propagation and the unstable propagation. And it is widely accepted that the nonlinearity of $P$-CMOD curve is mainly associated with the FPZ. In the calculation of the effective crack length $a_c$, it should includes both the unrecoverable deformation $CMOD^*$ and one part of the elastic deformation $CMOD^e_n$, the difference between unloading compliance $C_u$ and the initial compliance $C_i$. While in the TPFM, in
order to employ the LEFM, only the latter part, the elastic $CMOD_e$ is taking into account, which will obviously underrate the true value of $a_c$. And in the tests, it is not so easy to control the unloading procedure in the peak load. To equalizing this deficiency, RILEM proposed to adopt the unloading compliance $C_u$ after 95% peak load as one means of compensation. From Table 3 and Table 5, it can be seen that the value of the effective crack length $a_c$ differs very marginally from the double-$K$ fracture criterion. So it can tell that this compensatory method is feasible.

Comparing with the TPFM, the double-$K$ fracture criterion covers more completely in describing the concrete fracture process: in addition to the unstable fracture toughness $K_{IC}^\text{un}$, similar to the $K_{IC}^\text{c}$ in the TPFM, representing the onset of the unstable crack propagation, the initial fracture toughness $K_{IC}^\text{ini}$ is also introduced to describing the commencement of stable crack growth, and they are correlated by the cohesive forces acting on the FPZ.

Besides the more established in the theory concept, the double-$K$ criterion is more practical in the applicability. In the TPFM, to obtain the unloading compliance $C_u$, a closed-loop testing system is required to achieve the stable unloading procedure. It is also shown in this report that the unloading procedure is uneasy to be accessed. Otherwise in the double-$K$ model, for the determining the fracture parameters, such as $a_c$, only a monotonic loading is needed to carried out, without unloading procedure.

From the comparison between the TPFM and the double-$K$ fracture criterion, it can be said that the double-$K$ fracture criterion is more complete in the theory concept, more simple and convenient in the testing method. For most common materials and structural labs, this model is practical.

**ACKNOWLEDGEMENT**

This paper is supported by the National Key Basic Research and Development Program (973 Program) No. 2002CB412709.
REFERENCE


Comparison between the Double-K Fracture Model and the Two Parameter Fracture Model


ABSTRACT

Besides experimental investigations related to the strengthening effects of resins to natural stone, there have been hardly any numerical simulations conducted to the effects of the conservation on the mechanical behaviour of conserved objects. In the present study a three-dimensional finite element code MASA was used to investigate the influence of the conservation procedure on the mechanical properties of the natural stone. The finite element code is based on the microplane material model. As a localization limiter the crack band method was used. A typical profile of sandstone resembling parts of a sculpture - with scaling, sandy decay and sound zones was discretized by a solid finite elements. Varied were material properties, temperature distribution over the depth of the specimen, cyclic effects due to the temperature variation and geometry of the specimen. Numerical results show that as a consequence of change of material properties after conservation procedure the cracks can be generated under environmental conditions that are most likely possible in the
practice. This is especially true for extreme temperature gradients, for repeated temperature conditions (cyclic loading) and for complex geometries. Regular temperature impact, like normal weather cycles, did not cause any risk of high tensions or ruptures. Also continuos material properties were not endangered by high tensile stresses. The numerical results have been partly verified by experiments.

ZUSAMMENFASSUNG


RESUME

A ce jour, très peu de simulations numériques des effets de consolidation des résines artificielles utilisées pour conserver la pierre naturelle ont été réalisées. En général, les études étaient de nature expérimentale. Dans l’étude pré-
sente, le logiciel « MASA » et un maillage tridimensionnel ont été utilisés pour analyser l’influence de la procédure de conservation sur les propriétés physiques de la pierre naturelle. Le code des éléments finis est basé sur le modèle « microplane ». Un segment typique d’une sculpture traitée avec une résine acrylique et contenant zones saines, écaillages, fissures et désagrégation sablonseuse a été discrétisé avec des éléments finis solides. Les propriétés mécaniques du matériau, la distribution de température, les variations de température et la géométrie ont été variées. Les résultats montrent que de fortes inhomogénéités, comme par exemple les fissures remplies de résines, peuvent, sous des conditions climatiques défavorables, être le lieu de pics de tension, voire de nouvelles fissures. Dans des conditions thermiques normales et sans changement abrupt des propriétés mécaniques du matériau, ces pics de tension et l’endommagement en résultant n’ont pas lieu. Une géométrie très filigrane, des gradients de température extrêmes et une résine très rigide sont particulièrement critiques. Les résultats numériques ont partiellement été confirmés par des essais.

**INTRODUCTION**

All masonry materials and especially building stones undergo aging and deterioration processes. A rather new method, which has been in last 20 years used for conservation of precious sculptures and structural parts, is an impregnation procedure by using acrylic resin monomers in special designed treatment equipment, performed by a restoration company. It has saved many distressed and offers the possibility to place them back at their original architectural surrounding. The resin penetrates in the predried objects completely and polymerises in the pore space with only minimal shrinkage. The preservation treatment is very good because the internal stone surface is completely covered by stable resin films. The resin consolidant has been investigated in many ways, however, it was not investigated as a part of a composite which consists of materials that have rather different mechanical and hydro-thermal properties.

In general, the consolidation treatment leads to the following change of properties: increase of tensile strength, increase of elastic modulus, different thermal conductivity and heat capacity, reduction of porosity almost to zero and reduction of moisture expansion properties (swelling).

The problem is that in spite of the increase of the strength after the conservation some cases of damage were reported. Furthermore, the mechanical
change of properties of the consolidated stone were of general interest. Therefore, the numerically investigated influence of the conservation procedure on the structural behaviour of the treated sandstone is reported here. The proposed model used for the finite-element simulations is a typical profile of decaying and scaling sandstone object (see Fig. 1).

Fig. 1 Typical deterioration of sandstones scaling on the surface and beneath it sandy decay

After the treatment with resin this profile of sandstone will, as an extreme case, be a quasi-multilayered structure with different percentages of resin in pore space and will possess different mechanical properties within each layer (4) (see Fig. 2). The plotted line indicates the loss of strength, from very low in the crack zone under the scale to 100 % in the core. The model consists of the surface scale A (weaker than the natural stone but still stable), a layer B which is brittle to sandy decaying but still connected to the scale, the crack zone C with highly increased porosity and up to 100 % resin in cracks. The next zone is the sandy to brittle transition zone D with increasing stiffness and finally the sound core E. The main assumption is that the strength of each layer is related to its porosity.

Initially, the change of the material properties caused by the treatment by artificial resin was investigated. Because of a lack of the characteristic values of a treated sandstone at different porosities a simple model to estimate these properties of the composite (sandstone – artificial resin) was proposed. This model, being closely related to homogenisation concepts in continuum mechanics, uses the material properties of the components and their volume fraction to estimate
the effective (macroscopic) properties of the composite. For the sandstone we used the technical properties of the “Schilfsandstone” a common Keuper fine sandstone from Baden-Württemberg consisting of a large portion of a rock fragments besides quartz, feldspar, cemented by feldspar and chlorite cements with an average porosity of 17 % by volume (3).

![Image](image_url)

**Figure 2** The distribution quantities of resin (percentage) in the pore space within the former deteriorated scaling zone. Profile from the outside weathering zone (left) to the sound core (right). Plotted line indicates the loss of strength because of weathering.

We calculated the physical properties by averaging the Reuss- and Voigt-approaches (1). In the one-dimensional case these two approximations can simply be imagined to be as shown in Fig. 3 and Fig. 4.

![Image](image_url)

**Figure 3** Homogenisation model of the material - Reuss-approximation (The stresses over the cross-section of the two materials are equal and constant).
Figure 4 Homogenisation model of the material - Voigt-approximation (The strains over the cross-section of the two materials are equal and constant).

It has been demonstrated that these values represent the real lower and upper bounds of the elastic modulus of composite materials with elastic behaviour. The homogenisation procedure is schematically shown in Fig. 5.

Figure 5 Effective macroscopic properties of composite material.

The physical properties from our model were partly verified by experiments and partly by the values from the literature or former investigations. Examples for the approximation of the tensile strength, the elastic modulus and the coefficient of thermal expansion are shown in Figs. 6, 7 and 8. After the initial calculation, the experimental data were used to adjust the starting values for the final calculation.
Finite Element Modelling of the conservation effects of an artificial resin on deteriorated heterogenous sandstone in building restoration

**Figure 6** Homogenisation of the compound materials theoretic approaches and experimental verification - the dependence between tensile strength and percentage of resin.

**Figure 7** Homogenisation of the compound materials theoretic approaches and experimental verification - the dependence between elastic modulus and percentage of resin.
Figure 8  Homogenisation of the compound materials theoretic approaches and experimental verification - the dependence between coefficient of thermal expansion percentage of resin.

Summary of the calculated material properties for different porosities is given in Table 1.

Table 1  Calculated material properties for different porosities according to the homogenisation theories.

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<th>volume fraction of resin $\varphi_{resin}$ [%]</th>
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<td>tensile strength $\beta_Z$ [N/mm$^2$]</td>
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The results shown in the Figs. 6-8 were experimentally verified on 3-4 impregnated samples of the Schilfsandstone. The experimental results show good agreement with the calculated values of the homogenisation theories. The elastic modulus has also been measured (see Fig. 7) and it shows relatively good agreement with the predicted values. The experimental verification is also shown in Fig. 9a to 9c (3). The impregnated material reveals a much higher modulus of elasticity and a brittle fracture at much higher tensile stresses.
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EXPERIMENTAL VERIFICATION

Figure 9a  Stress-displacement-curve of three different samples of sound Schilfsandstone (untreated). Results of a centric tension test of prismatic samples with a diameter of 5 cm and length of 15 cm.

Figure 9b  Stress-displacement-curve of two different samples of sound Schilfsandstone completely impregnated by resin. Results of a centric tension test of prismatic specimen with a diameter of 5 cm and length of 15 cm.
Figure 9c: Comparison of stress-displacement-curves of two different samples of Schilfsandstone specimen g is completely impregnated by resin and specimen u untreated. Results of a centric tension test of prismatic specimen with a diameter of 5 cm and length of 15 cm.

MODELLING

The proposed geometry of the model and boundary conditions were chosen such that they resemble parts of a sculpture like an arm of a statue or a detail of a balustrade (see Fig. 10). For the FE-simulations the model was discretized by four-node 3D elements, whereas different material properties were discretized by the layers of finite elements. Figure 10 shows the FE-model, a section of a semi-cylinder, exploiting the symmetry conditions. The marked layer is the intermediate layer (layer C in fig. 2) consisting of almost 100% resin (compare Fig. 2). From the mechanical point of view in the initial state of modelling each layer of the material is assumed to be isotropic.
The model of layers, as an approximation of the consolidated sandstone, consists of the layers like proposed in Fig. 2. It consists of the surface scale (weaker than the natural stone but still stable), a layer which is brittle to sandy decay but still connected to the scale, a crack zone, a second sandy decay zone with increasing stiffness and a gradual change towards sound stone, called zones A to D. It has to be pointed out, that after impregnation the strength of the profile is completely changed, because of the rather high strength of the resin and the variation of the percentage of resin.

NUMERICAL ANALYSIS

The three-dimensional FE-analysis consists of two parts: calculation of thermal loading and calculation of stresses and strains.

Because of the treated sandstone being fully saturated with resin there are no more chemical or moisture influences, e.g. like moisture expansions. The eigenstresses due to the polymerisation and shrinkage of the resin were not considered because they degrade with time (relaxation). Besides external causes like dead or working loads (as normally also regarded in structural analysis) have been also neglected in the present study.

THERMAL LOADING

The following climatic effects were taken into account. Time dependent thermal influences (air temperature, direct and indirect solar radiation, precipitation) lead to the different transient temperature distributions. They were calculated using the finite-element-method by approximating the solution by the
weak form of the Fourier differential-equation with different boundary conditions. The applied numerical solution is based on the Crank-Nicolson-method (8), (9).

From physics-of-construction references, 8 cases of extreme climate situations sometimes occurring in Germany were chosen for the modelling of extreme temperature load (7). Initially, it has been shown that the regular temperature variations of the climate have no critical impact. However, there are several critical temperature loads. Besides the loading cases ‘‘warming-up’’ and ‘‘cooling down’’ in summer and winter, respectively, the cases ‘‘thunder shower in summer’’ – which simulates (cold) rainfall on a warmed surface were also considered. They leads to extreme temperature distributions. Figures 11 and 12 show this extreme temperature situation which were calculated for two different specimen profiles.

![Diagram](image)

*Figure 11 Temperature distribution caused by “thunder-shower”.*
Finite Element Modelling of the conservation effects of an artificial resin on deteriorated heterogenous sandstone in building restoration

![Figure 12: Temperature distribution caused by “thunder-shower” with indirect radiation from the right side and direct and indirect radiation from the left side.]

It can be seen that because of the multilayered structure and the low thermal conductivity of the intermediate resin layer an extreme gradient occurs. In comparison to the homogeneous profile (no layers of different materials) the temperature on the surface of the multilayered geometry is much lower.

**CALCULATION OF STRESS AND STRAIN DISTRIBUTION**

The finite element code MASA (5) employed in the present study can be used for the non-linear finite element (FE) analysis of quasi-brittle (concrete-like) materials. It is based on the microplane material model and the smeared crack concept. As a regularization procedure the crack band approach is used (2).

To calculate distribution of stresses and strains one needs the material constitutive law. Here, the microplane model is used. The microplane model (6) is characterized by a relation between the stress and strain components on planes of various orientations. These planes may be imagined to represent the damage planes or weak planes in the microstructure, such as contact layers between aggregates in concrete. In the model the tensorial invariance restrictions need not to be directly enforced. Superimposing the responses from all microplanes in a suitable manner automatically satisfies them.
The recently proposed version of the microplane model for "concrete materials" is based on the so-called relaxed kinematic constraint concept (6). In the model the microplane (see Fig. 13) is defined by its unit normal vector of components \( n_i \). Normal and shear stress and strain components \( (\sigma_N, \sigma_{Tr}, \varepsilon_N, \varepsilon_{Tr}) \) are considered on each plane. Microplane strains are assumed to be the projections of the macroscopic strain tensor \( \varepsilon_{ij} \) (kinematic constraint). Based on the virtual work approach, the macroscopic stress tensor is obtained as an integral over all possible, in predefined, microplane orientations (\( \Omega \) denotes the surface of the unit sphere):

\[
\sigma_{ij} = \frac{3}{2\pi \Omega} \int_{\Omega} \sigma_N n_i n_j \, d\Omega + \frac{3}{2\pi \Omega} \int_{\Omega} \sigma_{Tr} (n_i \delta_{ij} + n_j \delta_{ii}) \, d\Omega
\]  

To realistically model quasi-brittle materials, the normal microplane stress and strain components have to be decomposed into the volumetric and deviatoric parts \( (\sigma_N = \sigma_V + \sigma_D, \varepsilon_N = \varepsilon_V + \varepsilon_D) \); see Figure 13), what leads to the following expression for the macroscopic stress tensor:

\[
\sigma_{ij} = \frac{3}{2\pi \Omega} \int_{\Omega} \sigma_N n_i n_j \, d\Omega + \frac{3}{2\pi \Omega} \int_{\Omega} \sigma_{Tr} (n_i \delta_{ij} + n_j \delta_{ii}) \, d\Omega
\]
\[
\sigma_{ij} = \sigma_v \delta_{ij} + \frac{3}{2\pi \Omega} \int \sigma_n n_i n_j \, d\Omega + \frac{3}{2\pi \Omega} \int \sigma_{tr} (n_i \delta_{ij} + n_j \delta_{ni}) \, d\Omega
\]  

For each microplane component, the uniaxial stress-strain relations are assumed as:

\[
\sigma_v = F_v(\varepsilon_{v,\text{eff}}) \quad ; \quad \sigma_D = F_D(\varepsilon_{D,\text{eff}}) \quad ; \quad \sigma_{Tr} = F_{Tr}(\varepsilon_{Tr,\text{eff}})
\]

where \( F_v, F_D \) and \( F_{Tr} \) are the uniaxial stress-strain relationships for volumetric, deviatoric and shear components, respectively. From known macroscopic strain tensor, the microplane strains are calculated based on the kinematic constraint approach. Finally, the macroscopic stress tensor is obtained from (2). The integration over all microplane directions (21 directions) is performed numerically.

The basic mechanical properties that are needed for the non-linear smeared fracture finite element analysis are Young’s modulus, Poisson’s ratio, uniaxial tensile and compressive strength and fracture energy. For each layer of the material these properties are obtained from the literature or from the experiments. In our case it was calculated as shown above (see Tab. 1) and verified by experiments.

The results of the stress and strain calculation show that extreme loads can cause stresses in the range of the tensile strength. For the extreme loading case, i.e. the “thundershower”, the stress and strain distributions are plotted in Fig. 14. Figure 15 shows the stress distribution over the depth of the FE-model.
Figure 14  Calculated distribution of stresses and strains for the temperature distribution caused by “thunder-shower” (mainly in tangential direction).

Figure 15  Calculated distribution of stresses at a chosen section over the depth of the model for the temperature distribution caused by “thunder-shower” (blue = layer consists of 100% resin).
The zone marked blue is the intermediate layer C consisting of 100% resin. The plot of the total strains shows that there is a highly expanding layer in radial direction. This occurs in the intermediate resin layer C. The corresponding stresses for this loading case are rather high, especially on the surface. This is a consequence of the incompatibility between the thermal expansions of the different layers next to the surface.

In order to investigate the consequence of several reference temperatures (that is the temperature of stress-free state of the material), initially for the calculations the reference temperature was set to $T=25^\circ C$. In additional calculations this temperature was varied. The resulting plots shown in Fig. 16 demonstrate that the influence of the reference temperature can in this special case be neglected with sufficient accuracy.

Figure 16 Influence of the reference temperature on the distribution of stresses ("thunder-shower" effect at different starting temperatures of the surface, blue = layer C consisting of 100% resin).
In another study the influence of the geometry of the specimen was investigated. The stress and strain distributions (see Fig. 17) show, that for the complex geometries (e.g. pleats or parts of the body of a sculpture) the same loading case as used before –“thunder shower”– can even cause cracks (red zones – principal strains). It can be seen that a curvatures at the surface of the specimen can lead to larger stress concentrations and therefore to crack development as well. Note that the cracks are not predefined, i.e. they occur automatically as a consequence of interaction between the thermal and the mechanical model.

In an additional study it was found that the repeated temperature conditions (cyclic temperature loading) can cause cracking even for not extreme temperature conditions and simpler geometries.

**CONCLUSIONS**

The finite element analysis based on the proposed multi-layered model indicates that large differences in the elastic properties of layers and large differences in coefficient of thermal expansion of resin-treated structure can under certain conditions cause damage (cracking). It is demonstrated that under normal climatical conditions no cracking occurs – the stresses are smaller than the material strength. However, depending on the distribution of temperature and on the geometry, in extreme cases cracking is possible. For resin-treated objects there is a danger of damage, especially if they have a complex geometries with high expanding intermediate layers of almost pure resin, like in our model the layer C. The study confirms that the finite element analysis is a useful tool to investigate the influence of the thermo-mechanical interaction on the structural response and to show the critical impacts, critical heterogenities and resulting main stresses and cracks. Moreover, the analysis can be used to select optimal material properties for conservation treatments. The results have been confirmed by experiments (see Fig 9a-c), however, further experimental and theoretical work is needed.
Finite Element Modelling of the conservation effects of an artificial resin on deteriorated heterogenous sandstone in building restoration

Figure 17 Calculated distribution of stresses and strains for the temperature distribution caused by “thunder-shower” – structure with not constant curvature, symbolising parts of a sculpture.

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SUMMARY

Several methods existing for testing the flowability of self-compacting concrete (SCC). In this article a simple method – based on the so-called J-Ring test – is presented which allows the quantification of the part of the blocked concrete volume. Furthermore some empirical relationships between different test results are presented which were found for the tested SCC mixtures.

ZUSAMMENFASSUNG

Es gibt eine Reihe von verschiedenen Prüfmethoden zur Bewertung der Fließfähigkeit von Selbstverdichtendem Beton (SVB). In diesem Artikel wird eine einfache Methode zur Quantifizierung des blockierten Betonvolumens vorgestellt, die auf dem sogenannten J-Ring Versuch beruht. Des Weiteren werden empirische Zusammenhänge zwischen den einzelnen Messwerten angegeben, die für die untersuchten SVB-Mischungen gefunden wurden.

RESUME

Il existe une série de différentes méthodes de contrôle pour évaluer la fluidité du béton autoplaçant (BAP). Dans l'article présent, une méthode simple pour quantifier le volume de béton bloqué est présentée Cette méthode est basée sur le test « J-Ring ». Des relations empiriques entre les valeurs mesurées obtenus pour les différents BAP examinés sont également présentées.

KEYWORDS: Self-compacting concrete, SCC, blocking, J-Ring, blocking ratio, step of blocking
1. INTRODUCTION

Self-compacting concrete (SCC) was developed in the middle of the 1980’s in Japan. SCC flows alone under its dead weight up to levelling, airs out and consolidates itself thereby without any entry of additional compaction energy and without a nameable segregation. SCC owns over three key characteristics which are shown in fig. 1. These characteristics were made possible by the development of highly effective water reducing agents (superpalsticizers), those usually based on polycarboxylate ethers. The mixture composition of SCC deviates from conventional concrete. The powder contents of SCC are normally lying (in some cases even considerably) above those of conventional concrete.

1. **Filling Ability:** Ability of to fill a formwork completely under its own weight.

2. **Passing Ability:** Ability to overcome obstacles under its own weight without hindrance. Obstacles are e.g. reinforcement and small openings etc.

3. **Segregation resistance:** Homogeneous composition of concrete during and after the process of transport and placing.

*Fig. 1: The three key properties of fresh self-compacting concrete*

Because of its special fluidity, SCC requires modified fresh concrete testing methods compared with conventional concrete. These testing methods are specified e.g. in [1] and [2]. The difficulty consists of the fact, that SCC responds very sensible to deviations of mixture proportions. Already slightest deviations can lead to a concrete that does not obtain one ore more of these key characteristics. This is usually connected with substantial lack of the finished construction unit, which lower not least the durability drastically and make in the worst case a construction useless. In the following the filling and the passing ability will be considered in a greater detail.
2. QUANTIFICATION OF THE PASSING ABILITY

It was already mentioned that SCC has to have the ability to flow through narrow openings without hindrance. This means that the so-called blocking of coarse aggregates through bridging has to be avoided. Fig. 2 shows the mechanism of blocking of coarse aggregate by a two-dimensionally illustrative model.

![Mechanism of blocking](image)

Fig. 2: Mechanism of blocking [3]

One possibility for assessment of the blocking behaviour is to perform the well-known slump flow test under use of the so-called J-Ring (Fig. 3).

![Necessary equipment for performing the J-Ring test](image)

Fig. 3: Necessary equipment for performing the J-Ring test [5] (schematic, just before setting the Abram’s cone on the worktop)

After the lifting motion of the Abrams’ cone the concrete must flow under its dead weight through the steel rods which shall simulate the flow through reinforcement in a real formwork. Fig. 4 shows a photography of the concrete shape which arose after the J-Ring test was performed. Dependent on the mixture composition (and also on the number of steel rods) a clearly increased content of coarse aggregates can be observed within the ring despite reaching a defined spread diameter. Apparently blocking can be determined. A second pa-
rameter (beside the spread diameter $s_J$ and the flow time $t_{500,J}$) can be quoted by measuring the height difference of the concrete behind and in front of the rods of the J-Ring. A mean value can then be calculated of four measuring points (two directions each with two measuring points). This mean value is defined as the step of blocking $s_{J}$. The method of measuring the step $s_{J}$ was also stated in [6].

![Image of J-Ring with 16 rods and measurement setup]

*Fig. 4: Photography of the measurement of the step (J-Ring with 16 rods, $s_{J} \approx 30$ mm) and idealised spread of concrete for calculation of the blocking index [4].*

From the measured step $s_{J}$ the portion of the blocked concrete volume can be derived (Fig. 4). Thus a blocking index $\beta$ as the ratio of the blocked concrete volume $V_{block}$ related to the total concrete volume $V_C$ can be calculated (eq. 1). The part of the blocked concrete volume $\beta$ is then directly proportional to the step $s_{J}$. It has to be annotated that the concrete shape isn’t exactly plane. It is rather curved due to the concrete yield value and the stresses caused from dead weight. This is the reason why a step $s_{J}$ can be measured despite apparently blocking is not determined. But for a first simplification the concrete shape can considered to be plane.

$$\beta = \frac{V_{block}}{V_C} = \frac{\pi D^2}{4} \cdot \frac{s_{J}}{V_C} = \frac{\pi D^2}{4} \cdot s_{J}$$  \hspace{1cm} (eq. 1)

with:

- $\beta$ blocking index
- $V_C$ whole concrete volume (volume of the Abram’s cone)
- $V_{block}$ blocked concrete volume
- $s_{J}$ step of blocking, mean value measured in two directions each with two measuring points at the ends
- $D$ diameter of the idealised concrete shape
3. TEST RESULTS

A set of fresh concrete investigations were accomplished in the context of an European GROWTH project called „Testing SCC” (GRD2-2000-30024) which served to investigate methods for quantification of the blocking behaviour of SCC. Therefore the filling and passing ability of fresh concrete SCC mixtures (contained crushed granite as coarse aggregates) have been assessed. The spread diameters and the flow times of the slump flow test (s, t_{500}) respective the J-Ring test (s_J, t_{500,J}), the step of blocking s_J and the flow time t_V of the so-called V-funnel test were measured.

SCC mixtures are often characterized by their funnel time t_V (which is often used as a degree of the apparent viscosity of mix) and their spread diameter s which stands for the filling ability. Fig. 5 gives an overview over the characteristic fresh concrete parameters of the tested mixtures; the results are ordered by the step s_J. Apparently no blocking could be observed if the step s_J was below 10 mm.

![Graph showing properties of tested SCC mixtures](image)

Fig. 5: Properties of tested SCC mixtures

It can be annotated to Fig. 5 that the blocking behaviour varies from non blocking to strong blocking within the range of 650 < s < 770 mm.
Fig. 6 shows the interrelation between the spread $s_j$ and $s$. The difference $\Delta s = s - s_j$ between the two diameters decreases with an increasing filling ability. However there exists a large scatter (Fig. 7).

![Graph showing the relationship between spread $s$ and $s_j$](image1)

*Fig. 6: Spread $s_j$ (J-Ring) vs. spread $s$ (slump flow)*

![Graph showing the relationship between deviation $\Delta s$ and spread $s$](image2)

*Fig. 7: Difference $\Delta s$ vs. spread $s$*

It was shown in the previous chapter that the part of the blocked concrete volume $V_{\text{block}}$ is proportional to the step $s_j$. It might be therefore of interest if there exists a relationship between the difference $\Delta s$ and the step $s_j$ (Fig. 8).
Even the difference $\Delta s$ is limited to 30 mm, the step $s_J$ scatters between 8 mm (no blocking) and 65 mm (strong blocking). It can be derived from Fig. 7 and Fig. 8 that a $\Delta s$-criterion (alone) is not suitable for quantification and evaluation of the blocking behaviour of a SCC mixture.

In the J-Ring test and also in the V-funnel test the concrete has to overcome a narrow opening obstacle. It can therefore assumed that there also exists a relationship between the measured parameters of these two tests. Fig. 9 contains the step $s_J$ and the flow time $t_V$.

Fig. 8: *Step $s_J$ vs. difference of spread $\Delta s$*

Fig. 9: *Relationship between the blocking behaviour and funnel time*
The funnel time $t_V$ is often used to estimate the apparent viscosity of a mixture. However, many factors are playing a role and influencing the result of the V-funnel test: the amount, shape and size distribution of aggregates and also the viscosity and amount of paste etc. This means that the funnel time does not necessarily correspond with the viscosity of a mix measured e.g. by a rheometer.

There was also found an empirical relationship between the spread $s_J$ and the step $s_J$ (Fig. 10). The relatively good stability index of $R^2 = 0.95$ can attributed to the fact that a geometric relationship exists among these parameters (the volume of the Abrams’ cone is fixed of about 5.5 litres).

\begin{equation}
y = -0.169x + 131.9
\end{equation}

\[ R^2 = 0.95 \]

*Fig. 10: Step $s_{J}$ vs. spread $s_J$*

It was already shown that the blocked concrete volume is linked with the funnel flow time $t_V$. But also the segregation resistance is linked with the apparent viscosity of a SCC mixture. To achieve a sufficient resistance to segregation, the funnel time may not fall below a minimum value. From practical sight of view it would be convenient to know a reliable relationship between the flow times $t_{500}$ respective $t_{500,J}$ and the funnel time $t_V$ of a SCC mixture. Then the V-funnel test could be skipped. The relationships which were found between the flow times are plotted in Fig. 11 and Fig. 12. The correlation between the $t_{500}$-value and the $t_V$-value is for the slump flow test better than for the J-Ring test.
However, the measurement of the $t_{500}$-value is more operator influenced than the measurement of the V-funnel flow time $t_V$.

**Fig. 11:** Relationship between the $t_{500}$-value (slump flow test) and the funnel time

**Fig. 12:** Relationship between the $t_{500J}$-value (J-Ring test) and the funnel time
4. SUMMARY

Several methods existing for testing the filling and passing ability of SCC. In this article a simple method was presented which allows the quantification of the part of the blocked concrete volume with the so-called J-Ring test. It could be shown that the blocked concrete volume is proportional to the step of blocking which adjusts itself directly in front and behind the steel rods of J-Ring. This method was compared with the conventional method which evaluates the blocking behaviour of SCC by the difference of spread between the slump flow test and the J-Ring test. It could be derived that the conventional method is not suitable to quantify the blocking behaviour. Also some empirical relationships between different test results were presented which had been found for the tested SCC mixtures.

5. REFERENCES


NAILS AND NAILPLATES AS SHEAR CONNECTORS FOR TIMBER-CONCRETE COMPOSITE CONSTRUCTIONS

NÄGEL UND NAGELPLATTEN ALS SCHUBVERBINDNER FÜR HOLZ-BETON-VERBUNDKONSTRUKTIONEN

CLOUS ET CONNECTEURS COMME ASSEMBLAGES DE CISAILLEMENT POUR LES STRUCTURES COMPOSITES BOIS-BÉTON

Simon Aicher, Wolfgang Klöck, Gerhard Dill-Langer, Borimir Radovic

SUMMARY

In many European countries an increasing trend for use of timber-concrete composite constructions cannot be overseen. The range of applications comprises upgrading and post-strengthening of existing timber floors in residential / office buildings as well as newly erected constructions for buildings and bridges. Several Technical Approvals have been issued by German Institute for Building Technique (DIBt) for such constructions in recent years.

The composite action is highly dependant on the type of employed shear connectors which also determine considerably the economical aspects. A large variety of mechanical or glued connectors has been investigated and / or used, some of them being unmodified timber-timber connectors and others having been specially developed for timber-concrete compounds.

This paper reports on mechanical properties of nails and nailplates representing probably the most basic shear connectors for timber-concrete constructions. Both types of connectors are used in two timber-concrete systems covered by Technical Approvals issued with the involvement of Otto-Graf-Institute. In detail, medium-sized smooth nails and small threaded nails, and three nailplate types with different application methods are regarded. Besides the connectors some details of the relevant timber-concrete constructions are given. An emphasis is laid on the explanation of systematic differences of slip modulus and shear strength of the connectors when used either in timber-timber, timber-steel plate or in timber-concrete connections.
ZUSAMMENFASSUNG


Das Verbundverhalten hängt hauptsächlich von der Art der Schubverbinder ab, die andererseits auch die ökonomische Rentabilität der Bauweise stark beeinflussen. Es wurde bereits eine größere Anzahl verschiedener nachgiebiger und geklebter Schubverbinder untersucht und / oder in der Baupraxis eingesetzt; einige von ihnen waren unverändert übernommene Holz-Holz Verbindungsmittel, andere wurden speziell für den Einsatz als Holz-Beton-Verbinder konzipiert.

RESUME

Dans de nombreux pays européens, la tendance croissante à l’utilisation de structures composites bois-béton ne peut pas être overseen (ignorée ?). Les applications de telles structures se situent au niveau de l’amélioration et du renforcement de planchers bois dans des constructions résidentielles ou des bâtiments publics, comme dans la réalisation de constructions neuves, bâtiments ou ponts. Plusieurs avis techniques ont été délivrés par l’Institut Allemand des Techniques de Construction (DIBt) pour différentes constructions au cours des années récentes. L’action du composite dépend fortement du type de connecteur utilisé pour la reprise des contraintes de cisaillement, ce qui a par ailleurs un impact économique considérable. Une grande variété d’assemblages mécaniques ou collés a été étudiée et/ou utilisée, certains étant des assemblages de type bois-bois non modifiés, d’autres ayant été spécifiquement développés pour les composites bois-béton.


KEYWORDS: timber-concrete composite constructions, shear connectors, smooth and threaded nails, nailplates, slip modulus, shear capacity, Technical Approval for timber-concrete constructions
1. INTRODUCTION

Timber-concrete constructions, although not that widely recognized, have a rather long tradition with prevalingly good experience especially for upgrading of (old) pure timber ceilings. Today, in Europe an increasing trend for use of this composite construction method for newly erected buildings can not be overseen, too. Perceivable and actually employed connectors comprise a wide variety of mechanical or glued connectors (Fig. 1). Some of them are unmodified pure timber-timber construction connectors and others have been specially adapted to or were developed for timber-concrete compounds.

In an attempt to categorize the different connectors quantitatively in terms of stiffness, strength and application features this paper, in a first step, reports on the probably most basic connectors, such as nails and nailplates, adopted (almost) unchanged from pure timber-timber applications. Altogether with the connectors some specifically related timber-concrete constructions, some of them recently approved by German Building authority (DIBt), are discussed.

![Diagram of timber-concrete connection systems](image)

*Fig. 1: Examples of timber-concrete connection systems from [1]*
2. NAILS AS TIMBER CONNECTORS

2.1 Medium-sized smooth nails

2.1.1 Timber-concrete joint

Smooth steel nails represent certainly the most basic and most extensively employed shear connector alternative for timber-concrete composites. Especially in former Czechoslovakia several 10 000 m$^2$ of originally pure solid timber beam ceilings have been post-strengthened with an additional concrete slab via smooth steel nails in the last four decades since its first reported application in 1960 [2, 3].

In the upgrading procedure of timber beam ceilings (Fig. 2a, b), followed up in Czechoslovakia, almost throughout smooth nails with dimensions of $6.3 \times 180$ mm were / are used. Depending on the type of construction, either with or without lost sheeting, the anchorage length of the nails in the timber varies between 120 to 140 mm and is about 40 mm in the concrete (C20) which is reinforced with one steel mat (Q 131). The nails are driven into partly (about 50% of anchorage length) predrilled holes.

![Fig. 2a, b: Timber-concrete connection with medium sized smooth nails after [3]](image)
Some results of the mechanical behavior of the specific timber-concrete connector test configuration, shown in Fig. 3, are reported by [3]. The compression shear tests gave, per nail, a mean shear load capacity and an initial slip modulus of

\[ R_{\text{mean}} = 4.5 \text{ kN} \quad \text{and} \quad K_{\text{ser}} = 30 \text{ kN/cm}. \]

As no coefficient of variation of the test results is specified, a C.O.V. of about 10 - 15 % for ultimate load is assumed, resulting in a characteristic (5 percentile) load capacity estimate in the range of \( R_k = 3.4 - 3.8 \text{ kN} \). The strongly non-linear range of the connection starts at about 1/3 of ultimate load capacity.

![Fig. 3: Timber-concrete composite compression shear test specimen used for smooth nail connections [3]](image-url)
2.1.2 Comparison with analogous timber-steel plate connection

It is of interest how the stiffness and strength values obtained for the timber-concrete connection are related to (mean) slip modulus and characteristic load capacity of a nailed timber-steel plate connection (thick steel plate) in single shear. Obviously, the latter material/geometry configuration should forward a similar load capacity as the failure mechanisms resemble each other closely. In both cases the steel nail shows a plastic hinge at the interface of timber to steel or concrete and a second one inside the timber (Fig. 4a). According to German draft timber design code E DIN 1052:2000 [4] the characteristic shear load capacity of a nailed timber-thick steel plate connection is (d = nail diameter)

\[ R_k = R_{k,\text{smooth nail}} = 2 \sqrt{M_{y,k} \cdot f_{h,k} \cdot d} \]  

where the characteristic values of the nail yield moment \( M_{y,k} \) (steel tensile strength \( \geq 600 \text{ N/mm}^2 \)) and of the embedment strength, \( f_{h,k} \), are

\[ M_{y,k} = 180 \cdot d^{2.6} \]  

(d in mm) \hspace{1cm} (2)

\[ f_{h,k} = 0.082 \cdot \rho_k \cdot d^{-0.3} \]  

(hole not predrilled) \hspace{1cm} (3a)

\[ f_{h,k} = 0.082 \cdot \rho_k \cdot (1 - 0.01 \cdot d) \]  

(hole predrilled) \hspace{1cm} (3b)

and \( \rho_k \) is the characteristic density of the timber in kg/m\(^3\).

In the given case for comparison with above stated test results, where \( d = 6.3 \text{ mm} \) and \( \rho_k \leq 350 \text{ kg/m}^3 \) (i.e. timber strength class roughly C 24) one obtains with \( f_{h,k} = 21.7 \text{ N/mm}^2 \) (average between predrilled and not predrilled hole) and \( M_{y,k} = 21.56 \times 10^3 \text{ Nmm} \) a characteristic shear resistance of \( R_k = 3.4 \text{ kN} \). This value corresponds very well with the above specified range of the characteristic load capacities of 3.4 – 3.8 kN for the timber-concrete connection.

Regarding slip modulus, E DIN 1052:2000 specifies for nails in predrilled and not predrilled holes, respectively, for timber-timber as well as for timber-steel connections at single shear condition

\[ K_{ser} = \rho_k^{1.5} \cdot d / 20 \]  

and \hspace{1cm} \[ K_{ser} = \rho_k^{1.5} \cdot d^{0.8} / 25 \]  

(4a, b)

The average slip modulus for predrilled and not predrilled holes (\( d = 6.3 \text{ mm}, \rho_k = 350 \text{ kg/m}^3 \)) then follows as \( K_{ser} = (20.6 + 11.4) / 2 = 16 \)
kN/cm. When comparing this value to the above given experimental timber-concrete connection result of 30 kN/cm it has to be stated that the $K_{ser}$ design value for the analogous timber-steel connection (here: timber-timber) is only about one half. The reason for this is, that the $K_{ser}$ design equation for a timber-steel connection, i.a. due to the generally oversized (1 mm) hole in the steel plate, does not consider any stiffness increase vs. pure timber-timber connections.

Comparing the deformation shapes of the nail in the timber-concrete and the timber-timber connection situation (with sufficiently thick members), see Figs. 4a, b, however, it is obvious that $K_{ser}$ in the timber-concrete application should be very roughly 2times higher. This actually is fully supported by the above specified $K_{ser}$ values.

![Fig. 4a, b: Shape of deformed dowel type fasteners in a concrete-timber (a) and a pure timber-timber joint (b)](image)

2.2 Small-sized threaded nails

2.2.1 Timber-concrete joint

In 1998 the first two German Technical Approvals for timber-concrete constructions and their respective connectors were issued by German Institute for Building Technique (DIBt). One of them, not regarded in this context, deals with special screws (Z-9.1-342 [5]) and the other (Z-9.1-331 [6]), considered
here, is based on the application of usual small-sized threaded (ring shank) nails with dimensions $3.4 \times 60$ mm (Fig. 5a). The electro-galvanically corrosion protected nail (company Paslode) must conform, proven by certificate KA 028 [7], to load bearing (axial withdrawal) class III.

The nail is employed in a prefabricated timber beam-concrete slab construction as shown in Fig. 5b. The gun-shot nails must be arranged in two rows along the small edge of the solid wood beams (minimally conforming to strength class C24) with cross-sectional dimensions (width×depth) of 45-72 mm × 170-250 mm. The thickness of the concrete slab (C35) can vary from 50-85 mm. The anchorage length of the nail in the concrete must be 20-25 mm and correspondingly 35-40 mm in the timber. Prefabricated elements of the described type with maximum dimensions of $3 \times 7$ m were / are successfully employed in buildings in Sweden and Germany.

![Fig. 5a, b: Timber-concrete connection system with small sized threaded nails for the EW element acc. to Z-9.1-331 [6]](image_url)

*a) fastener     b) lay-up and dimensions of the timber concrete construction*
According to the Technical Approval [6], which is based on extensive tests at Swedish National Testing and Research Institute and test evaluations/expertise at Otto-Graf-Institute, the characteristic and allowable shear capacity and the slip modulus shall be assumed, per nail, as
$$R_k = 1.2 \text{kN}, \quad R_{\text{allow}} = 0.5 \text{kN} \quad \text{and} \quad K_{\text{ser}} = 12 \text{kN/cm}.$$  

2.2.2 Comparison with analogous timber-steel plate connection

For assessment of the specified load capacities and slip property an analogous timber-steel plate connection (thick steel plate, single shear) is regarded. The characteristic shear capacity of a threaded nail (load capacity class III acc. to E DIN 1052:2000 [5]) is

$$R_k = R_{k,\text{smooth\_nail}} + \Delta R_k$$  

$$\Delta R_k = \min(0.5 \cdot R_{k,\text{smooth\_nail}}; 0.25 \cdot R_{\text{ax},k})$$

with $R_{k,\text{smooth\_nail}}$ acc. to Eq. (1) and

$$R_{\text{ax},k} = f_{1,k} \cdot d \cdot l_{ef} \quad \text{with} \quad f_{1,k} = 50 \cdot 10^{-6} \rho_k^2$$

The additional load capacity term $\Delta R_k$, as compared to smooth nails, accounts for the tension force activation in the bent nail resulting from the grip/friction of the profiled nail surface in the timber. Evaluating the shear capacity with $d = 3.4 \text{ mm}$, $\rho_k = 350 \text{ kg/m}^3$ and $l_{ef} = 35 \text{ mm}$ (embedment length of the nail in the timber), which results in $f_{1,k} = 6.1 \text{ N/mm}^2$, we obtain

$$R_{k,\text{smooth\_nail}} = 1.08 \text{kN}, \quad R_{\text{ax},k} = 0.73 \text{kN}$$

and then $R_k = 1.08 \text{kN} + 0.18 \text{kN} = 1.26 \text{kN}$.

As anticipated, the calculated timber-steel plate shear capacity is very close to the experimentally based value specified above for the timber-concrete connection (difference $= 6\%$).

Regarding the slip modulus of a threaded nail in a timber-steel plate connection the same equation (4b) as for smooth nails in not predrilled nail holes applies, giving $K_{\text{ser}} = 7 \text{kN/cm}$. So, very similar to the situation with the medium-sized smooth nail, the calculated slip modulus for the timber-steel plate connection is only about 60% of the stiffness of timber-concrete connection.
3. NAILPLATES

3.1 Timber-concrete joints

The first thorough investigations on the use of punched metal plate fasteners (short: nailplates) as timber-concrete connectors in timber-concrete composites used for walls and floors were reported by Girhammar [8, 9].

A later extensive research work on nailplates as timber-concrete connectors was performed at University of Karlsruhe [10, 11, 12]. In the year 2003 the first German Technical Approval for a timber-concrete construction based on nailplates was issued (Z-9.1-474 [13]); the respective tests and expertises were done at Otto-Graf-Institute. In the following it is focussed on the nailplate constructions investigated in Karlsruhe and in Stuttgart. Figures 6 and 7 show the specific connector applications used.

![Diagram of Nailplate Connection](image)

*Fig. 6: Nailplate connection investigated in [10]. The figure shows the shape / dimensions of the investigated compression shear specimens*

The connectors investigated in Karlsruhe were nailplates bent at mid-width, as shown in Fig. 6. One half of the plate with not removed nails is pressed into the narrow timber beam face, whereas the other half with removed nails is embedded in the concrete.
Two test series (A-NAG and B-NAG) with two different nailplates of equal width and length dimensions of $114 \times 266$ mm were performed. In the first test series, A-NAG, with 5 specimens, the nailplate type "Gang Nail GN 200" acc. to Z-9.1-230: 1966 [14] was used. In the second test series, B-NAG, with a considerably higher specimen number of 46, nailplate type "Merk nailplate MNP-A" acc. to Z-9.1-273 [15] was employed. Figure 8 shows a view of the latter nailplate type MNP-A with nail length, width and thickness of 20 mm, 3.2 mm and 2 mm, respectively.

The connection configuration tested in Stuttgart [13], resembles closely the nailplate joints used by Girhammar [8, 9]. In both cases, different from the Karlsruhe approach, unmodified nailplates (not bent, no removed nails) are used. In detail, the timber-concrete joint is based on the "Wolf Nailplate, type 15N" acc. to Technical Approval Z-9.1-210 [16], shown in Fig. 9.

Length and thickness of the nail (15.5 and 1.5 mm) now are considerably smaller as compared to afore mentioned nailplate type MNP-A. No difference exists with respect to width of the nails, being $b = 3.2$ mm. Although not of primary importance for the slip and shear load capacity of the nail plate joint of Technical Approval Z-9.2-474 [13] it should be mentioned that the regarded joint is employed in a quite unusual timber concrete construction where the timber beams are used in compression and the steel bar reinforced concrete is loaded in tension. The reason for this at first view rather awkward use stems from the specific application of the prefabricated element in a specific prefabricated house construction. There the concrete slab provides an immediate usable wall and ceiling surface.
Fig. 8: View of "Merk nailplate, type MNP-A" acc. to Z-9.1-273 [15]

Table 1 contains a compilation of the test results concerning slip modulus and shear load capacity of the timber-concrete nailplate joints with "GN 200" and "MNP-A", given in [10], and the results obtained in Stuttgart for "Wolf 15N".

The table specifies the results for the total plate and for better comparison, also per centimeter of plate main direction being parallel to the timber-concrete interface. It can be seen that the slip moduli obtained for the three different nailplate types, however with similar anchorage depth in the timber, are very closely together, i.e.

$$\bar{K}_{ser} = 18.7; \ 18.4; \ 18.4 \ \text{kN} \left( \frac{1}{\text{cm}} \right) \text{ for GN 200; MNP - A; Wolf 15N}$$
Second, also the mean load capacities differ very little:

\[ f_v, \text{mean} = 2.0; \quad 1.8; \quad 2.1 \text{ kN} \left( \frac{1}{\text{cm}} \right) \text{ for GN 200; MNP – A; Wolf 15N} \]

The specified scatters (C.O.V.s) of the load capacities have to be seen in view of the different specimen numbers tested, being higher for "MNP-A" with \( n = 46 \) and considerably lower for "GN 200" and "Wolf 15N" with 5 and 14 tested joints, respectively. So, following, for a rough calculation of a 5% fractile an equal C.O.V. of 14% (= that of "MNP-A") was assumed for all three configurations.

<table>
<thead>
<tr>
<th>Reference</th>
<th>Nailplate type</th>
<th>( K_{s} \text{per cm of nailplate length} )</th>
<th>( R_{v,0} \text{per cm of nailplate length} )</th>
<th>C.O.V.</th>
<th>( X_{0} \text{kN/cm} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>5.3%</td>
<td>18.7</td>
</tr>
<tr>
<td>Karlsruhe tests [10; 11]</td>
<td>MNP-A [Z-91-2250]</td>
<td>400</td>
<td>18.4</td>
<td>14.0</td>
<td>0.8</td>
</tr>
<tr>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>5.3%</td>
<td>31.5</td>
</tr>
<tr>
<td>Stuttgart tests [Z-91-275]</td>
<td>MNP-A [Z-91-2250]</td>
<td>400</td>
<td>18.4</td>
<td>14.0</td>
<td>0.8</td>
</tr>
<tr>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>5.3%</td>
<td>24.3</td>
</tr>
<tr>
<td>Wolf 15N [Z-91-474]</td>
<td>Wolf 15N [Z-91-474]</td>
<td>280</td>
<td>18.4</td>
<td>14.0</td>
<td>0.8</td>
</tr>
</tbody>
</table>

\( X_{0} \) values based on C.O.V. = 14%, see text

Table 1 Results of compression shear tests on timber-concrete connections with different types and application modes of nailplates
The obtained estimates for the characteristic values range from 
\[ f_{v,0,k} = 1.4 \text{ to } 1.6 \text{ kN} \left( \frac{1}{\text{cm}} \right). \]

The slip modulus \( K_{ser} = 15.1 \text{ kN} \left( \frac{1}{\text{cm}} \right) \) specified in the Technical Approval Z-9.1-474 [13] for the timber-concrete application of the "Wolf 15N" nailplate, see Table 2, is somewhat reduced (about 20%) as compared to the test results (see Table 1) but quantitatively correct. The allowable shear force per centimeter plate length parallel to plate length (\( \alpha = 0^\circ \)) is specified in [13], obviously highly conservative, as \( f_{v,0,allow} = 0.4 \text{ kN/cm}; \) further, no characteristic design values are given.

*Fig. 9: View of "Wolf 15 N" nailplate acc. to Z-9.1-210 [16]*
3.1 Comparison with timber-timber connections

As in case of the pure nail connections discussed in chapter 2, it is of interest, how the quantity of the slip modulus of the "Wolf 15N" timber-concrete nailplate is related to pure timber-timber joints with this nailplate type.

According to Technical Approval Z-9.1-210 [16], the slip modulus for pure semi-rigid timber-timber composite action (see Fig. 10) for a couple of nailplates is

\[
K_{\text{ser}} = K_{\text{ser}}^{0} \cdot 0,25 \cdot ef \frac{1}{(1 + \kappa_c)}
\]  

(9 a)

where \(K_{\text{ser}}^{0} = 3.75 \text{kN/cm}, \ ef \ A = 2 (b - 2c) \cdot \ell \)

(9 b, c)

and \(\ell, b = \) length and width of nail-plate, \(c = \) marginal plate strip \((c = 10 \text{ mm})\) and

\[
\kappa_c = \frac{3 \cdot (b + 2c)^2}{(b - 2c)^2 + 4 \ell^2}
\]

(9 d)

So, in case of the regarded nailplate dimensions of "Wolf 15N" with \(\ell = 152 \text{ mm}\) and \(b = 127 \text{ mm}\), slip modulus per nailplate, \(K_{\text{ser}}\), and for a single nailplate length, \(K_{\text{ser}}\), evolve as

\[
K_{\text{ser}} = 94 \frac{\text{kN}}{\text{cm}} \quad \text{and} \quad \overline{K_{\text{ser}}} = 6.2 \frac{\text{kN}}{\text{cm}} \left( \frac{1}{\text{cm}} \right).
\]

The slip modulus for ultimate limit state is defined in [16], as usual, by

\(K_u = 2/3 \overline{K_{\text{ser}}}\).

Assuming, that the obtained slip modulus for the timber-timber connection can be taken as a realistic estimate of an experimental mean value underlaying
the Technical Approval, an even more marked difference between the timber-timber and the timber-concrete joint as in case of the afore regarded nailed connections is obvious. Now, in case of the nailplate (specifically "Wolf 15N") the slip modulus of the timber-concrete application is 3 times higher as compared to the timber-timber joint. (Note: in case of the nails, the increase was by a factor of 2.) It is presumed that the restriction of any rotation of the nailplate as a total contributes to the extra stiffness increase. A thorough explanation for this will be forwarded separately.

Comparing the shear capacities of the nailplates in timber-concrete connections vs. timber-timber joints no published characteristic or mean shear load capacities are known to the authors for the regarded timber-timber nailplate

<table>
<thead>
<tr>
<th>type of connector</th>
<th>shear capacity f(_{v, a}) per cm of nailplate length</th>
<th>slip modulus per nailplate</th>
<th>type of joint</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>f(_{v, a, b}) (\text{in KN/cm})</td>
<td>(K_{\text{sec}}) (\text{in KN/cm}^{1/2})</td>
<td>Demerit timber-concrete</td>
</tr>
<tr>
<td>0.17</td>
<td>2.0</td>
<td>15.1</td>
<td>0.4</td>
</tr>
</tbody>
</table>

Table 2: Compilation of slip moduli and allowable shear capacities of regarded nailplates specified in Technical Approvals for timber-concrete and timber-timber joints
joints. According to the current approach for derivation of allowable stresses / capacities a global safety factor of 3 is applied to the mean value. This procedure delivers the mean load capacity value estimations in Tab. 2; the characteristic values (5%-fractiles) are estimated by assumption of a C.O.V. of about 15%. Not focussing on the absolute numbers of the estimated mean / characteristic shear capacities of the MNP-A and Wolf 15N nailplates, when applied in a timber-timber joint, the comparison with the related timber-concrete joints clearly indicates a significant, very roughly 1.5 times increase of the load capacity in case of the timber-concrete joint. This statement does not include a minor, here unknown correction factor for a deviating yield strength of the nailplates in the timber-concrete joints as compared to the nominal requirements. One of the reasons for the load capacity increase could be, that the failure of the nailplate in the timber-concrete joint is fully determined by yielding and destruction of the nailplates in the concrete-timber interface, as shown in Fig. 11, which in that expressed manner does not occur in pure timber-timber connections. Here, clearly further research is needed.

![Fig. 11: View of timber-concrete nailplate connection acc. to [13] in a compressive shear test](image)

a) test set-up  

b) failure state of the nailplate
CONCLUSIONS

Recapitulatory it can be stated that conventional smooth and threaded nails of medium and small sizes, when used for timber-concrete connections, should show very roughly

- the same (characteristic) shear capacity and
- a 2 times higher slip modulus

as obtained / calculated for a timber-(thick) steel plate connection subjected to single shear. Both findings are qualitatively sensible.

Nailplates subjected to shear loading and embedded roughly equally in timber and concrete shows considerably increased stiffness and strength values as compared to an analogous timber-timber joint. In a very rough approximation, valid for the discussed dimensional range of regarded nailplates, it can be well assumed, that, compared to timber-timber joints

- mean / characteristic shear capacity increases by a factor of about 1.5
- slip modulus increases by a factor of about 2.5 – 3.

ACKNOWLEDGEMENTS

The continuous good co-operation between Wood and Timber Construction Department of Otto-Graf-Institute and Laboratoire de Rheologie du Bois de Bordeaux (LRBB) is gratefully acknowledged. Special thanks are indebted to the director of LRBB, Dr. Patrick Castera, in this context, for his repeated yearly favor of French translation of our OGI-paper abstracts.

REFERENCES


SUMMARY

The paper deals with rectangular holes in glulam members subjected to bending. Introductory, the decisive design relevance of the stresses perpendicular to fiber and beam direction is outlined. Then, exemplarily, the influence of essential geometric quantities, being – radius of curvature of the corners and aspect ratio of the rectangular holes – are revealed. Hereby the stochastic defect structure of the material glulam is considered, too.

Next, the design approaches according to the drafts of DIN 1052 and EC 5, differing fundamentally with respect to idealisation of the mechanical problem, are outlined. The design proposal of DIN 1052 incorporates a classical strength of materials criterion whereas the EC 5 design model is based on fracture mechanics.

A comparison of the two stated design approaches reveals partly extreme differences of the computational characteristic load capacities. The reason therefore results from the different mechanical models and the different recognition of obviously relevant influencing parameters. A newly granted research project shall contribute to the elaboration of a unanimously accepted, empirically validated design model for holes in glulam beams.

ZUSAMMENFASSUNG

Der Beitrag befaßt sich mit rechteckigen Durchbrüchen in biegebeanspruchten Brettschichtholzträgern. Einführend wird kurz die ausschlaggebende

Es folgt eine Darlegung der Bemessungsansätze in den Entwürfen zu DIN 1052 und EC 5, die sich hinsichtlich der Idealisierung des mechanischen Problems fundamental unterscheiden. Dem Bemessungsvorschlag in DIN 1052 liegt ein klassisches Höchstspannungskriterium zugrunde während das EC 5 Bemessungsmodell von einem bruchmechanischen Ansatz ausgeht.


**RESUME**

On s’intéresse dans cet article à la présence de trous rectangulaires dans des poutres en lamellé collé sollicitées en flexion, en portant l’attention sur les contraintes perpendiculaires aux fibres, décisives pour le dimensionnement. Ainsi, par exemple, l’influence de grandeurs géométriques essentielles – rayon de courbure des angles et rapport de forme du trou – est mise en évidence.

La nature stochastique des défauts du lamellé collé est également considérée. En s’appuyant sur les règles de dimensionnement relatives aux projets de normes DIN 1052 et EC5, on obtient des différences fondamentales sur l’idéalisation du problème mécanique. La proposition émanant de la norme DIN 1052 utilise un critère de résistance des matériaux, alors que le modèle de dimensionnement de l’EC5 est basé sur la mécanique de la rupture.

La comparaison des deux approches fait apparaître des différences extrêmes sur la capacité portante simulée. La raison provient donc des différents modèles mécaniques utilisés et d’une prise en compte différente de paramètres dont l’influence est évidente. Un nouveau projet de recherche financé contribuera à
l’élaboration d’un modèle de dimensionnement unanimement accepté, et validé expérimentalement.

KEYWORDS: Glulam, rectangular holes, design approaches, stresses perpendicular to grain, Weibull stress, hole aspect ratio, curvature of corner

1. INTRODUCTION

The design of glulam beams with holes is treated considerably different in timber design codes. Examples are the latest drafts of Eurocode 5 and of the German timber design code DIN 1052. In the first case a solution based on a linear fracture mechanics approach is stated whereas in the latter case a strength of materials design is given. Further, in both design models essential geometrical and section force influences are treated considerably different. Concerning round holes, the stated differences have been treated earlier in [1]. In this paper the issue of rectangular holes is discussed.

The paper first shortly reveals the design relevance of tension stresses perpendicular to grain. Following the influence of radius of curvature of the corners and of aspect ratio of the hole is discussed. Both mentioned design approaches are then compared for representative configuration of different beam, hole and section force combinations. The effect of different glulam strength classes is considered, too.

2. SOME BASIC CONSIDERATIONS ON THE PROBLEM

In the following only straight beams subjected to bending are regarded. This means that the hole periphery is in general subjected to a combined shear force and moment action. In rare occasions pure moment loading of the member section with the hole may occur.

The hole disturbs the stress flow due to shear force V and/or bending moment M; this influences all stress components. The distributions of the stresses $\sigma_x$, $\sigma_y$ and $\tau_{xy}$ at selected paths parallel to beam depth in the area/vicinity of a square hole for a general, combined $M + V$ load case are shown in Fig. 1. In the given example with $M/V = 3$, the radius of the not sharp edged corner was taken as $r = 0.05 h_d$. This matter is discussed in more detail in chap. 3. The orthotropic stiffness ratios employed in the FE analysis were throughout assumed as $E_x/E_y = 30$, $E_x/G_{xy} = 16$ and $\nu_{xy} = 0.015$. The diagrams show that at the design
relevant path II all stress components reveal a pronounced peak at the locations of the corners. At the upper corner of path II the peaks of tension stresses parallel and perpendicular to grain interact with a shear stress peak. Regarding the magnitude of the three stresses relative to their respective strength values, for instance via a Norris stress interaction criteria, we see that tension stress perpendicular to grain is by far most damage relevant. This is the reason that the design approaches in the drafts of DIN 1052 and EC 5 account explicitly (DIN 1052) or implicitly (EC 5) exclusively for a damage relevance of tension stress perpendicular to grain. Following the focus is also only on tension stress perpendicular to grain, however the sketched stress interaction should be kept in mind.

Fig. 1: Distributions of stresses $\sigma_x$, $\sigma_y$ and $\tau_{xy}$ along selected paths in the vicinity of a rectangular hole subject to a combined moment/shear force load case $M/V = 3 \, h$. Beam geometry, sizes: $h_d/h = h/3$, $h = 900 \, \text{mm}$, $b = 120 \, \text{mm}$, $r = 0.05 \, h_d$
For assessment of the influences of section forces M and V, considered differently in both code drafts, it is advantageous to regard the effect of the load case pure moment action and the fictive load case “pure” shear force action separately. A detailed description of the stress computation for the fictive “pure” V load case is stated in [2].

Due to pure moment action M the stress concentration is located at the vertical edge of the hole (Fig. 2a) whereas due to “pure” shear force action V the stresses concentrate in the corners of the hole (Fig. 2b). The shapes of the stress fields for the two load cases are similar to those obtained for round holes [2]. A combined M+V load case produces an unsymmetrical stress field around the hole which is a superposition of the two pure load cases.

![Stress distributions perpendicular to grain at the hole periphery for the two pure load cases](image)

**Fig. 2a, b:** Stress distributions perpendicular to grain at the hole periphery for the two pure load cases  
*a)* pure moment action  
*b)* “pure” shear force action

### 3. EFFECT OF CURVATURE OF THE CORNERS

A crucial matter for rectangular holes are the corners. In case of rectangular sharp notched corners, i.e. radius of curvature \( \rightarrow 0 \), a stress singularity arises. In order to avoid this, the corner generally will be not made right-angled but produced with a curvature \( 1/r \). It is trivial that the maximum stress depends strongly on the radius. However, for failure initiation due to tension stresses perpendicular to grain in a brittle material with stochastically distributed defects, the area/volume, here denoted by \( \Omega \), and the shape of the stress distribution \( \sigma_y = \sigma_{90} \)
of the high stressed region are more relevant. An adequate procedure to quantify the damage relevancy of an inhomogeneously stressed volume is the so-called Weibull stress

\[
\sigma_{90,\text{wei}} = \left( \frac{1}{\Omega} \int_{\Omega} \sigma_{90}^m(x,y,z) \, d\Omega \right)^{1/m}.
\]

The effect of two different radii on maximum and Weibull stresses is revealed exemplarily for a beam of depth \( h = 900 \text{ mm} \) with a relative hole size of \( h_d/h = 0.3 \) (\( h_d = 270 \text{ mm} \)) subjected to “pure” shear force action (Fig. 3). An increase of the radius from \( r = 0.05 \, h_d = 13.5 \text{ mm} \) to \( r = 0.15 \, h_d = 40.5 \text{ mm} \) forwards a strong reduction of the maximum stresses at the corner, giving a stress ratio of

\[
\frac{\sigma_{90,\max,r=0.15h_d}}{\sigma_{90,\max,r=0.05h_d}} = 0.64
\]

Apart from the immediate hole vicinity the stress distribution is not affected by differences of the radii as shown in Fig. 3. Considering now the whole stress field perpendicular to grain in the corner area and calculating the Weibull stress with a generally agreed size exponent \( m = 5 \) the difference becomes considerably smaller

\[
\frac{\sigma_{90,\text{wei},r=0.15h_d}}{\sigma_{90,\text{wei},r=0.05h_d}} = 0.88.
\]

Fig. 3: Tension stress perpendicular to grain \( \sigma_y = \sigma_{t,90} \) at highest stressed section for two different radii of the corners in case of “pure” shear force action of 10 kN
In the drafts of DIN 1052 and EC 5 the corner radius is equally prescribed as \( r \geq 15 \text{ mm} \), irrespective of hole and beam size. According to the authors’ knowledge, this construction detailing is not bound to any considerations of the above type and should be analysed appropriately.

4. INFLUENCE OF THE ASPECT RATIO OF THE HOLE

The aspect ratio of the rectangular hole is considered considerably different in the drafts of DIN 1052 and EC 5. Whereas DIN 1052 does not consider any influence of the aspect ratio of the hole on load capacity, EC 5 specifies a significant load capacity reduction with increasing aspect ratio for same hole depth \( h_d \). Apart thereof, both design codes state equally the following absolute/relative limits for the dimensions of rectangular holes, being

\[
a \leq h \quad \text{and} \quad h_d \leq 0.4 \, h
\]

where \( a \) and \( h_d \) are the hole dimensions parallel and normal to beam axis. Thus the maximum “allowable” aspect ratios reach from

\[
a/h_d \leq 10 \quad \text{for} \quad h_d/h = 0.1
\]

\[\text{to}\]

\[
a/h_d \leq 2.5 \quad \text{for} \quad h_d/h = 0.4.
\]

![Graph showing tension stress perpendicular to grain \( \sigma_y \) at highest stressed section acc. to “pure” shear force action of 10 kN for three different aspect ratios](image)

**Fig. 4:** Tension stress perpendicular to grain \( \sigma_y = \sigma_{t,90} \) at highest stressed section acc. to “pure” shear force action of 10 kN for three different aspect ratios
Table 1: Maximum and Weibull stresses for different aspect ratios; also given are the stress values normalised to the square hole reference case

<table>
<thead>
<tr>
<th>stress</th>
<th>unit</th>
<th>aspect ratio a/h_d</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \sigma_{y,\text{max}} )</td>
<td>N/mm(^2)</td>
<td>0.512 0.628 0.746</td>
</tr>
<tr>
<td>( \sigma_{\text{wei}} )</td>
<td>N/mm(^2)</td>
<td>0.144 0.170 0.193</td>
</tr>
<tr>
<td>( \sigma_{y,\text{max},n} \text{ 1) }</td>
<td>-</td>
<td>1.00 1.23 1.46</td>
</tr>
<tr>
<td>( \sigma_{\text{wei},n} \text{ 1) }</td>
<td>-</td>
<td>1.00 1.18 1.34</td>
</tr>
</tbody>
</table>

1) normalised to the aspect ratio a/h_d = 1

The appropriateness of a consideration of the aspect ratio in the design equations was checked in the frame of this paper exemplarily for the beam configuration, studied before with respect to the influence of the corner radius. Now, in all cases \( r = 0.05 \ h_d = 13.5 \ \text{mm} \) is considered. Additionally to the square hole aspect ratio of \( a/h_d = 1 \) regarded in Fig. 3, Fig. 4 specifies the \( \sigma_y \) stress distributions for the aspect ratios \( a/h_d = 2 \) and \( 3 \). Table 1 contains the maximum and Weibull stresses for the different aspect ratios; also given are the stress values normalised to the square hole reference case. It can be seen that the maximum stresses increase for the aspect ratios \( a/h_d = 2 \) and \( 3 \) pronouncedly by 23% and 46%, respectively. The Weibull stresses increase slightly less but comparable by 18% and 34%. It is evident, that the aspect ratio should be accounted for in the design equations.

5. DESIGN OF RECTANGULAR HOLES ACCORDING TO DRAFT DIN 1052

Following the design for rectangular holes as specified in the revised draft of the new semi-probabilistic German timber design code [3] is given. The design model represents a classical strength of materials approach. Hereby the design tension force perpendicular to grain at the hole periphery, \( F_{t,90,d} \), is compared to the design value of the resistance \( R_{t,90,d} \) (\( R_{t,90,d} \) not specified explicitly)

\[
\frac{F_{t,90,d}}{R_{t,90,d}} = \frac{F_{t,90,d}}{0.5 \ l_{t,90} \ b \ f_{t,90,d}} \leq 1 \quad (2a)
\]

where

\[
l_{t,90} = 0.5 \ (h_d + h) \quad (3)
\]

is the distribution length of the assumed triangular stress distribution perpendicular to grain (see also Fig. 5), \( b \) is beam width and \( f_{t,90,d} \) is the design ten-
Design of rectangular holes in glulam beams

Tension strength perpendicular to grain. Rewritten as the ratio of a design stress \( \sigma_{t,90,d} \) vs. design strength \( f_{t,90,d} \), Eq. (2a) reads

\[
\frac{\sigma_{t,90,d}}{f_{t,90,d}} \leq 1 \quad \text{where} \quad \sigma_{t,90,d} = \frac{F_{t,90,d}}{0.5 l_{t,90} b} \quad (2b), (4)
\]

The design value of the tension force \( F_{t,90,d} \) is composed of two additive parts bound to the separate actions of shear force and bending moment

\[
F_{t,90,d} = F_{t,V,d} + F_{t,M,d} \quad (5)
\]

where

\[
F_{t,V,d} = V_d \eta_V \quad \text{and} \quad \eta_V = \frac{1}{4} \frac{h_d}{h} \left[ 3 - \frac{h_d^2}{h^2} \right] \quad (6)
\]

\[
F_{t,M,d} = M_d \eta_M \quad \text{and} \quad \eta_M = \frac{0.008}{h_r} \quad (7)
\]

and \( V_d, M_d \) absolute values of design shear force and bending moment at the hole edge

and

\[
h_r = \min \{ h_{rl}; h_{ru} \} \quad \text{where} \quad h_{rl(ru)} \geq 0.25 h \quad (8)
\]

Further, as already mentioned in chap. 3, the restrictions \( h_d \leq 0.4 h, a \leq h \) and \( r \geq 15 \text{ mm} \) apply.

---

1 whichever delivers unfavorable results

2 DIN notations \( h_{ro} \) and \( h_{ru} \) were changed to EC 5 notations \( h_{ru} \) and \( h_{rl} \)
Some comments on the background and limits of the specified equations seem appropriate (in the following, for sake of simplicity, the subscript \(d\) is omitted, i.e. nominal resp. characteristic values are regarded):

- Tension force \(F_{t,V}\) bound to the shear force \(V\), specified in Eq. (6), represents one half of the resultant of the shear stresses \(\tau_{xy}\) which can not be transferred in the hole area (see Fig. 5)

\[
F_{t,V} = b \int_0^{h_v/2} \tau_{xy} dy = V \eta_V, \quad \tau_{xy} = \frac{3 V}{2 b h} \left( 1 - 4 \frac{y^2}{h^2} \right)
\]

(9a,b)

By integration of the stresses perpendicular to grain, as resulting from FE analysis, it can be shown that Eqs. (6) and (9) deliver the correct stress resultant when the integration is performed over the whole stress distribution length (including also compression stress areas until the stresses perpendicular to grain become zero).

- Tension force \(F_{t,M}\) bound to the bending moment, specified in Eq. (7), is not based on analytical or numerical stress analysis but stems from a calibration to experimental data in different literature sources [4]. The performed calibration procedure can be questioned. A preliminary finite element study for determination of \(F_{t,M}\) delivered a considerably different result similarly as in the analogous case of round holes, analysed in [2].

- The assumed triangular stress distribution represents a rather crude but somehow acceptable engineering approximation of the actually exponential stress distribution. However the distribution length \(l_{t,90}\) as specified by Eq. (3) is considerably too long. This is illustrated exemplarily in Fig. 6. The graph shows the distribution of tension stress perpendicular to grain according to finite element analysis and for the given DIN 1052 design approach for a “pure” shear force load case \((M/V = 0)\). Two different radii of curvature are regarded. In detail, the comparison of the stress distributions is performed for the ultimate (= characteristic) shear force state \(V_k\) defined by the DIN approach through Eqs. (2a), (5), (6) and (7), giving

\[
V_{k(DIN)} = f_{t,90,k} 0.5 l_{t,90} b / \left( \eta_V + \eta_M M / V \right).
\]

(10)
With tension strength $f_{t,90,k} = 0.5 \text{ N/mm}^2$ (constant for all glulam strength classes acc. to draft DIN 1052). Eq. (10) delivers $V_{k(DIN)} = 80.5 \text{ kN}$ as input for the FE analysis. It can be seen from the graphs that the nonlinear stress distribution according to continuum analysis shows a distinctly higher stress gradient and a much higher stress level closer to the hole periphery and hence shorter stress distribution lengths $l_{t,90}$. The maximum stresses according to continuum analysis might at first view be considered too high; however for this judgement, not followed up here, the actually stressed volume has to be taken into account.

- The assumed triangular stress distribution acc. to DIN 1052 is independent from the moment/shear force ratio, what is not corresponding with numerical solutions.

![Graph showing tension stress vs. stress distribution length](image)

Fig. 6: Tension stress $\sigma_y$ perpendicular to grain vs. stress distribution length $l_{t,90}$ at highest stressed section for a “pure” shear load action at failure state $V_k$ according to E DIN 1052 ($V_k = 80.5 \text{ kN}$) and according to continuum analysis bound to load $V_{k(DIN)}$

- Tension stress/force perpendicular to grain, $F_{t,90}/\sigma_{t,90}$, acc. to E DIN 1052 are equal for square and rectangular holes. However, as shown in chap. 3, the aspect ratio of the hole has an influence on stresses which increase considerably with aspect ratios $a/h_d > 1$. 
6. DESIGN OF RECTANGULAR HOLES ACCORDING TO DRAFT OF EUROCODE 5

The linear fracture mechanics based strength verification for a glulam beam with a rectangular hole subjected to design shear force $V_d$ and design moment $M_d$ at the center of the hole is conducted as for a notched beam subjected to a shear force $V_d/2$ [5] (see Fig. 7). The effect of the additional moment on the load capacity is not considered. The design equation formally reads as an approach based on the comparison of design shear stress $\tau_d$ vs. design shear strength $f_{v,d}$ which is reduced by a factor $k_V$ depending on absolute beam depth and relative hole size

$$\frac{\tau_d}{k_V f_{v,d}} \leq 1 \quad \text{and} \quad \tau_d = \frac{1.5 V_d}{b h_{ef}}. \quad (11a, b)$$

Factor $k_V$ is defined by

$$k_V = \min \left\{ \frac{1}{\sqrt{h^*}} \left( \frac{1}{\sqrt{\alpha (1-\alpha)}} + 0.8 \frac{x}{h^*} \sqrt{\frac{1}{\alpha} - \alpha^2} \right) \right\} \quad (12)$$

where

- $h^* = h/2$
- $x = \text{distance from line of shear force action to the corner} = a/2$
- $\alpha = h_{ef}/h^*$

$k_n = \begin{cases} 5 & \text{for solid timber} \\ 6.5 & \text{for glulam} \end{cases}$

The relevant constructive restrictions were mentioned in chap. 3.

![Fig. 7: Dimensions of rectangular holes in beams and respective approximations for the notched beam design according to EC 5; leftside: actual geometry; rightside: notched beam approximation](image-url)
The following comments on the background and limits of the specific equations seem appropriate:

- The fracture mechanics bound design equation for an end-notched beam is based on total energy release rate [6]. As fracture mechanism, exclusively Mode I crack opening was assumed. So the implicitly incorporated basic material resistance is the characteristic fracture energy \( G_{f,k} \) in tension perpendicular to grain. The formally shear strength based resistance side in Eq. (11a) is simply the result of an equation multiplication by \( f_{v,k}/f_{v,k} \).

- The basic analytical end-notched beam solution was calibrated to experimental results for the end-notched beam case with a factor of 2/3.

- Characteristic fracture energy \( G_{f,k} \) was eliminated from the resistance side by the approximation that expression

\[
k_n = \frac{1}{3} \sqrt[3]{\frac{G_{f,k} E_{90,05}}{f_{v,k}^2}}
\]

is approximately 5 and 6.5 for solid wood and glulam, respectively, throughout all strength classes.

- The design according to EC 5 takes into account the shape of the rectangular hole. Coinciding with the results in chap. 3, rectangular holes with aspect ratios \( a/h_d > 1 \) result in reduced \( k_v \) values.

7. **COMPARISON OF LOAD CAPACITIES ACCORDING TO DRAFTS OF E DIN 1052 AND EC 5**

For a quantitative comparison of both design approaches these are evaluated for characteristic shear force with and without consideration of a bending moment influence. The comparison comprises the following beam, hole sizes and geometries:

- beam depth \( h \): \( h_1 = 450 \text{ mm} \), \( h_2 = 2 \ h_1 = 900 \text{ mm} \) and \( h_3 = 3.33 \ h_1 = 1500 \text{ mm} \)

- beam width \( b \): \( b = \text{constant} = 120 \text{ mm} \)

- hole to depth ratio: ranging from 0.1 to 0.4

Another important aspect when comparing the two drafts consists in the considered glulam strength class. In principle, strength class should have no impact, i.e. both design approaches should agree/disagree similarly for all strength
classes. Unfortunately this is not the case, as shear strength $f_{v,k}$ and tension strength perpendicular to grain $f_{t,90,k}$, relevant in this context, are not specified equally for same glulam strength classes in DIN 1052 and EN 1194. (Note: The latter standard is the European glulam strength class standard to be used in EC 5.) The differences are shown in Tab. 2. It can be seen that the characteristic strength values $f_{t,90,k}$ and $f_{v,k}$ according to DIN 1052 remain constant for all glulam strength classes whereas the respective values according to EN 1194 depend strongly on the glulam strength class. So, the comparison of the design models for holes in glulam is superimposed by obvious uncertainties on the true strength properties $f_{t,90,k}$ and $f_{v,k}$. Therefore the hole design comparison is conducted for two glulam strength classes, one with rather dissimilar strength values/ratios (= the low glulam strength class GL 24c) and one with rather similar strength values in both codes (= the high glulam strength class GL 32h).

Table 2: Characteristic strength values [N/mm$^2$] for glulam of combined (c) and homogeneous (h) build-up acc. to European Standard EN 1194 and the German draft design code DIN 1052

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<thead>
<tr>
<th>Standard</th>
<th>Characteristic strength value</th>
<th>Glulam strength class</th>
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<td>DIN 1052</td>
<td>$f_{t,90,k}$</td>
<td>GL 24c 0.5 GL 24h 3.5</td>
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<tr>
<td></td>
<td>$f_{v,k}$</td>
<td>GL 28c 0.40 GL 28h 0.45</td>
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<tr>
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<td></td>
<td>GL 32c 0.45 GL 32h 0.50</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GL 36c 0.50 GL 36h 0.60</td>
</tr>
<tr>
<td>EN 1194</td>
<td>$f_{t,90,k}$</td>
<td>0.35 0.40 0.40 0.45 0.45 0.50 0.50 0.60</td>
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<tr>
<td></td>
<td>$f_{v,k}$</td>
<td>2.20 2.70 2.70 3.20 3.20 3.80 3.80 4.30</td>
</tr>
</tbody>
</table>

First, the computational shear force capacities without consideration of a bending moment influence (M/V = 0) are regarded. Figures 8a, b show the shear force capacity $V_k$ depending on the hole to depth ratio $h_d/h$ for the 3 different beam depths as resulting from the EC 5 and DIN 1052 approach. Figure 8a illustrates the case for strength class GL 24c and Fig. 8b refers to GL 32h. A comparison of the results of the different design approaches reveals in general considerable discrepancies, discussed below in more detail.

Regarding strength class GL 24c (Fig. 8a), the two design approaches reveal similar characteristic shear force capacities for small beams (here: $h = 450$ mm). However, with increasing beam depth the shear force capacities differ extremely. This is an immediate consequence of the different recognition of the depth effect in the linear fracture mechanics approach (EC 5) and in the strength
Design of rectangular holes in glulam beams

Fig. 8a, b: Characteristic shear force capacity of glulam beams with a square hole without consideration of a bending moment influence according to drafts of EC 5 and DIN 1052 depending on the hole to depth ratio and on the beam depth

a) for glulam strength class GL 24c
b) for glulam strength class GL 32h

Fig. 9a, b: Characteristic shear force capacity of a glulam beam with a square hole depending on the hole to depth ratio for different moment shear force ratios M/V according to drafts of EC 5 and DIN 1052

a) for glulam strength class GL 24c
b) for glulam strength class GL 32h
of materials approach (DIN 1052), respectively. In the first case, load capacities increase proportional to $\sqrt{h}$ and in the second case proportional to $h$.

Regarding strength class GL 32h (Fig. 8b) the characteristic shear force capacities according to EC 5 increase by the ratio of characteristic shear strengths 3.8/2.2 = 1.7 vs. the results specified for GL 24c whereas the values according to DIN 1052 do not change. Consequently, now the predicted load capacities differ strongly for small beams whereas for medium sized beam depths throughout a rather good agreement can be stated. This is also true for large beams when regarding medium sized holes ($h_d/h \approx 0.15 – 0.25$).

Figures 9a, b show the shear force capacity for both design approaches, now considering the influence of a bending moment, too. In detail the results are given for a medium sized beam ($h_2 = 900$ mm) for the glulam strength classes GL 24c and GL 32h. The parametric dependency of the DIN solution on the section force ratio $M/V$ is specified for the realistic range of $M/V = 0$ to 10 $h$. Ratios $M/V$ of up to about 2.5 $h$ relate to holes very close to the supports, the larger ratios increasingly mirror constructions with holes closer to mid-span. It can be taken from the graph that the additional incorporation of the bending moment influence results in a tremendous reduction of the load capacity of the beam according to DIN 1052 as compared to the EC 5 approach.

Finally, the differences of both design codes concerning aspect ratio of the hole are quantified. As stated, DIN 1052 does not account explicitly for the aspect ratio of the hole. (Note: Implicitly the effect is considered in some way, as the design section forces in DIN 1052 are to be evaluated for the vertical edges of the hole what includes aspect ratio on the action side.) Contrary, EC 5 incorporates an expressed influence of the aspect ratio on the resistance side. The effect is depicted in Fig. 10 as the ratio of shear force capacity of a rectangular hole with aspect ratio $a/h_d > 1$ vs. a square hole with $a/h_d = 1$. The ratio is given for different hole to depth ratios $h_d/h$. Further the specified constructive limits $a \leq h$ and $h_d/h \leq 0.4$ are accounted for.

The graph reveals a marked capacity reduction with increasing aspect ratios. So for example, for a realistic hole geometry with a hole to depth ratio of $h_d/h = 0.3$ and an aspect ratio of $a/h_d = 3$, the shear force capacity drops by 40% vs. the square hole. This strength reduction may be compared with the results of the continuum calculation on the influence of aspect ratio in chap. 4. There, the
studied example delivered for the same hole geometry shows increases of maximum and Weibull stresses of 1.46 and 1.34, respectively. Assuming a usual strength of materials criterion, the load capacity reduction vs. the square hole case would be 32% and 25%, respectively. This confirms very roughly the order of magnitude indicated by the EC 5 approach, which probably delivers a too conservative estimate of the influence of the aspect ratio.

8. CONCLUSIONS

A comparison of the design approaches in DIN 1052 and EC 5 for rectangular holes in glulam beams revealed strong discrepancies in many aspects. The design approaches which are based on fundamentally different mechanical models differ not only in the predicted load capacities, being very unsatisfactory from a safety point of view, but also in a different recognition of essential parameters. Important influences which should be accounted for are:

- size effect (generally acknowledged in tension perpendicular to grain problems)
- moment to shear force ratio
- hole to depth ratio
- aspect ratio of the hole
- model invariance vs. different glulam strength classes
The two design approaches account for the above mentioned parameters as following:

- A size effect is considered in the EC 5 approach whereas no size effect is regarded in the DIN 1052 approach. The EC 5 model incorporates a depth influence according to linear fracture mechanics with the factor \( \sqrt{h} \).

- The moment to shear force ratio is considered in the DIN 1052 approach but not in EC 5. However, the magnitude of the M influence according to DIN 1052 does not agree with results from FE analysis.

- Both design approaches describe the influence of the hole to depth ratio similar for the “pure” shear force case. This parameter seems to be implemented correctly.

- The aspect ratio of the hole is taken into account in the EC 5 approach but not in DIN 1052. It was confirmed that a correct model should consider this influence.

- The agreement/disagreement of both design models depends strongly on the specifically regarded glulam strength classes as the design relevant strength values are specified different in the respective codes/ supplemental standards.

In view of the multiplicity of important design parameters and recognising the strength variability of the material glulam it is obvious that an improved, commonly agreed model needs a substantial experimental data base for design equation calibration. The discussed aspects are followed up in a ongoing research project at Otto-Graf-Institute.

**ACKNOWLEDGEMENTS**

The authors want to express sincere thanks to Patrick Castera, Head of Laboratoire du Bois de Bordeaux (LRBB), for his repeated favour in performing the translation of the French abstract.

The authors are especially grateful to German Institute for Building Technique (DIBt), Berlin, for recently granted funding of a research project on design of holes in glulam beams.
The funding of basic aspects of damage evolution at the periphery of holes in glulam beams by Deutsche Forschungsgemeinschaft via grants to SFB 381 and hereby to subproject A8 “Damage and NDT of the natural fibre composite material wood” is gratefully acknowledged.

REFERENCES


Measurements of Acoustic Anisotropy of Soft and Hard Wood; Effects on Source Location

Thomas Ringger, Lilian Höfflin, Gerhard Dill-Langer, Simon Aicher

SUMMARY

The paper reports on measurements of the acoustic anisotropy of wood, i.e. the dependency of the velocity of ultrasonic bulk waves propagating through wood at different angles between the direction of the wave propagation vector and the principal growth directions. For both, soft wood (represented by spruce) and hard wood (represented by beech), the velocity measurements were conducted by means of transmission method of pulsed ultrasound (US). The main part of the measurements was performed with small cubes of clear wood (for both, spruce and beech), another part with one macroscopic board-like timber specimen (for spruce only). The velocity determination was based on the "time-of-flight" of the US pulses, defined as the time lag between the trigger time of the pulse generator and the on-set time of the transmitted fully recorded signal.

The velocity results showed some characteristic differences for the comparison between spruce and beech with a throughout more pronounced acoustic anisotropy in case of spruce.

Based on the results for spruce, the influence of acoustic anisotropy on the location of “artificial” ultrasound sources has been analysed. The calculation of the source co-ordinates by means of time-of-flight measurements with several spatially distributed transducers forwarded a pronounced improvement of location results, when the empirically obtained acoustic anisotropy was considered.
ZUSAMMENFASSUNG


Die Ergebnisse der Geschwindigkeitsmessungen zeigten einige charakteristische Unterschiede zwischen Fichte und Buche mit einer durchgängig deutlicher ausgeprägter Anisotropie im Falle der Nadelholzart Fichte.


RESUME

Cet article présente et analyse des mesures d’anisotropie acoustique du bois, i.e. de la dépendance de la vitesse de propagation d’ondes ultrasonores dans le bois vis-à-vis de l’angle entre la direction du vecteur de propagation et les axes principaux de croissance. Les mesures de vitesse ont été effectuées sur un résineux (l’épicéa) et un feuillus (le hêtre) par une méthode basée sur la transmission d’ondes ultrasonores (US). Une partie des analyses a été réalisée sur de petits cubes de bois sans défaut (épicéa et hêtre), et une autre partie sur des spécimens de dimension structurale (épicéa seulement). La détermination de la vitesse est basée sur le temps de propagation de l’onde, défini comme
l’intervalle de temps entre l’impulsion du générateur d’ondes et l’enregistrement du signal transmis par un récepteur.

Les résultats obtenus font apparaître des différences prononcées entre l’épicéa et le hêtre, dont une anisotropie acoustique plus visible dans le cas de l’épicéa.

A partir des résultats obtenus sur l’épicéa, l’influence de l’anisotropie acoustique sur la localisation des sources artificielles d’émission acoustique a été analysée. Le calcul des coordonnées de la source à partir des mesures de temps de propagation jusqu’à des récepteurs spatialement répartis a conduit à une amélioration des résultats de localisation lorsque l’on utilise les valeurs expérimentales d’anisotropie acoustique, plutôt que les valeurs de la littérature.

Nous avons trouvé que la localisation des sources artificielles d’émission acoustique n’est possible que si l’on prend en compte l’anisotropie acoustique.

KEYWORDS: acoustic anisotropy, wood, non-destructive testing, ultrasound, pulse transmission, location of ultrasound sources

1. INTRODUCTION

The location of vibration sources, e.g. earthquake epicentres or acoustic emission sources, by means of relative arrival times of the generated waves at several spatially distributed receivers is an important method for damage characterisation both, in Geophysics and Material Science [6, 7 and 3]. Thereby, the velocity of wave propagation is the decisive parameter for accurate source location results.

While the time-of-flight of pulsed ultrasound (US) in an isotropic medium depends solely on the distance between source and receiver, the transition time in an anisotropic medium depends also on the angles between the pulse wave vector and the directions of the material anisotropy axes. The natural material wood represents growth-bound a pronounced anisotropic material with approximately cylindrical symmetry and three principal directions, being the longitudinal direction (L) parallel to fiber or stem axis, the tangential direction (T) following the curvature of the annual rings and the radial direction (R) perpendicular both, to the tangential and the longitudinal direction.

For the dependency of the wave velocity on the angle between the propagation vector and the growth directions, several studies can be found in literature
[1, 2, 12]. Under certain assumptions (pure elastic material law, plane waves, infinite test volume) the acoustic anisotropy can be calculated from the anisotropic stiffness parameters [5, 10]. However, the velocity results due to time-of-flight measurements of ultrasound pulse transmission differ considerably from results due to phase shift method. Moreover, published data vary considerably due to several influencing factors, being e. g. variability within the wood species, boundary conditions of the respective test set-up and pulse shape in the time and frequency domain.

The reported ongoing study aims at the establishment of a reliable empirical data base for the dependency of ultrasound pulse velocity on angle between propagation and principal directions. This is first done for a given set of circumstances, i.e. for two specimen geometries, two wood species and one pulse shape technically bound to the available ultrasound pulse generator. The results shall serve as a basis for an adequate application of the ultrasound source location method for wooden specimens and construction elements.

2. SCOPE OF THE EXPERIMENTAL PROGRAM

The main test series of the reported study comprised time-of-flight measurements on small-sized clear wood specimens with a systematic variation of the angle between propagation direction and principal growth direction. The main test series was conducted with both, beech and spruce wood specimens.

In order to exemplarily apply the results to the location of ultrasound sources, one board-like timber specimen with structural dimensions has been investigated, too. The respective time-of-flight measurements should validate, whether the acoustic anisotropy data from small clear specimens can be transferred to timber in spite of longer flight-paths, a different geometry, uncertainty about the exact angle with respect to the growth directions R and T and the presence of smaller defects.
3. EXPERIMENTAL SET-UP

3.1 Investigated specimens

Following a similar test set-up for measurements on the angle dependent velocity of ultrasonic waves in [5, 10], the clear wood specimens of the main test series were of prismatic (nearly cubic) geometry and small-sized with dimensions (length l × width b × depth h): 30 mm × 30 mm × 15 mm for both spruce and beech. For the chosen specimen size, in connection with the sawing pattern of the boards further from pith, the year ring curvature is negligible. So in an approximation the cylindrical anisotropy of wood reduces to a rhombic orthotropy. The specimens were throughout free of any visible growth defects.

The specimens were cut from solid wood boards in such a manner, that the wider (30mm*30mm) faces of the parallelepipeds were co-planar to one of four selected planes, being the three principal planes of wood (LR, LT and RT) and a plane spanned by the L-vector and a vector within the RT-plane at 45° between R and T principal directions (in the following named L-R/T plane). Figure 1 shows the cylindrical anisotropy of sawn solid wood and a schematic representation of the four acoustically investigated planes.

For of each studied plane the angle between the longer (30mm) edges of the parallelepiped (i.e. the propagation directions of the transition measurements) and the respective growth directions were varied. The angle is defined as

Fig. 1: a) Schematic representation of the cylindrical anisotropy of sawn solid wood  
b) plot of the investigated planes of ultrasound wave propagation
the rotation angle of the material co-ordinate system vs. the two possible directions of the ultrasonic pulse transmission within the regarded planes. Measurements at 0° resp. 90° to the materials axes L, R or T are called on-axis measurements. Pulse transmission tests with specimens including angles unequal to 0° resp. 90° to the material axes are called off-axis measurements.

Schematic drawings of the small specimens are shown in the Figures 2a and b. There the vectors $\vec{x}$ and $\vec{y}$ are place holders for the material axes spanning the regarded principal plane. For example in the case of tests on a set of specimens within the RT plane, $\vec{x} = R$ and $\vec{y} = T$.

![Fig. 2: Sketches of the small cubic specimens and definitions of on- and off-axis measurements. The ultrasonic pulses are transmitted along the two directions “pd1” resp. “pd2” for each specimen. $\vec{x}$ and $\vec{y}$ represent the respective on-axis material co-ordinate directions](image)

For the planes LR, LT and L-R/T, including the longitudinal direction L, the off-axis angle $\varphi$ is defined by the angle between the L direction and the chosen propagation direction ("pd1" or "pd2"). For the measurements on specimens in the RT plane, the off-axis angle $\vartheta$ is defined as the angle between the respectively chosen pulse transmission direction ("pd1" resp. "pd2") and the orientation of the radial R axis. The chosen definitions for $\varphi$ and $\vartheta$ are depicted in Figure 3a and 3b.
Table 1 specifies the test program with the small clear specimens conducted so far.

The exemplary investigated board-like spruce specimen had the dimensions (length l × width b × depth h): 500 mm × 120 mm × 25 mm with length l parallel to fibre direction (see Fig. 4). Within the cross-section b×h, contrary to the small clear specimens, the annual ring curvature was non-negligible, resulting in angles of 0 to about 80° between depth direction h and radial growth direction. The normal density of the specimen was $\rho_{12} = 428$ kg/m$^3$, the mean year ring width 2.1 mm and the moisture content 8%. The specimen had been selected carefully in order to incorporate as few and as small visible defects as possible. However, compared to the small clear specimens, some defects such as minor small knots or fiber deviations were accepted.
Table 1: Test program for small clear specimens; given are the number of measurements and specimens per investigated plane, angle and species

<table>
<thead>
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</table>

| total number of small clear beech specimens: | 34 |
| total number of small clear spruce specimens: | 43 |
3.2 Performance of the measurements

The determination of the acoustic properties of the small clear specimens of spruce and beech was throughout performed by means of a pair of piezoelectric (US) transducers. For each single specimen an ultrasonic pulse synthesised by a generator unit was applied to the centre of one of the specimen’s surfaces (15 mm × 30 mm) by the piezoelectric transmitter. The receiver was positioned at the centre of the opposite surface, leading to a length of the assumed straight pulse path of 30 mm. In all test series the transmitter and the receiver were fixed to the surface by a hot melt adhesive. Thus, the adhesive served as a coupling agent, too. Figure 5 shows a schematic view of the experimental set-up for the small clear specimens. For each specimen three repetitive measurements (3 US pulses) per direction of pulse transmission were performed.
The measurements on the board-shaped specimen were performed as following. Ultrasonic pulses were applied to one of the wide faces of the specimen at deliberately chosen "source points" denoted by co-ordinates \((x^s, y^s)\). The ultrasound waves propagating through the specimen were then recorded by six spatially distributed ultrasonic receivers at fixed positions as shown in Fig. 6. The propagation of the pulse waves to each of the receivers (A - F) was assumed to follow a straight path with shortest distance between the source and the respective receiver. For each “source point” and each receiver, the distance and the respective angle enclosed with the longitudinal fibre direction was recorded.

Fig. 6: Test set-up and dimensions of the tests with the board-shaped spruce specimen
3.3 NDT Equipment

The employed generator unit (USG 20; Geotron Electronics), originally optimised for NDT measurements on concrete, produces high voltage pulses. The main frequencies of the pulses are between 20 kHz and 350 kHz and the duration of a single pulse is less than 1 ms. Simultaneously with applying a synthesised pulse, the apparatus gives out a trigger impulse, defining the zero point on a time axis.

The employed ultrasonic transducers (= transmitter and receivers) comprised two different types of piezoelectric converters, both types showing a multi-frequency characteristic with a main sensitivity between 20 and 150 kHz.

i ) The measurements on the small clear specimens were performed with an ultrasonic transmitter (UPG-D) and a receiver (UPE-D), both by Geotron Electronics. The diameter of the coupling surface of the transducers is 3 mm.

ii ) For the board-shaped specimen, the mentioned ultrasonic transmitter UPG-D was used. Differently now, six receivers of type VS150-M (vallen systeme) were employed. The size of the coupling surface of the receivers is 20 mm in diameter.

The received ultrasonic pulses were amplified by a broadband amplifier (AM 502; Tektronix) with a maximum amplification factor of 100 dB. The complete signals were recorded with a PC based transient recorder with 12 bit amplitude and 20 MHz time resolution.

3.3 Characterisation of signal-parameters

Due to the piezo-electric nature of the receivers, the ultrasonic pulses are detected as chronologically oscillating voltages with limited duration, being typical for events with burst-like disposal of energy.

The recorded signals were evaluated in the context of this paper exclusively for “time-of-flight” (TOF), defined as the time lag between the externally given zero time point $t_0$ and the on-set (begin) of the recorded signal. The definition of the signal on-set used for the signal evaluation in the context of this paper is dis-
discussed in detail in [2, 3 and 8]. Figure 7a shows a typical full length signal recorded in the performed tests; the derived signal parameter TOF is depicted in Fig. 7b. Trivial, but for sake of “clarity”, the velocity of the ultrasonic pulse was determined as $v = s / TOF$ where $s$ is the distance between transmitter and receiver.

Fig. 7: Graph of a typical signal

a) full signal  b) close-up with definition of signal parameter TOF

4 RESULTS OF WAVE VELOCITIES FROM SMALL CLEAR SPECIMENS

The results of ultrasonic pulse velocity measurements for planes including the longitudinal direction (LR, LT and L-R/T) are depicted in Figures 8a – c. In detail, Figure 8a shows the dependency of US velocity on off-axis angle $\varphi_{LR}$ within the principle LR-plane spanned by the longitudinal (L) and radial (R) di-
measurements. Analogously, Figure 8b gives the velocity-off-axis angle $v(\phi_{LT})$ relationship in the LT plane and Figure 8c contains the results for the L-R/T-plane. In each of the Figures 8a to 8c, the individual results for spruce are depicted as open symbols and filled symbols are used for beech. The plotted lines connect the mean values evaluated or a single data point in the case when only one result per angle exists.

From the Figures 8a-c the following observations can be stated:

- The acoustic anisotropy in the LR- and LT planes is quite similar, for both, spruce and beech. The difference between the maximum velocity values in L-direction and the minimum values in directions perpendicular to fiber direction is about 3500 m/s for beech and about 4500 m/s for spruce, i.e. the acoustic anisotropy in the LR-and LT-planes is considerably more expressed for the softwood spruce.

- In the L-R/T plane a reduced acoustic anisotropy can be observed as compared to the results of the principal LR- or LT-planes. The difference between maximum and minimum velocity values is about 4100 m/s for spruce and 3100 m/s for beech. In the velocity vs. angle curves it is obvious that for small angles to fibre direction (about $\phi = 15^\circ$) the span between spruce and beech is considerably smaller in comparison with the LR and LT-planes.

The results of measurements in the RT-plane of spruce and beech have already been reported in detail in [2, 3]; here only a short summary is given.

Figure 9 shows the results $v(\theta_{RT})$ within the RT-plane perpendicular to fiber direction. The plotted lines are representing approximation curves fitted to the measured data; in detail polynomials of 6th degree have been used.

The difference between the maximum and the minimum values of the ultrasonic wave velocity was about 500 m/s for both tested wood species and the maximum value was in both cases observed for $\theta_{RT} = 0$, i.e. for the R-direction.
Fig. 8: Ultrasonic wave velocities of wood from small clear specimens
   a) L-R plane  b) L-T plane  c) L-R/T plane
While the velocity of ultrasonic waves in beech is monotonically decreasing with rotating the direction of transmission from the radial to the tangential axis (i.e. with an increasing off-axis angle $\vartheta_{RT}$), the results for spruce behave different: At an off-axis angle of about 45° a local minimum of 1500 m/s occurs with subsequently increasing velocities towards the tangential direction. The velocity span between radial direction (about 2000 m/s) and the tangential direction (about 1700 m/s) is ca. 300 m/s, being 200 m/s smaller than the absolute span between R- and intermediate (45°-) direction. In [2, 3] the reason for the local velocity minimum value of soft wood at an angle of about 45° between R and T axis, i. e. the shear coupling effect in combination with a very low shear modulus of soft wood, is discussed in detail.

5 WAVE VELOCITIES FROM LARGE SPRUCE TIMBER SPECIMEN

As lined out in chap. 3, for each deliberately chosen “source point” the applied US pulse was received by 6 spatially distributed receivers. This, in general delivers 6 different wave paths, each with different length and angle to fiber direction and hence 6 different velocity values are obtained. The results for the angle dependent wave velocities evaluated from 34 “source points” (i.e. $34 \times 6 = 204$ wave velocities) are shown in Figure 10a all together with a fitted approximation curve (polynomial of 6th degree). For purpose of easier comparati-
son, the L-R/T-curve from small clear specimens is additionally plotted in Fig. 10a.

In order to assess the influence of the different specimens’ scales and geometries, the results for the velocity measurements of both specimen types (small clear and board-shaped timber) are plotted in the Figures 10a and 10b. In Fig. 10b the $v(\varphi)$-curves for the LR, LT and L-R/T planes are given as line-connected symbols, representing the respective mean values per discrete angle values. From the synopsis of Figs. 10a and b it is obvious that the velocity dependency in the L-R/T plane is in best accordance with the results of the timber specimen measurements. When regarding the fact, that for the timber specimen the propagation component perpendicular to fiber direction is in general somewhere between the radial and the tangential direction, the best match of timber results with the L-R/T curve for small clear specimens can be understood qualitatively.

![Diagram](image)

Fig. 10: Angle dependency of the ultrasonic pulse velocity
a) obtained from the board-shaped specimen
b) obtained from the small clear specimens
6 DETAILS TO THE LOCATING METHOD

The algorithm to calculate the co-ordinates of US pulse sources is based upon a minimisation function, where calculated runtimes $t^{\text{calc}}$ for assumed source locations $(x^\text{test}, y^\text{test})$ are compared with runtimes $t^{\text{emp}}$ estimated by evaluating the signals recorded during the experiment. The minimisation of the Residual function $R^2$, given by Eq. (1) describing the summation of squared and weighted differences between the calculated and the measured (relative) runtimes yields the estimated source location $(x^{\text{source}}, y^{\text{source}})$.

$$R^2(x^{\text{test}}, y^{\text{test}}) = \sum_{n} \left( \frac{(t^{\text{emp}}_n - t^{\text{emp}}_0) - (t^{\text{calc}}_n(x^{\text{test}}, y^{\text{test}}) - t^{\text{calc}}_0)}{t^{\text{emp}}_n} \right)^2$$  \hspace{1cm} (1)

Time $t^{\text{calc}}_0$ represents the shortest of the N calculated runtimes, where N gives the number of the used receivers. $t^{\text{emp}}_0$ is the shortest of the N measured runtimes. Time $t^{\text{calc}}_n$ is calculated from the measured data and the geometry of the experimental set-up.

$$t^{\text{calc}}_n(x^{\text{test}}, y^{\text{test}}) = \frac{\ell_n}{v(\varphi)}$$  \hspace{1cm} (2)

where

$n = \text{index for the investigated receiver at the surface of the specimen},$

$$\ell_n = \sqrt{(x^{\text{receiver}}_n - x^{\text{test}})^2 + (y^{\text{receiver}}_n - y^{\text{test}})^2}$$

and

$v(\varphi) = \text{anisotropic US pulse velocity depending on angle } \varphi \text{ of direction of wave propagation. As a result of the velocity measurements the approximation function given in Fig. 10a for the timber specimens was used.}$

$$\text{Min}(R^2) \Rightarrow (x^{\text{source}}, y^{\text{source}})$$  \hspace{1cm} (3)
6.1 Effect of neglected or considered acoustic anisotropy

From qualitative considerations it is obvious that different assumptions on the acoustic anisotropy of wood will lead to pronouncedly different results of pulse source location. In a thorough analysis of the topic, to be analysed in a separate paper, several assumptions should be evaluated and compared, including the wave velocity results of small clear specimens (LR, LT and L-R/T), the analytical v(ϕ)-curve due to plane-wave approximation [5, 10] and empirical results from different literature sources. In the frame of the reported ongoing study, two simple assumptions have been tested, being the extreme cases of isotropic behaviour and anisotropic behaviour due to empirical velocity measurements at the exemplary board-like timber specimen.

In order to quantify the dimension of the locating error induced by the use of isotropic pulse velocities, the following cases were checked:

A1: \( v_{A1} = \text{const.} = 6089 \text{ m/s} = \text{maximum of measured velocity data} \)
A2: \( v_{A2} = \text{const.} = 1640 \text{ m/s} = \text{minimum of measured velocity data} \)
A3: \( v_{A3} = \text{const.} = 3865 \text{ m/s} = \text{mean value of } v_{A1} \text{ and } v_{A2} \)

Comparing the predetermined co-ordinates of the US pulse sources with the calculated co-ordinates enables quantifying the locating error, being the difference between the applied and the estimated US pulse source co-ordinates. In Table 2, the mean values and the standard deviations of the location error for the assumptions A1 to A3 are given separately for the direction perpendicular (x co-ordinates) and parallel (y co-ordinates) to the longitudinal fibre direction.

The best possible isotropic assumption for the velocity of ultrasonic pulse waves was found to be at the maximum measured velocity of about 6100 m/s. This value for the best isotropic assumption led to mean differences between the estimated and the applied co-ordinates of \( \Delta x = 32 \text{ mm} \) perpendicular and of \( \Delta y = 29 \text{ mm} \) parallel to the longitudinal direction.
Table 2: mean values and standard deviations for the differences between applied and numerically estimated NDT source co-ordinates, neglecting acoustic anisotropy properties

<table>
<thead>
<tr>
<th>Differences between applied and estimated US pulse source co-ordinates</th>
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<tr>
<td>a) <strong>perpendicular</strong> to the longitudinal fibre direction (the specimen’s x-axis)</td>
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<tr>
<td>assumed ( v ) [m/s]:</td>
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<td>mean value [mm]:</td>
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<td>standard deviation [mm]:</td>
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<td>b) <strong>parallel</strong> to the longitudinal fibre direction (the specimen’s y-axis)</td>
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<tr>
<td>assumed ( v ) [m/s]:</td>
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<tr>
<td>mean value [mm]:</td>
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<td>standard deviation [mm]:</td>
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For a first preliminary evaluation of anisotropy influence on the amount of location error, the velocity results of the board-shaped timber specimen were approximated by a fitted polynomial curve without any model assumption (see Fig. 11). In Table 3, the achieved locating “accuracy” by using the empirical approximation curve to the measured velocity data is compared with the accuracy that was achieved by using the best isotropic fit for the US pulse velocity. As anticipated, the consideration of empirically determined anisotropy leads to a much better accordance between the predetermined and the calculated US pulse source locations than the isotropic assumption for the velocity. The mean difference perpendicular to the longitudinal fibre direction was reduced from \( \Delta x = 32 \) mm to \( \Delta x = 4 \) mm and for the direction parallel to the longitudinal fibre direction the mean difference was reduced from \( \Delta y = 29 \) mm to then \( \Delta y = 6 \) mm.
Figure 11: Measured data and two assumed functions for the angle dependent velocity of ultrasonic (pulse) waves propagating in the anisotropic solid, i.e. the board-shaped specimen of spruce.

Figure 12a shows the results of the located US pulse sources with using an isotropic assumption for the ultrasonic wave velocity (here mean velocity of 3685 m/s) was employed. In figure 12b, the results of the located ultrasonic pulse sources for the fitted approximation function based on empirical anisotropy measurements are shown. Table 3 gives a comparison of the location errors of the isotropic and anisotropic case.

Table 3: mean values and standard deviations for the differences between applied and numerically estimated US pulse source co-ordinates, considering acoustic anisotropy properties

| Differences between applied and estimated co-ordinates of US pulse sources | a) **perpendicular** to the longitudinal fibre direction, i.e. the x-axis of the specimen |
|---|---|---|
| assumed v [m/s]: | value of best isotropic fit | fitted curve to the measured anisotropic velocity data |
| mean value [mm]: | 32 | 4 |
| standard deviation [mm]: | 17 | 3 |

| b) **parallel** to the longitudinal fibre direction, i.e. the y-axis of the specimen |
|---|---|---|
| assumed v [m/s]: | value of best isotropic fit | fitted curve to the measured anisotropic velocity data |
| mean value [mm]: | 29 | 6 |
| standard deviation [mm]: | 23 | 4 |
Measurements of Acoustic Anisotropy of Soft and Hard Wood

Fig. 12: Applied and estimated co-ordinates of ultrasonic pulse sources on a board-shaped specimen of spruce resulting from
a) isotropic assumption $v = \text{const.}$ for the ultrasonic wave velocity
b) best fit curve fitted to the measured velocity data

7. CONCLUSIONS

The presented preliminary results of the ongoing study on the impact of acoustic anisotropy on location of ultrasound pulse sources can be summarised as following

- The US pulse transmission method on small clear specimens is an apt method in order to establish a database for acoustic anisotropy. By the reported results the differences between hard wood (beech) and soft wood (spruce) species could be reproduced.

- The velocity measurements on an exemplary board-like timber specimen (made of spruce) were in good accordance with the results for small clear spruce specimens in the L-R/T, i.e. an intermediate plane
spanned by the longitudinal fibre direction and a perpendicular direction within the RT-plane at an angle of 45° to the principal axes.

- The numerical location procedure of ultrasound pulse sources applied to the surface of the exemplary studied timber specimen yielded reasonably low error values, when an empirical approximation function based on velocity measurements was applied.

ACKNOWLEDGEMENTS

The continuous good co-operation between Wood and Timber Construction Department of Otto-Graf-Institute and Laboratoire de Rheologie du Bois de Bordeaux (LRBB) is gratefully acknowledged. Special thanks are indebted to the director of LRBB, Dr. Patrick Castera, in this context, for his repeated yearly favor of French translation of our OGI-paper abstracts.

The financial support of German Science Community (DFG) via grant to Sonderforschungsbereich 381 “Characterisation of damage evolution in composite materials using non-destructive test methods” and hereby to sub-project A8 “Damage and NDT of the natural fibre composite material wood” is gratefully acknowledged.

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