PROCESSING AND IMAGING SIMULATED ULTRASONIC B-SCANS OF CONCRETE

VERARBEITUNG UND ABBILDUNG SIMULIERTER ULTRASCHALL B-SCANS VON BETON

LE TRAITEMENT ET LA FORMATION IMAGE DES B-SCANS ULTRASONIQUE SIMULE EN BETON

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SUMMARY

Simulating ultrasonic experiments on concrete and processing the data gained in these simulations is a useful tool in nondestructive testing. The supplied data enables us to get a better understanding of the different scattering processes in the model. These synthetic datasets are also applicable for testing different processing schemes. Here, two new features are introduced for the processing of the data before or after using a SAFT-algorithm [Langenberg et al., 1993] for imaging. They both take into account that the main reflections get disturbed by the scattering effects of small air-inclusions in concrete. There are also some resuming thoughts on improvement of imaging algorithms by extending the experiments and the data accumulation to offset dependent data.

ZUSAMMENFASSUNG

Die Simulation von Ultraschallexperimenten an Beton und das Verarbeiten der dadurch gewonnenen Daten ist in der zerstörungsfreien Prüfung äusserst nützlich. Es ergibt sich dadurch die Möglichkeit einen Einblick in die Streuvorgänge im Modell zu bekommen. Desweiteren eignen sich die synthetischen Daten zum Testen verschiedener Processing-Verfahren. Das Abbilden (Imaging) wird mit Hilfe des SAFT-Algorithmus [Langenberg et al., 1993] durchgeführt. In diesem Artikel sollen zwei neue Features vorgestellt werden, welche berücksichtigen, daß die Daten durch die vielen kleinen

RESUME

La simulation des expériences ultrasoniques sur béton et le traitement des données gagnées dans ses simulations est en tres utile essai nondestructif. Il y a la possibilité pour obtenir une perspicacité de des procédés de dispersion dans le modèle et aussi il y a la possibilité de recevoir un ensemble de données relativement facile d'essai pour différents arrangements de traitement. Voici deux nouveaux dispositifs présentés pour le prétraitement des données avant ou après l'utilisation d'un SAFT-algorithme [Langenberg et al., 1993] pour la formation image. Ils tous les deux tiennent compte que les réflexions principales obtiennent troublées par les effets de dispersion des petites inclusions d'air en béton. Également il y a quelques pensées de reprioe sur l'amélioration des formations images-algorithme en étendant les expériences et l'accumulation aux données excentrer-dépendantes.

KEYWORDS: synthetic ultrasonic experiments, imaging, processing

INTRODUCTION

Data of ultrasonic experiments on concrete have a very complex and problematic structure. Even extremely accurate FD-simulations cannot represent all the effects to be taken into account when dealing with such a complicated medium. The output of such simulations lacks the influence of the receiver characteristics (coupling, resonance frequencies, diameter). Beside the scattering noise produced by small air inclusions, there is no experimental noise. Real experiments always are 3D-experiments where out of plane reflections occur. Of course, this side effect is not considered when working with 2D-simulations. Even the data of a simple synthetic 2D-configuration is hard to handle when one creates an image of the inner structure of the investigated concrete in order to detect cracks and flaws in a given depth. Because real data are always of minor quality compared to synthetic data, our synthetic data, strongly dominated by air
inclusion scattering are a good experimental environment to test processing- and imaging-algorithms before applying them to real ultrasonic B-Scans.

Fig. 1: A Model with 1% air-inclusions and its B-Scan recorded at the surface.

SIMULATION AND PROCESSING

To test the new processing features the same simulation models and the same synthetic datasets as in the last article [Burr et al., 1997] were used. The concrete model existed of a two-dimensional 10cm*10cm block with ellipses of different sizes, which represent the stuffing material and with very small ellipses which represent the air-inclusions that are responsible for the high scattering noise in the data. All ellipses are randomly placed and orientated. The percentage of the air-inclusion is varied from one percent to four percent to gain datasets with a different degree of difficulty. A small flaw with a fractal shape is placed horizontally in three centimeters depth and with a lateral extension of two centimeters. In the model with four percent air-inclusions the flaw is placed a bit diagonally.

A plane wave is propagated through the medium and the reflections are recorded at every of the 500 gridpoints at the surface of the model (Fig. 1 and 2). The received B-Scans are processed in different ways.
The imaging process is based on the SAFT-algorithm which here presupposes a homogenous velocity of 4000 m/s. Because of the stuffing and air-inclusions concrete is treated as a random medium. Therefore, a randomizer is brought into action while building a model. The random medium gives rise to talk about an effective velocity rather than a homogenous velocity.

The image is more robust and interpretation is easier and more obvious when one calculates the envelope after imaging. The envelope or instantaneous amplitude represents the absolute value of the analytical or complex signal [Buttkus, 1991], respectively. This processing step was first used in geophysics for yielding an image from seismic reflection experiments in strongly fractured cristalline rocks [Simon, 1998]. The Envelope procedure avoids destructive interference of amplitudes due to imperfect velocity estimations applied in the imaging algorithm.

A coherency criteria is also implemented into the imaging algorithm. It is called semblance [Taner and Köhler, 1996] and corresponds to the normalized ratio of the output to the input energy. An exact phase alignment along the summed trajectory yields the maximum whereas white noise the minimum value. The values are in the interval $0 \leq S \leq 1$. 
After the preprocessing the SAFT-algorithm generates an image of the dataset. Every imagepoint is associated with the sum of amplitudes lying on a parabolic shaped trajectory in the data space (Using a point source would result in summing along a diffraction hyperbola.).

With respect to the results of [Burr et al., 1997] the imaging algorithm has been improved by implementing a weighing factor which considers two physical aspects. The first one is called 'geometrical spreading factor' or 'spherical divergence', respectively. The intensity, i.e. the quantity of energy that flows through a unit area normal to the direction of wave propagation in unit time, is inversely proportional to the distance between source and wavefront. therefore,
we can write \( \frac{I_2}{I_1} = \left( \frac{R_2}{R_1} \right)^M \) where \( M = 0, 1, \) or 2 according as the wave is plane, cylindrical, or spherical. \( I_1, I_2, R_1 \) and \( R_2 \) are the intensities and radii of the wavefront at time 1 and 2. The second aspect takes into account that an incident wave forms an angle \( \phi \) with the normal of the surface and receiver, respectively. This obliquity factor considers that only the cosine of the amplitude has been registered.

Fig. 4: Different images out of the B-Scan of concrete with 4% air inclusions.

RESULTS
In the first simulation there was used one percent air-inclusions in the concrete model (Fig. 1). The second example is a bit more complicated with four percent air-inclusions (Fig. 2). Both new features were tested individually at the two synthetic datasets. The data resulting from these different processing schemes is imaged with the SAFT-algorithm and compared with the unprocessed example. So we gain an envelope-image, a semblance-image and an unprocessed image for every synthetic dataset.

In figure 3 the semblance-image brings a small improvement for the detection of the flaw. The lateral extend of the flaw is better than in the unprocessed image but not as good as in the envelope-image. Also the image of the flaw is shifted to the left in comparison with the model. Every artifact in the unprocessed image has a counterpart in the semblance-image. So the depressing of multiple reflections and other artifacts, for example due to shear and surface waves or sizeeffects, is not possible with this processing scheme. But it is expected to bring better results if the signal to noise ratio is lower, for example in medias with weak scattering inhomogenities.

As it is seen in Fig. 3 and Fig. 4, there is a much higher dynamic range of the amplitudes that survives the imaging process, when the envelopes are formed after the use of the SAFT-algorithm. In comparison with the other images, the envelope-image is the best without artifacts, the flaw is always placed in the right area and its inclination in comparison to the surface is correct. In the 4% model (Fig. 2) the flaw was a bit more bent and in the 4% envelope-image (Fig. 4) this difference is seen very well. The envelope-images have also the highest contrast between flaw and environment and the best lateral dissolution. The only thing to criticize is the unsharp profile of the flaw, but the upper edge in the image is always representing the correct depth.

In every image there is an artifact latarally under the right side of the image of the flaw. This is due to the energy reflected at the flaw not directly back to the surface but with another reflection at the right side of the model. Maybe the imaging-algorithm should take these reflections into account and when the
parabola reaches the edge of the data-space, the summation shouldn’t be stopped. There is the possibility for image-points near the sides of the image-array to mirror the branch of the parabola reaching the edge back into the data-space and to continue the summation. This would be an easy task for such simple shaped images like in our example and this step would eliminate the last artifact out of the envelope-image.

CONCLUSIONS

Because there is further a unsatisfactorily yield due to reduced resolution there should be thought about a recording of more complex datasets then only B-Scans. If there are some other possibilitys to illuminate the investigated body not only with a transducer but with different source-receiver-constellations the dataset would have more information but it would be also more complex to image and more expensive to measure.

In these simulations there were used perfect pointlike receivers. In real experiments there is always the problem: Take a receiver with a small diameter and it has a reduced sensitivity and is only available for lower frequencies. Take a receiver with a long diameter than you have a higher sensitivity but you average out all the informations in the high frequencies. It would be a large improvement if averaging would only be done by the processing- or imaging-algorithm and not by the receiver.

So the main problem in ultrasonics is how to get as much information as possible into the data and how to extract the necessary informations for the image. These two processing schemes introduced in this article are a small step in the right direction. Mainly the formation of envelopes as a processing step gives really a rough estimate of the orientation and placement of the flaw.

The used programe and its features has also being designed for interactive use under Windows and this version will be necessary as soon as real
experiments on concrete will take place in the innovative research group of the FMPA.

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REFERENCES


Pull-out tests with short embedment length were carried out with 7.5 mm thick aramid composite bars in 60 MPa concrete. Standard climate, natural weathering, and thermal cycles between -20 and +40°C were applied during about one year. Results are reported with respect to maximum bond stress, bond stress-slip relation and bond creep.
un an. Les résultats concernant la résistance de la liaison d’adhérence, la relation contrainte-glissement et fluage de l’adhérence sont présentés.

**KEYWORDS**: Prestressed concrete, non-ferrous reinforcement, ceramid, bond, temperature cycles, bond creep

1. **MOTIVE**

Reinforced and prestressed concrete has proven to be durable in most environments. However, there are some harsh conditions where reinforcing and prestressing steel may suffer from corrosion. Such situations may occur in marine environment with splash water, along traffic areas with deicing salt, or in thin walled elements with rapid carbonation. Under those circumstances fibre polymer composites can replace steel and make a reinforced concrete structure more durable. Besides very high strength of the fibers and corrosion resistance of fibres and polymer there is some concern about the thermal mismatch of concrete and the composite bar. Concrete has a coefficient of thermal expansion which is about $10 \cdot 10^{-6} \text{K}^{-1}$ and the aramid composite has a coefficient of thermal expansions of $-2 \cdot 10^{-6} \text{K}^{-1}$ along the bar and $70 \cdot 10^{-6} \text{K}^{-1}$ normal to the bar. This means that the bar will more expand and contract in transverse direction and less in longitudinal direction than concrete. These differential movements cause stresses and even splitting cracks which may impair bond. That this can happen has already been experienced in [De Sitter, Tolman, 1995] and predicted in [Matthys, S. et al., 1996]. However, more experimental data is necessary in order to quantify the effect of temperature cycles, time and exposure condition on bond. This paper reports on a series of experimental investigations and will discuss the results. For detailed information see [Gollas, 1998].
2. TESTING PROGRAMME AND METHODS

To study the bond behaviour of aramid in concrete 150 mm cubes were cast with an aramid bar embedded in the center over a length of 30 mm. The concrete was a normal weight concrete with a mean cube strength of 55 MPa. The aramid consisted of 200,000 filaments with a diameter of 12 µm embedded in vinylester resin. Matrix and fibres take each about 50% of the bar. The outer circular dimensions of the bar are about 7.5 mm diameter. The surface of the bar was covered with quartzitic sand of 125-250 µm grain size. The tensile strength of the bar amounted to 1520 MPa, Young’s modulus was 60640 MPa, and the ultimate strain 2.5%. Fig 1 shows the cube with the aramid bars.

![Diagram of cube with aramid bar](image)

Fig. 1: Cube with aramid bar used for pull-out tests

Three exposure conditions were applied: standard climate with 20°C and 65% RH, natural exposure on the roof of the laboratory building (460 m NN), and temperature cycles between -20 and +40°C (1 cycle in 8 hours). Some specimens were continuously loaded, others not. After a certain time of exposure pull-out tests were performed.

The short term pull-out test equipment is shown in Fig. 2. Relative displacements between concrete and aramid were measured at the loaded and non-loaded side of the specimen.
Several specimens were kept under load while exposed either to standard environment or thermal cycles. Fig. 3 shows the long term loading equipment which consists of a series of plate springs and displacement gauges.

This equipment allowed to measure the relative displacement as function of time, i. e. bond creep. After sustained loading, the specimens were subjected to short term loading in order to measure the residual bond stress-displacement relation.
3. TEST RESULTS

Test results of short term loading are mainly received as pull-out force vs. displacement relation. Fig. 4 shows the bond stress as function of relative displacement.

Bond stress is defined as pull-out force divided by embedment length times perimeter of the aramid bar. The embedment length was 30 mm, the bar diameter 7.5 mm. The specimens A1 and A4 were stored in natural weather not sheltered from rain during one year. Specimens K3 and K10 were stored in a room with standard climate 20°C/65% RH until the age of 187 days. Finally, specimens K7
and K12 were subject to 350 temperature cycles prior to testing. It should be mentioned that these six individual results were selected such that each couple represents the highest and the lowest maximum bond stress of one exposure condition. The results which are not shown were lying between them.

![Graph showing bond stress vs. relative displacement as result of pull-out test](image)

**Fig. 4:** *Bond stress vs. relative displacement as result of pull-out test*

The concrete compressive strength was measured on three 150 mm cubes which were stored in the same way. The results were 61.4 MPa for cubes in natural weather, 59.0 MPa in standard climate, and 61.5 MPa after the temperature cycles. These results show no significant difference between the storage conditions. This means that concrete strength cannot explain the considerable differences of the lines of Fig. 4.

All lines in Fig. 4 show a linear part at low stresses which is followed by displacement hardening until the maximum stress is reached, and finally a gradual decay of stress with increasing slip. The slope of the linear part scatters by a
factor of about two, however there is no systematic influence of the exposure condition. The maximum bond stress and the slip which occurs at maximum stress as measured at the free end of the bar is given in Table 1.

Table 1: Maximum bond stress $\tau_{\text{max}}$ and accompanying slip $\Delta$

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Exposure condition</th>
<th>$\tau_{\text{max}}$ MPa</th>
<th>$\Delta$ mm</th>
<th>Mean values</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1, A4</td>
<td>Natural weather</td>
<td>11.1, 13.6</td>
<td>0.56, 0.62</td>
<td>12.4, 0.59</td>
</tr>
<tr>
<td>K3, K10</td>
<td>Standard climate</td>
<td>9.8, 14.0</td>
<td>0.50, 0.88</td>
<td>11.9, 0.69</td>
</tr>
<tr>
<td>K7, K12</td>
<td>Thermal cycles</td>
<td>8.1, 6.1</td>
<td>0.62, 0.64</td>
<td>7.1, 0.63</td>
</tr>
</tbody>
</table>

The mean values of $\tau_{\text{max}}$ and $\Delta$ show that natural weather caused the largest value of $\tau_{\text{max}}$ and the smallest of $\Delta$, i.e. the pull-out resistance is greatest. Standard climate is a little less, and thermal cycles clearly reduced the pull-out resistance to half compared to natural weathering. Since compressive strength of concrete is the same, it must be concluded that the mismatch of thermal expansion of aramid and concrete has had a rather detrimental effect. It can also be concluded that thermal cycling between -20 and +40°C is much more detrimental than natural weathering. In contrary, natural weathering over one year has caused an improvement of bond which is probably due to the moderate climate with high humidity (about 70% RH) and not extreme temperatures.

Sustained loading has been performed in standard climate and under thermal cycles. Table 2 shows the loading regime of the specimens. To determine the strength level of the specimen there were companion specimens which were loaded after a certain time of exposure in standard climate (159 or 74 days) or under thermal cycles (222 to 465 cycles).

Table 2: Loading regime of sustained loading
<table>
<thead>
<tr>
<th>Specimen</th>
<th>$\tau_{\text{max}}$ [MPa]</th>
<th>Stress level $\tau_{\text{sust}}/\tau_{\text{max}}$</th>
<th>Sustained stress MPa</th>
<th>Exposure during sustained loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>A15</td>
<td>10.2</td>
<td>85</td>
<td>8.67</td>
<td>Standard climate</td>
</tr>
<tr>
<td>A6</td>
<td>10.2</td>
<td>85</td>
<td>8.67</td>
<td></td>
</tr>
<tr>
<td>A7</td>
<td>10.2</td>
<td>75</td>
<td>7.65</td>
<td></td>
</tr>
<tr>
<td>A5</td>
<td>10.2</td>
<td>75</td>
<td>7.65</td>
<td></td>
</tr>
<tr>
<td>K4</td>
<td>10.6</td>
<td>80</td>
<td>7.65</td>
<td></td>
</tr>
<tr>
<td>K1</td>
<td>9.2</td>
<td>80</td>
<td>7.36</td>
<td>Thermal cycles</td>
</tr>
<tr>
<td>K9</td>
<td>9.2</td>
<td>80</td>
<td>7.36</td>
<td></td>
</tr>
<tr>
<td>K11</td>
<td>9.2</td>
<td>80</td>
<td>7.36</td>
<td></td>
</tr>
<tr>
<td>A22</td>
<td>8.5</td>
<td>50</td>
<td>4.25</td>
<td></td>
</tr>
<tr>
<td>A27</td>
<td>8.5</td>
<td>50</td>
<td>4.25</td>
<td></td>
</tr>
<tr>
<td>A25</td>
<td>8.5</td>
<td>30</td>
<td>2.55</td>
<td></td>
</tr>
<tr>
<td>A26</td>
<td>8.5</td>
<td>30</td>
<td>2.55</td>
<td></td>
</tr>
</tbody>
</table>

Short term loading led to $\tau_{\text{max}}$ in Table 2. The sustained stress is 30 to 85% of $\tau_{\text{max}}$. The load duration in the standard climate was confirmed to 10 months, while the specimens subject to thermal cycles were removed when the slip increased drastically. Fig. 5 shows the results of sustained pull-out tests in standard climate in double logarithmic scale.

The slip is measured on the loaded side of the bar and contains the initial displacement due to loading and the time dependent displacements due to sustained loading. It should be noted that there was no bond failure at the end of test (max. duration 9288 hours).
Fig. 5:  Results of sustained pull-out tests in standard climate

Fig. 6:  Results of sustained pull-out tests with simultaneous thermal cycles.
Fig. 6. shows the slip-time relation of the specimens which were also subject to thermal cycles. It is obvious that the initial slip depends clearly on the stress level and that a large slip increase occurs as function of time. All specimens failed (failure is defined as showing tertiary creep) after a certain time depending on the stress level: at the highest level after 200 h, at the lowest level after about 1000 h, or in terms of thermal cycles, after 25 cycles and 125 cycles, resp.

The specimens which were loaded in standard climate were removed from the sustained loading rig and then loaded in a short-term test in order to determine the residual strength. Fig. 7 shows the results of five specimens. Four of them are rather similar while specimen A15 shows an unexplainable high bond strength.

Fig. 7: Residual bond stress vs. slip relation of specimens from standard climate

The four similar specimens showed a residual bond strength of 7.8 to 10.6 MPa without a significant dependance on the level of previous sustained loading. However there is some degradation with respect to the strength before loading (10.2 MPa acc. to Table 2.). The specimens which were subject to thermal cycles were not tested for residual strength since they have failed already under sustained loading.

4. DISCUSSION OF RESULTS
4.1 General

Bond of prestressing bars is essential for pretensioned members with direct bond. Good bond means a short anchorage length (transmission length) and, thus, full loading capacity also at the end of a structural member. If bond is affected by environmental effects means that the transmission length increases. It could also mean in a most detrimental situation that splitting cracks occur and bond is reduced to a small percentage of the original value.

4.2 Short-term loading after various exposure

To model bond the bond-slip relation was introduced by [REHM, 1961] which is measured on a short embedment length and which can be used in the appropriate differential equation. To handle the differential equation it is advantageous to describe the bond slip equation by an equation which can be integrated in a closed form. [NOAKOWSKI, 1978] used a power function which has further been evaluated by [KRIPS, 1984] and [BRUGGELING, 1991]. The function reads

\[ \tau = C \cdot \Delta^N \]  

with \( \tau \) the bond stress as averaged over the short embedment length of about 3 times the diameter of the bar, \( \Delta \) the slip, and \( C \) and \( N \) constants. To determine the constants \( C \) and \( N \), at least two points of the measured \( \tau-\Delta \) curve are selected. Usually the maximum bond stress and the accompanying slip are used and a point between zero and maximum bond stress. After having evaluated the curves of Fig. 4 the results of Table 3 were received.

Compared to reinforcing bars in normal strength concrete, the values of \( C \) and \( N \) are rather high which means that the bars with sand covered surface have good bond in the 55 MPa concrete.

Table 3: Material constants \( C \) and \( N \) for results of Fig. 4
It can also be seen that thermal cycles have decreased bond which is mainly manifested by the multiplying factor $C$. It should be noted that eq. (1) is fitting only the ascending branch of the $\tau$-$\Delta$ curve.

A good indication of bond is the transmission length of a pretensioned bar in concrete. Making use of eq. (1) the transmission length is given by the following equation [BRUGGELING, 1991]

$$
I_{pt} = \left[ \frac{1 + N}{2} \right] \frac{d_p}{4} \frac{\sigma_{pr}}{C} \frac{\sigma_{po}}{E_p} \left[ 1 + \frac{1}{N} \right], \quad \frac{2E_p}{(1-N)\sigma_{pr}}
$$

(2)

with $d_p = \text{diameter of pretensioned bar}$, $\sigma_{pr} = \text{pretensioning stress in the bar}$, $\sigma_{po} = \text{pretensioning stress after demoulding}$, $E_p = \text{Young’s modulus of bar}$, and $C$ and $N$ material constants. Inserting the values of the aramid bar into eq. (2) the transmission length takes the values as given in column 5 of Table 3. There, it can be seen that the rather short transmission length is considerably increased due to thermal cycles but there is only little effect due to natural weather.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Exposure condition</th>
<th>$C^{1)}$</th>
<th>$N^{1)}$</th>
<th>$l_{pt} \text{ [mm]}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>Natural weather</td>
<td>13.1</td>
<td>0.28</td>
<td>208</td>
</tr>
<tr>
<td>A4</td>
<td></td>
<td>15.5</td>
<td>0.27</td>
<td>179</td>
</tr>
<tr>
<td>K3</td>
<td>Standard climate</td>
<td>12.1</td>
<td>0.28</td>
<td>221</td>
</tr>
<tr>
<td>K10</td>
<td></td>
<td>14.7</td>
<td>0.36</td>
<td>224</td>
</tr>
<tr>
<td>K7</td>
<td>Thermal cycles</td>
<td>9.5</td>
<td>0.32</td>
<td>287</td>
</tr>
<tr>
<td>K12</td>
<td></td>
<td>7.2</td>
<td>0.39</td>
<td>400</td>
</tr>
</tbody>
</table>

1) Constants apply for $\tau$ in MPa and $\Delta$ in mm.
4.3 Bond creep

The increase of slip in a pull-out test at sustained loading is called bond creep. It can be described by the power function [FRANKE, 1976]

$$\varphi(t) = (1 + 10t)^a - 1$$ (3)

with $\varphi(t)$ the creep coefficient which is the ratio of time dependent slip and short-term slip at loading, $t$ the time in hours, and "$a$" a constant. If the curves of Fig. 5 are evaluated the values of Table 4 are received. There is a tendency that a lower stress ratio (0.75) loads to larger creep than a high stress ratio (0.85).

Table 4: Constant $a$ for results of Fig. 5

<table>
<thead>
<tr>
<th>Specimen</th>
<th>A5</th>
<th>A6</th>
<th>A7</th>
<th>A15</th>
<th>K4</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Constant $a$</td>
<td>0.035</td>
<td>0.013</td>
<td>0.030</td>
<td>0.008</td>
<td>0.051</td>
<td>0.027</td>
</tr>
<tr>
<td>$\tau_{sust}/\tau_{max}$</td>
<td>0.75</td>
<td>0.85</td>
<td>0.75</td>
<td>0.85</td>
<td>0.80</td>
<td>-</td>
</tr>
</tbody>
</table>

However, the intermediate stress ratio (0.80) does not fit into this row. But this may be due to the different pre-exposure (K4 only 74 days compared to all other specimens with 159 days).

Evaluating eq. (3) with $a = 0.027$ leads to a creep coefficient after 15 years ($10^5$ h) equal to 0.45, i.e. the total slip is 1.45 times the instant slip at the time of loading.

5. CONCLUSIONS

The investigation leads to several conclusions:

- bond between a sand covered aramid composite bar in concrete shows a bond strength of about 12 MPa
• natural weathering during one year did not impair bond strength
• 350 thermal cycles between -20 and +40°C reduced bond strength to about 7 MPa
• transmission length as determined from bond stress-slip relation increased by about 50% after 350 thermal cycles
• bond creep coefficient amounts to 0.45 in standard climate after 15 years
• thermal cycles between -20 and +40°C cause bond failure also at low stresses.

REFERENCES


SUMMARY

Permeability, diffusion and capillary absorption represent the main mechanisms for water and water vapour transport in concrete. Up to present the relationship between temperature and the individual transport mechanisms is unclear. In the present work the influence of temperature was therefore examined at 20, 50 and 80 °C. The results show that an increase of temperature is always accompanied by an acceleration of the transport processes. For this purpose, concretes with different quantities of acrylate dispersion, high performance concretes with different types of cement and aggregates, and the French BPR were examined and compared with a reference concrete and among themselves.

ZUSAMMENFASSUNG

unterschiedlichen Zementarten und Zuschlägen, sowie der französische BPR untersucht und mit einem Nullbeton bzw. untereinander verglichen.

RESUME

La perméabilité, la diffusion et l’absorption capillaire représentent les principaux mécanismes de transport d’eau et de vapeur d’éau dans le béton. Jusqu’à présent, la relation entre la température et les différents mécanismes n’est pas encore déterminée. Dans l’étude présentée, l’influence de la température a été examinée à 20, 50 et 80 °C. Les résultats montrent qu’une augmentation de température est toujours accompagnée d’une accélération des processus de transport. Les bétons examinés sont des bétons avec adjonction de différentes quantités de dispersion acrylate, des bétons à haute performance avec différents ciments et granulats, aussi que le BPR français. Les résultats pour les différents bétons sont comparés entre eux, aussi qu’avec un béton reference.

KEYWORDS: Permeability, diffusion, concrete, capillary absorption

1. INTRODUCTION

In the context of the BMBF Research project "hot water storage in high performance concrete tanks" the vapour behaviour of the concrete should be examined under simultaneous action of temperature and hydraulic pressure. The previous concept for the seasonal storage designated to line the concrete containers with an interior liner made of stainless steel with 0.50 mm thickness in order to prevent a moisture penetration of the thermal insulation. After the first storage tank in Rottweil was finished and the planning of the second one in Friedrichshafen was concluded, the following disadvantages of an interior liner made of stainless steel can be indicated: expensive and difficult to construct.

With a high performance concrete (HPC), higher values for the vapour diffusion resistance can be expected than for concrete with normal strength (normal concrete). Thus heat storages by HPC are conceivable without a stainless steel lining.
2. CONCRETE

The composition of the concretes investigated are shown in table 1.

Table 1: Composition of the examined concrete

<table>
<thead>
<tr>
<th>Mixture</th>
<th>w/c ratio</th>
<th>Cement</th>
<th>Content [kg/m³]</th>
<th>Aggregate</th>
<th>Fly-ash [kg/m³]</th>
<th>Additive</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mowi 45</td>
<td>0,45</td>
<td>CEM II 32,5 A-L</td>
<td>330</td>
<td>AB 32</td>
<td>FA 30</td>
<td>Mowilith, 45</td>
</tr>
<tr>
<td>Mowi 30</td>
<td>0,45</td>
<td>CEM I 32,5</td>
<td>330</td>
<td>AB 32</td>
<td>FA 30</td>
<td>Mowilith, 30</td>
</tr>
<tr>
<td>Ref. 35</td>
<td>0,45</td>
<td>CEM I 32,5</td>
<td>330</td>
<td>AB 32</td>
<td>FA 40</td>
<td>-</td>
</tr>
<tr>
<td>36</td>
<td>0,33</td>
<td>CEM I 42,5</td>
<td>400</td>
<td>AB 16</td>
<td>-</td>
<td>MS, FM</td>
</tr>
<tr>
<td>37</td>
<td>0,33</td>
<td>CEM I 32,5 NW/HS</td>
<td>400</td>
<td>AB 16</td>
<td>-</td>
<td>MS, FM</td>
</tr>
<tr>
<td>40</td>
<td>0,37</td>
<td>CEM II 32,5 A-L</td>
<td>360</td>
<td>AB 32</td>
<td>FA 40</td>
<td>MS, FM</td>
</tr>
<tr>
<td>41</td>
<td>0,37</td>
<td>CEM II 32,5 A-L</td>
<td>360</td>
<td>AB 16</td>
<td>FA 40</td>
<td>MS, FM</td>
</tr>
<tr>
<td>BPR</td>
<td>0,17</td>
<td>Powder</td>
<td>900</td>
<td>-</td>
<td>Plastic.</td>
<td>-</td>
</tr>
</tbody>
</table>

MS: MICRO SILICA, FM: SUPERPLASTICIZER

3. TESTING METHODS

3.1 Permeability

The permeability of the different concretes was tested with a special test cell made by the company Mess- and Feinwerktechnik Sommer (Schmidtheim), (see fig. 1).
This permeability measuring system was modified at the institute of construction materials, so that it was possible to test concrete disks with a thickness of only 3 cm at a water pressure to 10 bar (100 m water column). The system consists of the test cell, the automatic controller unit, as well as the connecting devices (PU pressure hoses).

**3.1.1 Specimens**

The specimens were stored up to the test under water. The disks are three centimetres thick and have a diameter of 15 cm. Both flat sites were polished and coated on the lateral surfaces with epoxy resin (Sikafloor 370, Sika Chemie GmbH, Stuttgart).

**3.2 Diffusion**

The diffusion tests were performed according to the standardised procedures in DIN 52615. Both methods, the “dry-cup“ and the “wet-cup procedure” were examined, although the “wet-cup method” is the most important. To achieve the correct humidity, the salt solutions according to table 2 were used.
Table 2: *Salt solutions or drying agents*

<table>
<thead>
<tr>
<th></th>
<th>20 °C</th>
<th>50 °C</th>
<th>80 °C</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 3 %</td>
<td>Silica gel</td>
<td>Phosphorpentoxide $\text{P}_2\text{O}_5$</td>
<td>Phosphorpentoxide $\text{P}_2\text{O}_5$</td>
</tr>
<tr>
<td>50 %</td>
<td>Natriumdicromate $\text{Na}_2\text{Cr}_2\text{O}_7 \cdot \text{H}_2\text{O}$</td>
<td>Natriumbromide NaBr</td>
<td>Natriumbromide NaBr</td>
</tr>
<tr>
<td>93 - 100 %</td>
<td>Amoniumdihydrogenphosphate</td>
<td>Amoniumdihydrogenphosphate</td>
<td>Amoniumdihydrogenphosphate</td>
</tr>
</tbody>
</table>

### 3.2.1 Specimens

In this work, specimens with a diameter of 10 cm and approx. 2 - 2.5 cm height were examined. Thinner specimens than the ones which are required by DIN, usually show larger diffusion coefficients, so that it can be assumed that the results, which were determined here, were considered to be on the safe side. When the specimens were installed, they were in moisture equilibrium.

### 3.3 Capillary absorption

Capillary absorption is dominant among the three mentioned transport mechanisms, because it causes the fastest transport of water. With the tests executed in this work, the uniaxial penetration behaviour by liquids, here water, into cylindrical concrete specimens was examined. Capillary absorption is described by the water absorption coefficient $A$. The basis of the procedure is DIN 52617: "Determination of the water absorption coefficient of building materials".
3.3.1 Specimens

The concrete specimens had a cylindrical form with a diameter of 15 cm and a height of approx. 12 cm. The CWA tests (capillary water absorption) were executed at three different temperatures, i.e. 20, 50 and 80°C.

First, the cylinders were dried at 60°C up to a constant weight. The temperature was intentionally limited at 60°C in order to exclude a crystal change in the concrete structure. After attaining the constant weight the lateral surface of the specimens were coated with transparent epoxy resin (Sika floor 370, Sika Chemie GmbH, Stuttgart).

4. INTERPRETATION OF TEST RESULTS

4.1 Permeability

The analysis of the results takes place with the following formula:

\[
k_w = \frac{V}{t} \cdot \frac{d}{A} \cdot \frac{1}{h_2 - h_1} \cdot \rho_w \cdot g \quad [\text{m/s}]
\]

with

\[
\rho_w = \text{density of water}, \quad V = \text{flow rate}, \quad d = \text{thickness},
\]

\[
A = \text{cross-section area}, \quad h_2 - h_1 = \text{pressure difference}
\]

4.2 Diffusion

In the stationary case, (constant concentration difference), Fick’s first law applies:
\[ \dot{m} = -D \frac{dc}{dx} \]

\( D \) is the diffusion coefficient, which is assumed constant over the regarded area. The concentration \( c \) (water vapour in air) is strongly influenced by temperature (see Table 3).

Table 3: *Saturation humidity content* \( c_s [\text{g/m}^3] \) and \( \delta_L [\text{kg/(mhPa)}] \).

<table>
<thead>
<tr>
<th>( T ) [°C]</th>
<th>( c_s [\text{g/m}^3] )</th>
<th>( \delta_L [\text{kg/(mhPa)}] )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>4.84</td>
<td>6.58</td>
</tr>
<tr>
<td>20</td>
<td>17.3</td>
<td>7.07</td>
</tr>
<tr>
<td>50</td>
<td>83.0</td>
<td>7.64</td>
</tr>
<tr>
<td>80</td>
<td>293.3</td>
<td>8.21</td>
</tr>
</tbody>
</table>

A further characteristic is the resistance coefficient of water vapour diffusion \( \mu \). It is dimensionless and indicates the ratio of the vapour diffusion of an air layer and a layer of the considered material, both with the thickness of 1m thus \( \mu = D_{\text{air}} / D_{\text{mat}} \). It is calculated as follows:

\[
\mu = \frac{1}{d} (\delta_L \cdot A \cdot \frac{p_1 - p_2}{I} - s_L)
\]

with:

\( p_1, p_2 \) water vapour partial pressures in [ Pa ]

\( I \) water vapour flow in [ kg/h ]

\( d \) medium thickness of the specimen in [ m ]

The diffusion transfer coefficient of water vapour of in air depends on both air pressure and temperature (see Table 2)
\[ \delta_L = \frac{0.083 \cdot p_0}{R_D \cdot T} \cdot \left( \frac{T}{p} \right)^{1.81} \]

with:

\[ R_D: \text{ gas constant of water vapour } 462 \text{ Nm/(kg K)} \]
\[ T: \text{ temperature in } [\text{ K} ] \]
\[ p: \text{ medium air pressure: } p_0: \text{ atmospheric pressure} \]

### 4.3 Capillary absorption

The data are presented in diagrams, in which the abscissa (time) is given in a square-root-of-time scale. The dependence between the surface-related water absorption \( W \) and the square root of time \( t \) is approximately linear, so that the water absorption coefficient results from the gradient of the straight line.

\[ A_i = \frac{\Delta W_i}{\Delta \sqrt{t}} \]

### 5. TESTING RESULTS

The following diagrams (fig. 2 and 3) describe the dependence of the permeability coefficient with on temperature. In fig. 4 and 5, the diffusion results are given. The capillary water absorption results are presented in fig. 6.
5.1 Permeability

Fig. 2: Permeability coefficient $k \ [m/s]$ versus temperature (after one hour)

Fig 3: Permeability coefficient $k \ [m/s]$ versus temperature (after 48 hours)
5.2 Diffusion

Fig. 4: Dependence of the resistance number $\mu$ on temperature (50 - 93 % R.H)

Fig. 5: Dependence of the diffusion coefficient $D$ on temperature (50 - 93 % R.H)
5.3 Capillary water absorption

![Graph showing water absorption coefficient vs temperature]

Fig. 6: Dependency of the water absorption coefficient $A$ from the temperature

6. DISCUSSION OF RESULTS

6.1 Permeability

Fig. 2 clearly shows the increase of permeability with temperature. The increase amounts approx. 10 % between 20 and 50 °C and approx. 8 % between 50 and 80 °C. The effect of the Mowilith content on permeability can be clearly recognised. A higher content of Mowilith is related with a reduction of the permeability coefficient. The difference between normal concrete and high-performance concrete becomes again very clear.

The permeability behaviour changes with time. In figure 3, the permeability coefficient after 48 h was plotted. The permeability decreased to approx. a tenth
of the values after one hour. They increase further approx. 10 to 20 % over the temperature range of 20 - 80 °C.

6.2 Diffusion

The most important concrete characteristic, apart from permeability, is diffusion. In fig. 4 the diffusion resistance numbers of the examined concretes are plotted together. Again the dependence of the transport processes on the temperature is shown. A reduction of the resistance number of approx. 10 - 20 % related to the temperature span of 20 - 80 °C can be read off clearly. For normal concrete, literature indicates values in the humid area procedure which are a third of those of the dry-cup procedures. According to DIN 4108, the values for the diffusion resistance number μ in the dry-cup-procedure are ranging between 70 - 150.

6.3 Capillary Absorption

Fig. 6 shows that the concrete specimen modified with 30 kg/m³ Mowilith has a very high absorption coefficient. The absorption coefficient is approx. 50 % higher than that of the normal concrete.

The values plotted, are the average values of three individual values. The high-performance concrete are situated with an A of 0,31 around approx. 11 % over the reference mixture.

Apart from the BPR, high-performance concretes are much less sensitive to temperature influences than e.g. the reference mixture or the 30 kg Mowilith mixture.

In contrast to the permeability or diffusion, the rise of the capillary absorption is almost linear with temperature.
7. CONCLUSION

The main conclusions are summarised as follows:

An increase of the transport processes with temperature increase is clearly shown. The different increase depends strongly on the kind of concrete.

- Concrete with Mowilith (30 kg/m³) is not suitable under the criterion of the increased strength and density with simultaneous price reduction. The physical properties differ only insignificantly from those of the reference mixture.

- However, concrete with Mowilith (45kg/m³) is suitable for the construction of heat storages under the criterion of increased strength and density at higher cost.

- All high-performance mixtures show very good results (density and strength). The concrete can be made without difficulties.

- The French concrete BPR represents the exception in this comparison. The density and the strength results are very good. However, the production requires much more know-how. With this concrete, the wall thickness of a storage tank could be substantially reduced.

8. REFERENCES


STUDY ABOUT THE CONTAMINATION OF PAH IN ROOMS WITH TAR PARQUETRY ADHESIVES

UNTERSUCHUNG VON PAK-BELASTUNGEN DURCH TEERHALTIGE PARKETTKLEBER IN INNENRÄUMEN

UNE ETUDE SUR LA CONTAMINATION EN HAP DUE AUX ADHESIFS POUR PARQUETS A BASE DE GOUDRON

Dagmar Hansen, Gerhard Volland

SUMMARY

In former times it was usual to use tar parquetry adhesives. Tar could be responsible for the contamination of the rooms with polycyclic aromatic hydrocarbons (PAH). In this study we investigated parquetry adhesives, dust, indoor air and outdoor air. The most important PAH compound is the benz(a)pyrene (BaP).

The indoor air was not significant contaminated with BaP. The values for the BaP in the parquetry adhesives were very high. But only in one case, it was possible to find a contamination of the dust. In this room the parquetry by itself was defect. In all the other rooms we could not find any contamination of the dust. The contamination of the rooms with BaP depends on conditions of the parquetry.

ZUSAMMENFASSUNG

Study about the contamination of PAH in rooms with tar parquetry adhesives


RESUME

Les adhésifs pour parquets à base de goudron ont été utilisés jusqu'à la fin des années 70. L'influence de ces adhésifs sur la contamination de l'air d'intérieur en hydrocarbures aromatiques polycycliques (HAP) est analysé dans cette étude. Nous avons pour cela mesuré la contenance en benz(a)pyèrne (BaP), le principal HAP, de l'adhésif, de la poussière, ainsi que de l'air d'intérieur et d'extérieur. Une contamination significative en BaP de l'air d'intérieur n'a pu être constatée dans aucun des bâtiments considérés.

La concentration en BaP dans les adhésifs variait entre 20 et 19000 mg/kg. Malgré ces hautes valeurs, une contamination de la poussière n'a pu être décelée que dans un seul cas. Dans cette pièce, le parquet était endommagé, en particulier le cachetage était défectueux. Une contamination n'a pu être décelée dans aucune pièce dont le parquet était intact, même si l'adhésif utilisé avait une haute concentration en BaP. L'état du parquet constitue une condition décisive pour une contamination en BaP.

KEYWORDS: tar parquetry adhesives, dust, indoor air, benz(a)pyrene

1. INTRODUCTION

A lot of buildings in Germany belonged to the American army. After the Americans left Germany these buildings were sold. At the beginning of this year it was suspected, that these buildings were contaminated with polycyclic aromatic hydrocarbons (PAH). This was suspected by measurements of the “Frankfurter Housing Area”. These measurements showed contamination’s of dust and indoor air with PAH’s. The use of tar parquetry adhesives in these
buildings could be responsible for this contamination. The PAH from the tar parquetry adhesives could be transported to the floor dust. Parents were fret about the contamination of the floor dust, because their infants often play at the floor. A study about the oral taking up of dust [AGLMB, 1995] shows, that infants (between 0 and 6 years) are taking up average 20 mg (95. percentile 100 mg) dust per day.

In April 1998 the Umweltbundesamt Germany gave an information about the investigation of rooms with tar parquetry adhesives [UMWELTBUNDESAMT, 28.04.1998]. Regarding to the toxicity of PAH’s only benz(a)pyrene (BaP) should be investigated in dark parquetry adhesives. For further information please have a look at appendix 1.

Our study was carried out to check these points. In this summer we started the examination of 10 US-flats, 10 new-building flats and 10 public buildings (like schools and kindergartens) in co-operation with the Sozialministerium Baden-Württemberg and the Landesgesundheitsamt Baden-Württemberg. We investigated parquetry adhesives, floor dust, old dust, indoor air and outdoor air. 15 EPA – PAH (except naphthaline) were determined.

2. SAMPLE COLLECTION

Some special points, like age and condition of parquetry, heating and ventilation of room, smoking in room, were noted in a catalogue, which has been developed by the Landesgesundheitsamt Baden-Württemberg [LANDESGESUNDHEITSAMT, 1998]. All samples were collected in the nursery. In order to compare the results and to exclude contamination’s of the samples, the samples were collected by the following general conditions:

- collection of indoor and outdoor air over 7 days
- room cleaned up four days before dust collection
- order of succession: first dust collection, second collection of parquetry
2.1 Parquetry adhesives

An expert opened the parquetry and collected the tar parquetry adhesives. It was difficult to get a representative sample. The parquetry should not be destroyed and therefore it was opened at the periphery. The quantity of used parquetry adhesives is very different (from punctiform to wing loading). Impurities (like wood, cardboard, mineral compounds) could be attached at the parquetry adhesives and, at least, in one single room the use of different parquetry adhesives is possible.

2.2 Floor dust

For the collection of the dust it is not possible to use a normal vacuum cleaner, because the vacuum cleaner absorbers dust and parquetry adhesives out of the joints. Further it is also not possible to get the fine dust.

We used an absorber with a special jet (diameter: 0.5 cm), a glass-fiber filter and a flow rate of 15 l/min. It is very important, to keep a distance between jet and joint. It must be about 1 cm. Only in this case no dust and no parquetry adhesives out of the joint will be collected. The examined surface measured 2 m². The collected dust was weighted several times under definite moisture conditions. This sample is called “absorber dust”.

We also collected dust by wiping the surface (1 m²) with a dry polyurethane-foam (diameter: 2 cm). This dust sample is called “wipe dust”.

2.3 Old dust

Old dust is defined as the normal old dust on wardrobes, bookcases, cupboards, door-cases etc. The old dust was collected by the “absorber dust” method, see above, please.
2.4 Indoor air

The indoor air was collected with a polyurethane foam. The collection was realised over 7 days under normal conditions [VDI 4300 Bl. 2] with a rate of 0.2 – 1 l/min.

2.5 Outdoor air

Corresponding to VDI 4300 Bl. 2 the outdoor air was collected at the same time like the indoor air.

3. ANALYTICAL METHOD

The samples were spiked with a quantitation standard, before collection (air sample) or before extraction (dust sample) respectively after extraction (parquetry adhesives). The samples were extracted with cyclohexane, the extract was cleaned by solid phase extraction with silicagele. Before measurement the extract was spiked with a recovering standard (except: parquetry adhesives).

We analysed the PAH with the gas chromatography coupled with the high resolution mass spectrometer (GC-HRMS). This is a very selective and sensitive process. It is often use to identify complex mixtures with low detection limits.

Another advantage of the mass spectrometer is the possibility of using isotope standards. The quantitation standard was a $^{13}\text{C}$ – isotope standard of all 15 PAH. The recovering standard included deuterate PAH – isotope. The comparision between the quantitation standard and the recovering standard gave the recovering rate.

The parquetry adhesives is very heterogeneous. Impurities (like wood, cardboard, mineral compounds) could be attached at the parquetry adhesives and falsified the weight. Therefore the evaporation residue of the cyclohexane
extration by 40°C over 24 hours was determined. This evaporation residue reproduced the soluble part of the parquetry adhesives. For the quantification this soluble part was used.

The preliminary examination of the parquetry adhesives was carried out by the nuclear resonance spectrometry (NMR). With this method we got first information’s about the compounds.

4. RESULTS AND DISCUSSION

4.1 Air

The readily volatile PAH as fluorene, acenaphthene and acenaphthaline were discriminated by this collection method. This is shown by the recovering rates. The less volatile compound phenanthrene has a recovering rate above 50% and the recovering rate of BaP is higher than 70%. So BaP could be determinate with this method.

We measured BaP concentrations in the indoor air between 0.1 ng/m³ and 1.2 ng/m³ and in the outdoor air between 0.1 ng/m³ and 0.3 ng/m³. In summer times this BaP concentrations are normal. A study [UMEG, 1996] showed, that the concentration of BaP in outdoor air in winter times is significantly higher than in summer times.

4.2 Dust

The recovering rates for the 15 PAH were between 80% and 120%. Only in one room we found BaP concentrations above 1 mg/kg in the old dust and in the floor dust. In all the others the concentration of BaP in the old dust and in the floor dust were lower than 1 mg/kg.

The exception with a high level for BaP was a public building. The old dust
had a concentration of 32 mg/kg and the floor dust about 45 mg/kg. According to the Umweltbundesamt Germany [UMWELTBUNDESAMT, 28.04.1998] (see also appendix 1) the limit for BaP was 10 mg/kg. Therefore it was necessary to repeat the measurement of the floor dust. The room was cleaned a short time before the repeat measurement. (Contrast to the general conditions for sample collection see point 2) The repeat test showed BaP values of 15 mg/kg and 10 mg/kg. Therefore, the first measurement was verified. Table 1 shows the comparison of the old dust and the floor dust for all 15 PAH.

Table 1: Comparison old dust and floor dust

<table>
<thead>
<tr>
<th>PAH</th>
<th>old dust 1 mg/kg</th>
<th>floor dust 1 mg/kg</th>
<th>floor dust 2a mg/kg</th>
<th>floor dust 2b mg/kg</th>
</tr>
</thead>
<tbody>
<tr>
<td>acenaphtheline</td>
<td>0.1</td>
<td>0.1</td>
<td>0.2</td>
<td>0.1</td>
</tr>
<tr>
<td>acenaphthene</td>
<td>2.0</td>
<td>8.2</td>
<td>2.4</td>
<td>1.6</td>
</tr>
<tr>
<td>fluorene</td>
<td>2.3</td>
<td>14</td>
<td>3.8</td>
<td>2.2</td>
</tr>
<tr>
<td>phenanthrene</td>
<td>76</td>
<td>180</td>
<td>49</td>
<td>31</td>
</tr>
<tr>
<td>anthracene</td>
<td>4.1</td>
<td>33</td>
<td>8.3</td>
<td>4.2</td>
</tr>
<tr>
<td>fluoranthene</td>
<td>86</td>
<td>147</td>
<td>35</td>
<td>23</td>
</tr>
<tr>
<td>pyrene</td>
<td>41</td>
<td>72</td>
<td>25</td>
<td>17</td>
</tr>
<tr>
<td>benzo(a)anthracene</td>
<td>26</td>
<td>64</td>
<td>18</td>
<td>12</td>
</tr>
<tr>
<td>chrysene</td>
<td>42</td>
<td>61</td>
<td>22</td>
<td>17</td>
</tr>
<tr>
<td>benzo(b)fluoranthene</td>
<td>36</td>
<td>43</td>
<td>12</td>
<td>9.8</td>
</tr>
<tr>
<td>benzo(k)fluoranthene</td>
<td>39</td>
<td>51</td>
<td>15</td>
<td>11</td>
</tr>
<tr>
<td><strong>benzo(a)pyrene</strong></td>
<td><strong>32</strong></td>
<td><strong>45</strong></td>
<td><strong>15</strong></td>
<td><strong>10</strong></td>
</tr>
<tr>
<td>indeno(1,2,3-c,d)pyrene</td>
<td>16</td>
<td>18</td>
<td>7.5</td>
<td>5.2</td>
</tr>
<tr>
<td>dibenz(a,h)anthracene</td>
<td>5.3</td>
<td>5.5</td>
<td>1.1</td>
<td>0.8</td>
</tr>
<tr>
<td>benzo(g,h,i)perylene</td>
<td>16</td>
<td>18</td>
<td>7.3</td>
<td>5.1</td>
</tr>
<tr>
<td><strong>sum 15 PAH</strong></td>
<td><strong>425</strong></td>
<td><strong>762</strong></td>
<td><strong>220</strong></td>
<td><strong>149</strong></td>
</tr>
</tbody>
</table>

old dust 1, floor dust 1: first measurement  
floor dust 2a, floor dust 2b: repeat measurement

The PAH concentration in the floor dust is slightly higher than in the old dust. The values for the repeat measurement are comparable.

Table 2 shows the comparison of the technique for the collection of the floor dust, once the absorber dust and once the wipe dust.
Table 2: Comparision absorber dust and wipe dust

<table>
<thead>
<tr>
<th>PAH</th>
<th>room A</th>
<th></th>
<th>room B</th>
<th></th>
<th>room C</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>absorber dust</td>
<td>wipe dust</td>
<td>absorber dust</td>
<td>wipe dust</td>
<td>absorber dust</td>
<td>wipe dust</td>
</tr>
<tr>
<td>acenaphthaline</td>
<td>0.1</td>
<td>21</td>
<td>0.84</td>
<td>26</td>
<td>1.5</td>
<td>24</td>
</tr>
<tr>
<td>acenaphthene</td>
<td>6.8</td>
<td>91</td>
<td>5.0</td>
<td>135</td>
<td>18</td>
<td>117</td>
</tr>
<tr>
<td>fluorene</td>
<td>13</td>
<td>231</td>
<td>9.2</td>
<td>258</td>
<td>21</td>
<td>197</td>
</tr>
<tr>
<td>phenanthrene</td>
<td>84</td>
<td>409</td>
<td>119</td>
<td>312</td>
<td>155</td>
<td>399</td>
</tr>
<tr>
<td>anthracene</td>
<td>8.0</td>
<td>35</td>
<td>5.5</td>
<td>21</td>
<td>14</td>
<td>34</td>
</tr>
<tr>
<td>fluorantheine</td>
<td>41</td>
<td>177</td>
<td>43</td>
<td>152</td>
<td>38</td>
<td>103</td>
</tr>
<tr>
<td>pyrene</td>
<td>29</td>
<td>137</td>
<td>25</td>
<td>107</td>
<td>26</td>
<td>95</td>
</tr>
<tr>
<td>benzo(a)anthracene</td>
<td>12</td>
<td>16</td>
<td>4.6</td>
<td>6.8</td>
<td>9.4</td>
<td>8.7</td>
</tr>
<tr>
<td>chrysene</td>
<td>13</td>
<td>33</td>
<td>14</td>
<td>40</td>
<td>12</td>
<td>29</td>
</tr>
<tr>
<td>benzo(b)fluorantheine</td>
<td>7.8</td>
<td>11</td>
<td>5.2</td>
<td>13</td>
<td>6.4</td>
<td>7.5</td>
</tr>
<tr>
<td>benzo(k)fluorantheine</td>
<td>13</td>
<td>11</td>
<td>4.7</td>
<td>11</td>
<td>6.9</td>
<td>6.8</td>
</tr>
<tr>
<td><strong>benzo(a)pyrene</strong></td>
<td><strong>8.5</strong></td>
<td><strong>8.3</strong></td>
<td><strong>1.7</strong></td>
<td><strong>2.2</strong></td>
<td><strong>5.6</strong></td>
<td><strong>3.1</strong></td>
</tr>
<tr>
<td>indeno(1,2,3-c,d)pyrene</td>
<td>1.9</td>
<td>3.8</td>
<td>1.4</td>
<td>1.1</td>
<td>3.3</td>
<td>1.5</td>
</tr>
<tr>
<td>dibenz(a,h)anthracene</td>
<td>&lt;0.1</td>
<td>0.3</td>
<td>0.1</td>
<td>&lt;0.1</td>
<td>0.8</td>
<td>&lt;0.1</td>
</tr>
<tr>
<td>benzo(g,h,i)perylene</td>
<td>3.0</td>
<td>5.1</td>
<td>2.2</td>
<td>1.2</td>
<td>3.6</td>
<td>2.8</td>
</tr>
<tr>
<td><strong>sum 15 PAH</strong></td>
<td><strong>240</strong></td>
<td><strong>1191</strong></td>
<td><strong>242</strong></td>
<td><strong>1086</strong></td>
<td><strong>321</strong></td>
<td><strong>1028</strong></td>
</tr>
</tbody>
</table>

For both methods, the BaP concentrations are comparable. But there is a great difference for the sum of PAH. The wipe dust showed substantially higher values for the sum of PAH than the absorber dust. The readily volatile PAH were discriminated by the collection with a absorber. This effect can be shown until benzo (a) anthracene.

The dust collection in private household was complicated, because the amount of the dust were often very low.

In public buildings the dust included gross impurities. In this cases the dust was fractionated. By the sample collection a sieve (mash width: 500 µm) were used. There was no relevant difference in the PAH concentrations between the
non fractionated and the fractionated dust.

4.3 Parquetry adhesives

The preliminary examination with NMR showed the different compounds of the parquetry adhesives like bitumen, tar and combinations of bitumen and tar. Table 3 presents the concentration of BaP [mg/kg] and sum 15 PAH [g/kg] once for the original sample and once for the soluble part of the sample. The soluble part of the parquetry adhesives were determined under defined conditions (see point 3).

Table 3: BaP and sum 15 PAH for original and soluble part of adhesives

<table>
<thead>
<tr>
<th>adhesives sample</th>
<th>BaP mg/kg original</th>
<th>BaP mg/kg soluble part</th>
<th>Σ15 PAH g/kg original</th>
<th>Σ15 PAH g/kg soluble part</th>
<th>NMR</th>
<th>soluble part</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>2</td>
<td>20</td>
<td>0.05</td>
<td>0.56</td>
<td>bitumene</td>
<td>8 %</td>
</tr>
<tr>
<td>B</td>
<td>15</td>
<td>19</td>
<td>0.21</td>
<td>0.26</td>
<td>bitumene</td>
<td>80 %</td>
</tr>
<tr>
<td>C</td>
<td>730</td>
<td>7430</td>
<td>22</td>
<td>224</td>
<td>tar</td>
<td>10 %</td>
</tr>
<tr>
<td>D</td>
<td>1990</td>
<td>15000</td>
<td>39</td>
<td>293</td>
<td>tar</td>
<td>13 %</td>
</tr>
<tr>
<td>E</td>
<td>1060</td>
<td>3380</td>
<td>18</td>
<td>57</td>
<td>tar, bitumene</td>
<td>31 %</td>
</tr>
<tr>
<td>F</td>
<td>280</td>
<td>430</td>
<td>14</td>
<td>22</td>
<td>tar, bitumene</td>
<td>64 %</td>
</tr>
</tbody>
</table>

Depending on the impurities of adhesives the soluble part was very different. The limit value of 10 mg/kg for the parquetry adhesives [UMWELTBUNDESAMT, 28.04.1998] (see also appendix 1) was exceeded even by pure bitumen parquetry adhesives.

Table 4 shows some BaP concentrations of the outdoor air, the indoor air, the old dust, the floor dust and the parquetry adhesives (refer to soluble part).

Table 4: Summary of the BaP concentrations

<table>
<thead>
<tr>
<th>outdoor air ng/m³</th>
<th>indoor air ng/m³</th>
<th>old dust mg/kg</th>
<th>floor dust mg/kg</th>
<th>adhesives mg/kg</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 0.1</td>
<td>&lt; 0.1</td>
<td>&lt; 1</td>
<td>&lt; 1</td>
<td>20</td>
</tr>
</tbody>
</table>
In no event we found a contamination of BaP in the indoor air (compare to [UMWELTBUNDESAMT, 28.04.1998]).

The BaP values for the parquetry adhesives are lying all over 10 mg/kg, by half of the samples they are lying above 3000 mg/kg. But only in one case we found a contamination of the dust. In this room the parquetry by itself was defect, especially the parquetry sealing was destroyed, joint width was large and the parquetry slats were disconnected.

### 5. CONCLUSION

According to the information of the Umweltbundesamt Germany [UMWELTBUNDESAMT, 28.04.1998] we could not find any contamination of the indoor air with BaP.

The BaP limit for the parquetry adhesives (by the Umweltbundesamt Germany) at about 10 mg/kg is not convenient, because pure bitumen parquetry adhesives already shows a higher concentration.

Responsible for BaP contamination of rooms with tar parquetry adhesives is the condition of the parquetry. Investigations in our institute show, that also the condition of the floor basis can cause a BaP contamination. By a damaged state of the floor basis, the parquetry can move and at stressing parts it can smash up. By this alternating effects, the tar parquetry adhesives can be pulverize and by a pump effect the tar can come to the surface of the parquetry.

By an uninjured parquetry we can not find any BaP contamination’s of the
room, even if the tar parquetry adhesives shows very high values (e.g. 19 g/kg).

The description of the condition of the parquetry floor is very important for a assess of a BaP contamination.

6. ACKNOWLEDGEMENT

This study was supported by the Sozialministerium Baden-Württemberg and the Landesgesundheitsamt Baden-Württemberg. We also want to thank the Gesundheitsamt Heilbronn and the District Heilbronn for the very good co-operation. Special appreciation must be expressed to Mr. Dr. Wuthe and Mr. Dr. Link from the Sozialministerium Baden-Württemberg; Mrs. Dr. Jovanovic and Mr. Dr. Gabrio from the Landesgesundheitsamt Baden-Württemberg; the inhabitant of the investigated rooms; and Mrs. Sanderbrand, Mrs. Pinkas and Mrs. Klein from the FMPA.
7. APPENDIX 1

Information from the Umweltbundesamt Germany 28.04.1998

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**Flowchart Description**

- **Adhesives:** 
  - **BaP > 10 mg/kg** → no action
  - **BaP > 3000 mg/kg**
    - yes → determination of BaP in dust
      - yes → Dust: BaP > 10 mg/kg
          - yes → short-term under mine action
          - no → determination of BaP in outdoor and indoor air
            - yes → I > 2A
              - yes → individual case: under mine action
              - no → I > A + 3
                - yes → no action
                - no → I.A: indoor-, outdoor air BaP [ng/m³]
      - no → determination of BaP in dust
        - no → no action
REFERENCES


LÄNDERGESUNDHEITSAMT: Check – Liste Begehung – PAK; Mai 1998


UMWELT BUNDESAMT, PRESSE-INFORMATION: Empfehlung zu polyzyklischen aromatischen Kohlenwasserstoffen (PAK) in Wohnungen mit Parkettböden; Ergebnisse des zweiten Expertengespräches am 28.04.1998 im Umweltbundesamt

VDI-RICHTLINIE VDI 4300 BLATT 2: Messen von Innenraumluftverunreinigungen
PROGNOSIS OF UNI-AXIAL COMPRESSIVE STRENGTH AND STIFFNESS OF ROCKS BASED ON POINT LOAD AND ULTRASONIC TESTS

PROGNOSE DER EINAXIALEN DRUCKFESTIGKEIT UND STEIFIGKEIT VON FESTGESTEINEN AUF DER BASIS VON PUNKTLASTVERSUCHEN UND DURCHSCHALLUNG

PROGNOSTIC DE LA RESISTANCE A LA COMPRESSION UNI-AXIALE ET LA RIGIDITE DE ROCHE A LA BASE D’ESSAIS A CHARGE PONCTUELLE ET A ULTRA-SON

Tobias Bräutigam, Arno Knöchel, Markus Lehne

SUMMARY

The strength- and deformation properties of in total 10 different rock types of South West Germany were investigated by uni-axial compressive tests according to E1-AK 19 (3) and modified point load tests with deformation measurement. Additionally the dynamic characteristics of rock were determind by ultra-sonic measurements.

Within the framework of test evaluations the measuring values of static and dynamic modulus of elasticity, of compressive strength and dynamic and static modulus of elasticity and of stiffness in the point load test for the different rock types were correlated.

ZUSAMMENFASSUNG

Im Rahmen der Versuchsauswertungen wurden die Meßwerte von statischem und dynamischem E-Modul, von Zylinderdruckfestigkeit und dynamischem E-Modul und von statischem E-Modul und der Steifigkeit im Punktlastversuch für die verschiedenen Gesteinsarten korreliert.

RESUME

Les propriétés de résistance et déformation de, au total, 10 types différents de roche d’Allemagne de Sud-Ouest ont été investigées par essais de compression uni-axiaux selon E1-AK 19 (3) et par essais à charge ponctuelle modifiés avec mesure de déformation. En outre les caractéristiques dynamiques de roche ont été déterminées par mesures à ultra-son.

Dans le cadre des évaluations d’essai les valeurs mesurées du module d’élasticité statique et dynamique, de résistance à la compression et module d’élasticité dynamique et de module d’élasticité statique et de la rigidité dans l’essai à charge ponctuelle de différents types de roche ont été correlées.

KEYWORDS: Strength of rock types of South West Germany, dynamic characteristics of rock types of South West Germany, point load test with deformation measurement for rock specimen, stiffness of rock in point load test, correlation of rock characteristics.

1. INTRODUCTION

For construction works in rock formations the uni-axial compressive strength is often used as one of the criteria for reimbursement. As a basis, the classification schemes in DIN 18301, Bohrarbeiten, (5) and DIN 18319, Rohrvortriebsarbeiten, (6) should be mentioned. In these, rocks are classified according to uni-axial compressive strength and fissuring.
Because of the low costs and speed at which the results are available, index tests for indirectly determining the uni-axial compressive strength are important. For these reasons tests such as the point load test and sometimes ultrasonic testing are quite often used by contractors and consulting engineers.

On large projects such as traffic tunnels and pipe jacking work rock samples are taken on a routine basis from different sections for strength testing. Thus it is possible to check the prognosticated strength of the rock and determine any deviations that could lead to change in the remuneration for the work carried out. Often the first checks are carried out using index tests.

The correlation factor with which the uni-axial compressive strength is calculated from the strength index is dependent on the rock type (13), (14) and should in order to obtain absolutely correct results be determined by reference tests in advance. In practice however an average correlation factor of $\alpha = 24$ which is recommended in E5-AK 19, (4) is often used with no differentiation of rock type. Recent tests (1), (11), (12) however show that the deviation of the correlation factors for different rock types from the combined average value can be so large that an exact determination for the purpose of fixing the remuneration of the construction work can be important. This is especially important when differences of opinion occur in border line cases.

Obtaining and preparation of rock cores for uni-axial compression tests is however often very expensive. In order to obtain more precise values for the correlation factors for the point load test two theses (11), (12) were carried out at the FMPA. In these theses strength tests on ten different rock types in South West Germany were carried out. The rock types include both sedimentary as well as igneous rocks. Similar tests with partly different aims have been carried out at Graz university in Austria (1) on alpine rocks.
The tests carried out in (11) and (12) were also used to correlate between:

- static modulus of elasticity and dynamic modulus of elasticity
- uni-axial compressive strength and dynamic modulus of elasticity
- static modulus of elasticity and stiffness in point load test.

2. SELECTION OF ROCK TYPES AND BLOCKS FOR THE INVESTIGATION

The sedimentary and igneous rocks investigated represent a group which is also important outside the borders of Baden-Württemberg as a raw material and thus important in construction works. The size of the blocks of rock taken from the various locations was so selected that the cores with a diameter \(d\) of 74 mm and a length \(l\) at least equal to the diameter \(l/d > 1\), ideally \(l/d \geq 2\) could be taken.

- **sedimentary rock**
  1. Katzenkopf Stubensandstein
  2. Ostfildern Angulatensandstein
  3. Dietersweiler Plattensandstein
  4. Dotternhausen Oxfordkalk
  5. Enzberg Nodosuskalk

- **igneous rock**
  6. Detzeln Paragneisanatexit
  7. Bötzingen Phonolith
  8. Tegernau Granit
  9. Ottenhöfen Quarzporphyr
  10. Hohenstoffeln Olivin-Nephelinit
fig. 1: Locations of the rock types investigated
3. DESCRIPTION OF TEST PROCEDURES

3.1 Ultrasonic measurements

The dynamic rock properties are determined by sending sound waves through the sample. The speed of ultrasonic longitudinal waves as well as the density of the sample are used to determine the dynamic modulus of elasticity.

\[ E_{dyn} = \rho \cdot v_{ul}^2 \text{ (MPa)} \]

The ultrasonic measurements in the test were carried out on rock cores.

3.2 Uni-axial compression test

The uni-axial compression test (3) is used to determine the compressive strength as well as the deformation characteristics of a rock sample under a one dimensional stress state in the laboratory. The compressive stress is the quotient of the uni-axial test load \( F \) and the area of the sample \( A \)

\[ \sigma = \frac{F}{A} \text{ (MPa)} \]

The strain is the quotient of the sample deformation \( \Delta l \) and the original length of the sample \( l \).

\[ \varepsilon = \frac{\Delta l}{l} \]

The static modulus of elasticity is determined in the linear part of the load deformation curve.
3.3 Modified point load test

In the point load test (4) a strength index $I_s$ is determined which allows the uni-axial compressive strength to be estimated. The rock sample is placed between two cone shaped loading points and the load is increased until failure occurs.

The strength index $I_s$ is the quotient of the failure load $F_u$ and the square of the distance between the two loading points at the start of the test.

$$I_s = \frac{F_u}{a^2} \text{ (MPa)}$$

In order to compare results (4) recommends a standard distance between the loading points of $a = 50$ mm. The standardisation can either be achieved using a nomogramme (4) or with the following formula (11), (12):

$$I_{s(50)} = \left(\frac{14 + 0.175a}{22.75}\right) \cdot I_s \text{ (MPa)}$$

The uni-axial compressive strength $\sigma_u$ is estimated using a correlation factor $\alpha$ which is typical\(^1\) for the rock type:

$$\sigma_u = \alpha \cdot I_{s(50)} \text{ (MPa)}$$

In the standard point load test the deformation of the sample during the test is not measured. In the modified point load test (11) and (12) the deformation during the loading is recorded (see fig.2).

The slope of the deformation curve of a rock from a point load test can be looked upon as the stiffness of the rock. This stiffness is called (11) and (12)

---

\(^1\) The correlation factor $\alpha$ can be significantly different for a particular rock type when the samples have different origins (7), (8), (9).
Iₜ-modulus. This Iₜ-modulus is calculated from the linear part of the load deformation curve.

fig. 2: Modified point load test with measurement of deformation
The slope of the deformation curve of a rock from a point load test can be looked upon as the stiffness of the rock. This stiffness is called \((11)\) and \((12)\) \(I_s\)-modulus. This \(I_s\)-modulus is calculated from the linear part of the load deformation curve.

4. TEST RESULTS AND CORRELATIONS

The average values for the density, dynamic elasticity modulus, static elasticity modulus and compressive strength (tables 1 and 2) were calculated from 6 individual values for each rock type. The average of the standardised strength index \(I_s(50)\) was calculated from 25 - 30 individual values for each rock type.

4.1 Correlation between point load strength and uni-axial compressive strength

The possible error in the calculation of the uni-axial compressive strength from the point load strength using a correlation factor \(\alpha = 24\) is given by BIENIAWSKI (2) as 20%. In table 1 the results of the test on sedimentary rocks carried out by LEHNE (12) are listed. The average values of \(a\) vary from \(a = 17.31\) for Angulatensandstein to \(a = 27.09\) for Oxfordkalk. The values for igneous rocks tested by KNÖCHEL (11) (see table 2) lie between \(\alpha = 9.64\) for Paragneisanatexit and \(\alpha = 15.80\) for Olivin-Nephelinit. The quotient of standard deviation to the average values gives some idea of the statistical distribution curve.
tab.1:  *Test results (Average) for sedimentary rocks*

<table>
<thead>
<tr>
<th></th>
<th>Platten- sandstein</th>
<th>Nodosus- Kalk</th>
<th>Stuben- sandstein</th>
<th>Angulaten sandstein</th>
<th>Oxford- Kalk</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>density ρ</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>2.35</td>
<td>2.64</td>
<td>2.16</td>
<td>2.38</td>
<td>2.56</td>
</tr>
<tr>
<td>Variation</td>
<td>0.004</td>
<td>0.005</td>
<td>0.022</td>
<td>0.010</td>
<td>0.007</td>
</tr>
<tr>
<td><strong>Dyn. E-modulus</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>17228</td>
<td>88146</td>
<td>5316</td>
<td>43163</td>
<td>59441</td>
</tr>
<tr>
<td>Variation</td>
<td>0.049</td>
<td>0.068</td>
<td>0.314</td>
<td>0.046</td>
<td>0.100</td>
</tr>
<tr>
<td><strong>Stat. E-modulus</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>9195</td>
<td>19773</td>
<td>2592</td>
<td>15169</td>
<td>16058</td>
</tr>
<tr>
<td>Variation</td>
<td>0.100</td>
<td>0.150</td>
<td>0.214</td>
<td>0.003</td>
<td>0.186</td>
</tr>
<tr>
<td><strong>Compressive strength σ_u</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>60.48</td>
<td>71.32</td>
<td>11.31</td>
<td>92.24</td>
<td>111.60</td>
</tr>
<tr>
<td>Variation</td>
<td>0.046</td>
<td>0.087</td>
<td>0.288</td>
<td>0.099</td>
<td>0.083</td>
</tr>
<tr>
<td><strong>I_s(50)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>2.67</td>
<td>4.02</td>
<td>0.42</td>
<td>5.33</td>
<td>4.12</td>
</tr>
<tr>
<td>Variation</td>
<td>0.128</td>
<td>0.303</td>
<td>0.369</td>
<td>0.399</td>
<td>0.144</td>
</tr>
<tr>
<td><strong>I_s-modulus</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>80.04</td>
<td>154.29</td>
<td>18.48</td>
<td>126.42</td>
<td>161.02</td>
</tr>
<tr>
<td>Variation</td>
<td>0.184</td>
<td>0.202</td>
<td>0.204</td>
<td>0.180</td>
<td>0.186</td>
</tr>
<tr>
<td><strong>Factor α</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>22.66</td>
<td>17.74</td>
<td>27.05</td>
<td>17.31</td>
<td>27.09</td>
</tr>
</tbody>
</table>
Prognosis of uni-axial compressive strength and stiffness of rocks

tab.2: Test results (Average) for igneous rocks

<table>
<thead>
<tr>
<th></th>
<th>Granit</th>
<th>Paragneis-anatexit</th>
<th>Quarz-porphyr</th>
<th>Phonolith</th>
<th>Olivin-Nephelinit</th>
</tr>
</thead>
<tbody>
<tr>
<td>density $\rho$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average [g/cm³]</td>
<td>2.61</td>
<td>2.76</td>
<td>2.52</td>
<td>2.40</td>
<td>3.12</td>
</tr>
<tr>
<td>Variation [ ]</td>
<td>0.010</td>
<td>0.014</td>
<td>0.004</td>
<td>0.021</td>
<td>0.004</td>
</tr>
<tr>
<td>Dyn. E-Modul</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average [MPa]</td>
<td>64631</td>
<td>70227</td>
<td>57430</td>
<td>62242</td>
<td>137918</td>
</tr>
<tr>
<td>Variation [ ]</td>
<td>0.057</td>
<td>0.080</td>
<td>0.059</td>
<td>0.111</td>
<td>0.040</td>
</tr>
<tr>
<td>Stat. E-modulus</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average [MPa]</td>
<td>28481</td>
<td>13454</td>
<td>22156</td>
<td>25045</td>
<td>40000</td>
</tr>
<tr>
<td>Variation [ ]</td>
<td>0.041</td>
<td>0.173</td>
<td>0.076</td>
<td>0.090</td>
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fig. 3: Correlation factors $\alpha$ for sedimentary rocks

fig. 4: Correlation factors $\alpha$ for igneous rocks
4.2 Correlation between dynamic elasticity modulus and static elasticity modulus

The dynamic elasticity modulus is generally higher than the static elasticity modulus. HENKE, GAY, KAISER (1974), (10) showed that for rock types with $E_{\text{dyn}} < 50000$ MPa there is a linear correlation with $E_{\text{stat}} = 0.7388 \times E_{\text{dyn}}$. The correlation was for both sedimentary and igneous rocks. The number of samples was however small.

The tests by LEHNE (1998), (12) on sedimentary rocks and by KNÖCHEL (1998), (11) on igneous rocks also show a large number of samples a linear correlation but the correlation factors are much smaller.

- Sedimentary rocks: $0.224 < E_{\text{stat}}/E_{\text{dyn}} < 0.534$, Average = 0.373
- Igneous rocks: $0.290 < E_{\text{stat}}/E_{\text{dyn}} < 0.441$, Average = 0.341

fig. 5: Correlation between dynamic modulus of elasticity and static modulus of elasticity for sedimentary rocks
Correlation between dynamic modulus of elasticity and static modulus of elasticity for igneous rocks

4.3 Correlation between dynamic elasticity modulus and uni-axial compressive strength

The ultrasonic waves run predominantly through the dense rock matrix. Any cavities are circumvented using rock bridges. Dense rock matrices have high ultrasonic wave speeds. The dynamic E-modulus depends on the square of the ultrasonic wave speed and so dense rock matrices have high dynamic elasticity moduli. With regard to the compressive strength, any cavities have a negative influence since the area of the cavities have to be subtracted from the total area of the test sample.
Prognosis of uni-axial compressive strength and stiffness of rocks

Fig. 7: Correlation between dynamic modulus of elasticity and uni-axial compressive strength of sedimentary rocks

Fig. 8: Correlation between dynamic modulus of elasticity and uni-axial compressive strength of igneous rocks
4.4 Correlation between $I_s$-modulus and static elasticity modulus

A comparison of the load deformation properties of rock samples in the uni-axial compression test and the point load test shows that for sedimentary rocks a good linear correlation exists (12) p. 50. On the other hand the results of the tests on igneous rocks did not show a very good correlation (11) p. 63.

![Correlation between $I_s$-modulus and static modulus of elasticity for sedimentary rocks](image.png)

fig. 9: Correlation between $I_s$-modulus and static modulus of elasticity for sedimentary rocks
5. ASSESSMENT OF TEST RESULTS AND NOTES FOR THE PRACTICAL USE

It is usual in the geotechnique to use the results of more economic index tests to serve as a correlation basis for rock parameters whose direct determination are both time consuming and expensive. In proven cases it is also a suitable method of determining the magnitude of individual rock parameters.

The work of KNÖCHEL (11) and LEHNE (12) show that general correlations exist between uni-axial compressive strength and point load strength, between static modulus of elasticity and dynamic modulus of elasticity as well as between uni-axial compressive strength and dynamic modulus of elasticity. For example (11) and (12) showed that a schematic use of a correlation factor $\alpha = 24$ for calculation the uni-axial compressive strength from the point load strength leads to on average to uni-axial compressive strength which is 13% too low for Oxford-Kalk and 250% too high for Paragneisanatexit.

Fortunately it seems that in practical cases – especially in questions of classification – the realisation that a testing of the validity and applicability of a
correlation for a particular rock can be worthwhile. For construction works with
large excavation masses in linear structures such as tunnels etc. it is worthwhile
thinking over whether a correlation for a particular job should be set up.

Further systematic testing in addition to that in (11) and (12) is required in
order to assess whether a general correlation exists between static E-modulus and
Iₜ-modulus.

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REAR ATTACHMENT OF PANELS FOR VENTILATED CURTAIN WALLS

RÜCKSEITIGE BEFESTIGUNG VON BEKLEIDUNGSPLATTEN FÜR VORGEHÄNGTE HINTERLÜFTETE AUßENWANDBEKLEIDUNGEN

FIXATION ARRIERE DES PANNEAUX POUR DES FACADES RIDEAUX

Rolf Lehmann

SUMMARY

The structure and function of attachment devices at the back of panels are described.

Possible interaction of panels and substructure and the necessity of symmetrical loading are indicated.

The method of testing the load bearing capacity of the attachment elements is explained and results of tests performed are commented on.

ZUSAMMENFASSUNG

Die Ausbildung und Wirkungsweise der rückseitigen Befestigungs- elemente für Bekleidungsplatten von vorgehängten Außenwandbekleidungen wird beschrieben.

Auf mögliche Wechselwirkungen zwischen Bekleidungsplatten und Unterkonstruktion und die Notwendigkeit einer symmetrischen Lasteinleitung wird hingewiesen.
1. GENERAL

Ventilated curtain walls consist of a wooden or metal substructure attached at a dictated distance to the external walls of a building and of panels attached on the substructure.

Panels suitable for rear attachment have to be of a minimum thickness of 8 mm and of a maximum size of about 1,5 m x 3,0 m.

High pressure laminated boards (HPL-plates), boards made of fibre cement, of crystallized glass, ceramics or natural stone are used. The boards are attached at several points to the substructure.

The easiest way of connecting is the direct attachment which will leave visible the heads of the attachment devices like nails, bolts and rivets (on board surface) or metal cramps (at board edges). This effect is often not wanted so that
the more complicated method of indirect application of the load at the back of the panel is preferred.

According to Standard [1] plates of natural stone are not attached at the surface but at the board edges by use of steel spikes. Using rear attachment devices installed at the back surface of the board the structural advantage of better use of the bending load carrying capacity of the boards (cantilevers) occurs. As a result of this effect, material can be saved and the dead load of the boards can be diminished in comparison to attachment methods used previously.

2. FORM AND FUNCTION OF ANCHORS FOR REAR ATTACHMENT OF PANELS

The only anchors providing satisfying results at tests up to now are the so-called undercut anchors.

Fig. 1: Longitudinal section of the back sloped bore hole.

For this type of anchor a firstly cylindrical borehole is widened conically according to the straddling zone of the anchor (figure 1). Then an anchor is inserted into the borehole and widened to its final form in the straddling zone until touching the sloped walls of the borehole.
Due to the lower thickness of the panels the depth of anchorage is clearly smaller than the one of anchors common in concrete. Depending on the panel thickness the depth of anchorage varies from 7 to 15 mm. The main geometrical difference in comparison to concrete anchors manifests in the ratio of anchor diameter / anchorage depth of approx. 1,0 - 1,5. For concrete anchors this ratio is clearly lower than 1, i.e. the anchors used in panel boards have a wider diameter than those used in concrete.

Superposed bending loads out of deflection have a greater influence on the load bearing capacity of anchors in thin boards than in virtually stiff concrete. This is a result of high flexural stresses appearing around the attachment points of thin panel boards that superpose with tension stresses resulting from anchor loading.

Anchors officially allowed for use in various board materials are shown in figures 2 and 3. The anchors shown are produced by the companies fischerwerke [2, 3, 4] and KEIL [5] respectively. For both companies the tests required for admittance were performed at the FMPA, Referat 23, as were the tests for the so-called Quadro-System by AGROB-BUCHTL [6] for attachment of ceramic Makropanels as shown in figure 6.

In this system a bolt is fitted into a small ceramic block so that its shaft is sticking out at one end. The ceramic block is sintered in a burning oven with special glass solder onto the back of a ceramic plate.

The concept of straddling used by the fischer panel anchor (figures 2a, 2c, 2d) consists of a festoon shaped straddling element that is slid over the tampered top of a threadbold sitting on the bottom of the borehole. Usually this is done by simultaneously screwing down a nut and pressing the anchor against the bottom of the hole. Regarding ceramic anchors the nut is pressing on a plastic sleeve adjacent to the straddling festoon, regarding natural stone anchors on a steel sleeve.
The subsequently annexed part is fixed with an additional nut (figure 4). The projecting length of the steel sleeve of the natural stone anchor over the stone surface can be varied (figure 4b). Consequently, varying thickness of stone plates can be adjusted up to about 4 mm by appropriate choice of depth of anchorage. In this way the visible surface of the stone plate can be provided plane even for differing thickness of the stone plates.

Alternatively, natural stone anchors can be placed flush to the stone surface by a purpose-built impact tool (figure 2d).

The rivet anchor made by fischer (figure 2b) consists of a cylindrical slit sleeve with integrated blind rivet [4]. As soon as the anchor is stayed up the widening of the straddling head of the rivet widens the sleeve and simultaneously jams the annex part (flush installation analogical to figure 5).
The KEIL anchor [5] (figure 3) consists of a slit sleeve with an internal thread of a conical form at its lower part according to the borehole geometry. For installation the sleeve is compressed at its lower end (figure 5). The sleeve is placed into the borehole and widened to its original shape by screwing down a fixation screw holding the annex part (figure 5).

An existing grip between the head of the fixing screw and the upper end of the anchorage sleeve is important for this system. Otherwise, e.g. in case of cavity (the annex element props up to the side of the anchor) undefined tension strain is put on the anchor when the fixation screw is tightened.
Regarding the Quadro-system [6] in figure 6 the necessary annex parts are attached with a nut to the screw sticking out of the ceramic block.

Fig. 6: Quadro-system by AGROB-BUCHTAL displaying a sintered ceramic block incl. agrafe and substructure

3. SUSPENDING STRUCTURES

There are two different possibilities:

Either single agrafes are screwed on the anchors and during assembly of the panels are hooked into horizontal fixing rails attached to the building (figure 4a, 5 and 6) or fixing rails are connected with 2 or more aligned anchors (figure 4b) and subsequently attached to the building. In the licences [2 to 6] the use of
single agrafes is limited to 6 to max. 8 agrafes per board out of reasons of safety of assembly.

For both single agrafes and fixing rails it has to be observed that the anchorage forces are applied and transferred centrally, i.e. normally symmetrical sections with adequate flexural and torsion rigidity are to be used. Using agrafes that can be hooked in only on one side or angle sections that can contort and are supported by the panels only with short lever arms to the anchor axis, the anchors may be loaded due to the lever action with a multiple of the calculated wind suction.

In the panel plane a widely unrestricted room for displacement due to expansion of the panel with changing temperature and / or humidity must be given. This can be achieved by fixing only one support and leaving the other supports in board plane free. Using hooking-agrafes (e.g. figure 6) it normally suffices to cancel the moveability given at an agrafe and so form a fixed-point. In fixing rails oblong holes have to be provided at the sliding points.

4. INTERACTION BETWEEN PANELS AND SUBSTRUCTURE

Especially for panels with more than 4 attachment points the actual stress of the anchors can differ largely from the stress estimated for a restrained support of the panel. This occurs as a result of the varying deformation of each fixing point of the substructure. As a consequence of this ‘joint displacement’ at the attachment points the loads can be changed up to 100 % compared to restricted supports [7].

As the stiffness of the panel board and the substructure, possible yielding of the anchor zone, twisting of the sections of the substructure and the distances and situation of the anchor points have to be taken into consideration, a reliable estimation of the performance of the total structure is only possible by testing elements under actual conditions.
5. TESTS FOR ESTIMATION OF THE PERFORMANCE OF REAR ATTACHMENT ELEMENTS ON PANELS

5.1 Aptitude tests

To determine the principal aptitude of an undercut anchor the tolerances of fabrication of boreholes have to be determined. Back sloped boreholes are made with specific drills, resp. drilling techniques so that additional influences to the fabrication of cylindrical boreholes have to be taken into account.

The according applies to the fabrication tolerances of anchors.

With the aptitude tests derivations exceeding the planned ones, e.g. an enlargement/amplification of the borehole-diameter or a diminuition of the back slope are checked. These tests are performed to determine that the performance of an anchor does not change significantly for slight discrepancies from the specifications of the setting depth.

Furthermore the influence of a deviation of the planned setting depth on assembly and / or performance of the anchors is analysed. Taking into account the worst combination of given tolerances to be anticipated tests with swelling loads and constant long-term loads as well as tests following alternate freeze-thaw-tests of anchors installed in panel segments are performed.

5.2 Collapse load testing to determine allowable stress of anchors

The majority of collapse load tests is performed by applying a centric tension force to the anchor. For each test anchor is fixed centrally (at the intersection of the diagonals) onto a panel section and loaded thereafter. The reaction forces are applied in a circle around the anchor using a ring. The panel section is placed on the support in a way that the anchor sits in the centre of the ring support.
The influence of superposed bending stress from panel bending in the anchorage zone is determined by use of different diameters of support rings. If necessary, the influence of different given panel thicknesses and/or depths of anchorage is analysed. The effect of small distances to the panel edges on the performance of the anchors is recorded by testing the anchors in panels with side dimensions equal to twice the value of the margin provided.

Furthermore, transverse tension tests are performed loading the anchor orthogonal to its axis and where appropriate adding a bending moment - created by a shearing force applied at a defined distance to the panel surface - as well as angular tension tests with an angle of 45° to the anchor axis.

6. SOME RESULTS OF THE TESTS PERFORMED

Figures 7 to 9 show the results of tests made with anchors in HPL-panels.

Fig. 7: Influence of depth of anchorage on failure loads of undercut anchors fixed to a
HPL-panel of 14 mm thickness

Figure 7 shows the decrease of failure loads of anchors in HPL-panels depending on the depth of anchorage.

For a tested panel thickness of 14 mm and a ring support of 55 mm the performance is not influenced worth mentioning by a superposed flexural load out of panel bending. It can be noticed that a decrease of approx. 80% (from 9 mm to 2 mm) of the depth of anchorage results in a diminution of approx. 95% of the failure load (from approx. 7.5 kN to approx. 0.5 kN).

Fig. 8: Influence of panel thickness and superposed flexural loading on performance of undercut anchors in HPL-panels

Figure 8 shows the influence of superposed flexural loading resulting from panel bending. With diminishing panel thickness the failure load decreases; namely stronger for a ring diameter of 347 mm than for a diameter of 135 mm.
Figure 9 clarifies the influence of flexural load on the performance of anchors for different panel thicknesses and on different setting depths of the anchors according to the panel thickness.

The failure load decreases stronger for 14 mm panels and a ring diameter of 50 mm to 150 mm than for larger ring diameters. This can be put down to the fact that in this range the failure mode ‘conical fracture’ is ruling and that the fracture load is reduced by flexural tension stresses in the panel more strongly for this failure mode than in ranges of higher flexural loading that are ruled by the failure mode of bending failure of the panels.

Fig. 9: Influence of superponed flexural load on the performance of undercut anchors in HPL-panels of varying thickness

Figure 10 shows a graph according to figure 9 for 2 different kinds of granite called W an G with 3 different thicknesses of 25, 20 and 15 mm. First, it has to be noted that the graphs are similar for both kinds of stone but the failure
load of the anchors in stone G with a lower flexural strength compared to stone W is smaller.

In the support diameter range from 55 to 135 mm a significantly higher decrease of failure loads (out of superponed bending stress for failure mode ‘conical fracture’) can be observed than for HPL-panels according to figure 9 that show an approx. 15 times higher flexural strength than the granites tested.

Fig. 10: Decrease of the failure load of undercut anchors in granite plates of varying thicknesses subject to the ring diameter (d)
In figure 11 the failure loads for different support diameters are shown in relation to the panel thickness. The numbers next to the letters W and G respectively state the diameter of the ring supports. It can be seen that up to a diameter of 135 mm the test values follow the graph as theoretically expected for the failure mode rupture of the plate and that they start to deviate at diameter 55 mm for stone G starting with a thickness of 20 mm and with a thickness of 15 mm for stone W.

\textit{varying}
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TEMPERATURE EFFECTS ON THE STRUCTURAL BEHAVIOR OF LAMINATED SAFETY GLASS

ZUM EINFLUSS DER TEMPERATUR AUF DAS TRAGVERHALTEN VON VERBUNDSICHERHEITSGLAS

A L'INFLUENCE DE LA TEMPERATURE AU COMPORTEMENT PORTANT DU VERRE FEUILLETE DE SECURITE

Gunter Krüger

SUMMARY

Shear and bending tests were performed in the temperature range from -20°C to +40°C to investigate the characteristic viscoelastic properties of polyvinyl butyral-resin (PVB), mostly used in laminated safety glass for architectural applications. A temperature dependent coupling parameter is used to describe the structural behavior of laminated safety glass subjected to bending stresses in comparison to a monolithic plate of the same thickness.

ZUSAMMENFASSUNG


RESUME

Des essais en cisaillement et en flexion ont été effectués à la température de -20°C à +40°C afin de déterminer des valeurs caractéristiques visco-élastiques de feuille faite en polyvinyl-butyral (PVB), appliquée en verre feuilleté de sécurité. A l'aide d'un paramètre de couplage dépendant de la température, on
peut décrire le comportement portant de verre feuilleté de sécurité en flexion en comparaison avec une vitre monolithique de même épaisseur.

**KEYWORDS:** Laminated Safety Glass, PVB, Temperature, Shear Stress, Creep

1. **INTRODUCTION**

The use of laminated safety glass has considerably enlarged the bounds of possibility for construction with glass in architectural applications, in particular for safety and overhead glazings. Laminated safety glass consists of a sandwich with two (or more) sheets of glass bonded to a thin layer of a macromolecular polymer, usually polyvinyl butyral (PVB). In comparison to monolithic glass, it has an improved impact resistance and provides substantial safety in accidents. In the case of failure laminated safety glass preserves a residual load-carrying capacity, the tough interlayer has the ability hold glass fragments in place.

![Fig. 1: Schematic representation of laminated safety glass subjected to bending stresses.](image)

The structural behavior of laminated safety glass subjected to bending stresses is considerably determined by the shear transfer in the interlayer. Fig. 1 exemplary shows the deformation of a sandwich beam with rigid faces (glass plates) and a thin, low modulus core (PVB). Flexural moments are mainly carried by the glass plates. The stiffness of the core determines the amount of shear transferred between the glass plates.
Since PVB shows a distinctive temperature dependence of its mechanical properties, the consideration of a favourite shear transfer in structural design calculations is not permitted in Germany [DIBt, 1998]. At a temperature $T=23^\circ\text{C}$ the PVB resin for architectural applications requires a tensile strength of $\geq 20\ \text{N/mm}^2$ and an elongation at break of $\geq 250\%$.

At room temperature PVB behaves viscoelastic. Relaxation and a dependence on the duration of loading is observed. Under long-term loading PVB has the tendency to creep. With falling temperature the shear moduls increases. Below the glass transition temperature the material becomes elastic with low damping and high tensile strength at break.

The experimental investigations reported in this paper, provide the temperature dependence of the material data of PVB and the structural behavior of laminated safety glass required as input data for calculations and for practical engineering purposes.

2. TESTS

2.1 Creep Tests in Simple Shear

The test setup is schematically shown in fig. 2. The test specimen with a cross-sectional area of 12mm*20mm was cut from a laminated safety glass with the construction 4 mm glass / 0.76 mm PVB / 4 mm glass. For a time period of 1000 s a load $F = 50\ \text{N}$ produced a constant shear stress $\tau = 0.2\ \text{N/mm}^2$ in the interlayer. The displacement $u(t)$ between the glass plates was measured with a LVDT during the load was applied and during further 1000 s of free relaxation after the load was released. The time-dependent shear $\gamma(t)$ was calculated with the nominal thickness $t_f = 0.76\ \text{mm}$ of the PVB resin.

Fig. 2: Shear test.

The decrease of the shear stiffness during the load $F$ is applied (time interval $0 \leq t \leq 1000$ s) can be described by

$$\gamma(t) = \tau \cdot f(t)$$

(1)

with the 4-parameter creep function

$$f(t) = a + b \cdot t + c \cdot [1 - \exp(-\lambda \cdot t)]$$

(2)

The parameter $a$ represents the time independent elastic part of the shear that follows instantaneously the external load. The shear modulus resulting from this pure elastic deformation is given by $G = \tau a$.

The linear term describes irreversible creep with a creep velocity $v = b/\tau$. The relaxation time $t = 1/\lambda$ gives the time span within the reversible part of the
shear deformation recovers. A dependence on the load level is not concerned by equ. 1.

Fig. 3: Measured shear \( \gamma(t) \) as a function of time.

Fig. 3 demonstrates the quite different behavior of PVB at temperatures \( T = \pm 0^\circ C \) and \( T = +40^\circ C \). With increasing temperature the shear stiffness decreases rapidly and creep becomes more significant. The parameters \( a, b \) and \( \lambda \) were evaluated by a fit of equ. 1 to the experimetal data in the interval \( 0 \leq t \leq 1000 \) s. Although the tests are not suitable for a precise determination of the shear modulus, the principal tendency as a function of temperature can be evaluated.

The shear modulus decreases about two orders of magnitude in the considered temperature range (fig.4), while the creep velocity \( v(T) \) increases (fig. 5) from \( v = 0 \) to the about of \( 10^{-3} \) mm\(^3\)/Ns. The relaxation time \( t = 1/\lambda \) was about 50 s to 200 s.
2.2 Bending Test

Fig. 4: Shear modulus $G(T)$

Fig. 5: Creep velocity $v(T)$
Fig. 6 shows the test setup for 3-point bending. A concentrated load $F=20$ N was applied at the midspan if a beam (span $L=400$ mm, width $B=30$ mm) consisting of laminated glass (4 mm glass / 0.76 mm PVB / 4 mm glass). The center deflection $u(t)$ was measured by a LVDT. The temporal procedure was the same as described in cap. 2.1.

The center deflection can be calculated by

$$u = \frac{F}{48 \cdot (E \cdot I)_{beam}} \cdot L^3$$

(E and I are the Young’s modulus and the moment of inertia of the beam. The moments of inertia of the glass plates $I_g$, of the core $I_f$ and the coupling term $I_c$ contribute to the total amount of the moment inertia $I$. The cross section area of the beam is shown in fig.6.

$$(E \cdot I)_{beam} = E_g \cdot I_g + E_f \cdot I_f + E_c \cdot I_c \approx E_g \cdot I_g + E_c \cdot I_c$$

$$I_g = 2 \cdot B \cdot t_g^3/12, \ I_f = B \cdot t_f^3/12, \ I_c = \kappa \cdot B \cdot (t_g + t_f)$$

Since $I_f \ll I_g$, the moment of inertia of the core can be neglected. The amount of coupling by the core is given by the parameter $\kappa$ with $0 \leq \kappa \leq 1$. In the
limiting case $\kappa = 1$ the total moment of inertia becomes equal that of a monolithic beam with the thickness $t = 2 \cdot t_g + t_f$. $\kappa = 0$ describes the case of zero coupling with $I = I_g$.

![Cross section](image)

**Fig. 7: Cross section**

The measured displacements $u(t)$ at temperatures $T = \pm 0^\circ C, +10^\circ C, +20^\circ C$ and $+40^\circ C$ are given in fig. 8. At low temperatures PVB and from that laminated glass behaves mostly elastic. Due to the decrease of the shear stiffness of the PVB interlayer with rising temperature the displacement $u(t)$ increases.

At temperatures above $40^\circ C$ the coupling between the glass plates is very small and the deformation of the laminated glass beam is determined mainly by the elastic deformation of the glass plates. It is obviously that creep becomes mostly significant at mid temperatures about $+10^\circ C$ to $20^\circ C$. Due to this, the bending stiffness of laminated glass is mostly influenced by the duration of loading in this temperature range.
Temperature effects on the structural behaviour of laminated safety glass

Fig. 8: Measured center deflection $u(t)$ as a function of time for different temperatures between $0^\circ C$ and $40^\circ C$
The fig. 8 gives the parameter $\kappa(T)$ that was calculated by means of the equi's. 4 and 6 and the measured displacement $u(t = 10s)$ and $u(t = 1000s)$. Not the nominal, but the actual values of $t_g$ and $t_f$ were used. The actual Young's modulus $E_g$ was determined from a comparative test with a monolithic glass plate.

Fig. 9: Coupling parameter $\kappa(T)$ for a load duration $t = 10 s$ and $t = 1000 s$
CONCLUSIONS

A 4-parameter viscolestic model was used to explain the structural behavior of PVB under long-term loading in simple shear. The shear modulus of PVB evaluated from the experimental data decreases about two orders of magnitude in the temperature range between -20°C and +40°C. The creep velocity increases with increasing temperature to about $10^{-3}$ mm$^3$/Ns at +40°C.

The structural behavior of laminated glass subjected to bending stresses is considerably determined by the shear transfer in the PVB interlayer. According to the temperature dependence of the shear modulus of PVB, laminated safety glass behaves as a monolithic plate at temperatures below ±0°C, at high temperatures above +40°C the behavior comes close to the limiting case with no coupling between the glass plates. Load-duration effects are mostly important at temperatures about 10°C to 20°C.
TESTS ON REPRODUCED BYZANTINE MASONRY

VERSUCHE AN NACHGESTELLTEM BYZANTINISCHEN MAUERWERK

ESSAIS SUR MAÇONNERIE BYZANTINE REPRODUITE

Holger Falter, Hans-Wolf Reinhardt

SUMMARY

Tests have been performed on reproduced Byzantine masonry with one type of brick and one mortar composition. The joint thickness has been varied between 20 and 60 mm. The storage conditions either standard climate 20°C/65% RH or sealed in aluminium foil. Incremental loading shows a strong plastic behaviour of the joints. Short term loading reveals clearly the influence of joint thickness on the mechanical behaviour.

ZUSAMMENFASSUNG


RESUME

Des essais ont été réalisés sur une maçonnerie byzantine reproduite avec un type de brique et une composition de mortier. L'épaisseur des joints a été variée entre 20 et 60 millimètres. Le stockage des spécimens a eu lieu soit sous climat standard 20°C/65% d'humidité relative, soit sous enveloppe hermétique en papier d'aluminium. Le chargement par paliers montre un fort comportement
plastique des joints. Le chargement de courte durée révèle clairement l'influence de l'épaisseur des joints sur le comportement mécanique.

KEYWORDS: ancient masonry, testing, strength, deformation, Byzantine church, San Vitale

1. MOTIVE

In Roman times and later during the Byzantine period (Vth to XIVth century), thick joint loadbearing brickwork was the general building techniques in representative buildings. Many of these buildings survived and are still in operation such as S. Michele in Africisco and San Vitale in Ravenna. There is not much knowledge about the structural behaviour of brickwork with thick joints and there is even less known about the effect of this building technique on the structure during construction. Therefore a project was started in cooperation with the Politecnico di Milano which was aimed at the experimental investigation of reproduced Byzantine masonry. The material was reproduced in Italy and the mechanical testing was carried out in the Otto-Graf-Institute in Stuttgart. The short report deals with the latter. For more information see [FALTER ET AL., 1998].

2. TESTING PROGRAMME

The testing programme was designed to simulate the construction procedure of a main column in San Vitale. The dead load is increased in steps of 20 kg during the first week, in steps of 30 kg in the 2nd week, in steps of 60 kg every 48 hours from the 30th day, and, finally, 120 kg steps every 48 hours from the 60th day. The maximum load was supposed to be 6 t or 60 kN.

The mortar consists of lime as the binder, quartzitic sand and crushed fired clay brick. It has been demonstrated that the crushed brick is puzzolanic. Great
emphasized was placed on the workability which should be plastic. It was measured on the DIN 1048 spread table and should amount to 320 mm. However, some mortars were a little stiffer, others a little softer.

Bricks were reproduced with the dimensions 510 mm x 310 mm x 40 mm. The specimens consisted of 4 bricks and 3 mortar joints. The joint thickness varied between 20 and 60 mm. The storage conditions were either standard climate of 20°C and 65% RH or wrapped in aluminium foils. Table 1 shows the variation of parameters.

Table 1: Testing variables

<table>
<thead>
<tr>
<th>Specimen nr.</th>
<th>Joint thickness mm</th>
<th>Environment</th>
<th>Consistency ¹⁾</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20</td>
<td>20°C/65% RH</td>
<td>0</td>
</tr>
<tr>
<td>2/1</td>
<td>40</td>
<td>20°C/65% RH</td>
<td>-</td>
</tr>
<tr>
<td>2/2</td>
<td>40</td>
<td>20°C/65% RH</td>
<td>+</td>
</tr>
<tr>
<td>2/3</td>
<td>40</td>
<td>20°C, wrapped</td>
<td>0</td>
</tr>
<tr>
<td>3/1</td>
<td>60</td>
<td>20°C/65% RH</td>
<td>-</td>
</tr>
<tr>
<td>3/2</td>
<td>60</td>
<td>20°C/65% RH</td>
<td>+</td>
</tr>
</tbody>
</table>

¹⁾ 0 as anticipated
+ a little softer
- a little stiffer

The loading rig consisted of a lever and a loading weight. The magnification of load was 1:5 in the lower range and could be changed to 1:40 in the higher range. Fig. 1 shows a schematic of the loading equipment. Dial gauges with a resolution of 1 µm were mounted on the four corners of the specimens.
After the tests with incremental loading, four specimens were subjected to monotonic loading until failure. These tests were carried out in a displacement-controlled hydraulic testing machine with displacement rate of 0.02 mm/s. The displacement was recorded at the corners of the specimens by four LVDTs.

3. TEST RESULTS

3.1 Incremental loading

The results of the incremental loading are summarized in Fig. 2. The horizontal axis shows the time from beginning of loading. The left vertical axis shows the strain with respect to the joints, i.e. it is assumed that the brick deformation is negligible with respect to the deformation of the joints. The right vertical axis shows the stress which increases incrementally. It can be seen that the strain increases very fast in the first 20 days despite the rather small stress. Prisms 3/2 and 3/1 deform most because the joint in thicker than of prisms 2/1 and 2/2 and 1.
Since prism 2/3 is lacking behind prisms 2/1 and 2/2 it should be assumed that drying shrinkage plays a major part in the deformation during the first 20 days. When stress increases, the joints of prism 2/3 are still water saturated and deformation increases more than that of prism 1, 2/1, 2/2, 3/1 and 3/2. The stress-strain behaviour of the mortar in the joint is the effect of various phenomena: compaction due to water absorption of the bricks, consolidation due to seepage of water, shrinkage due to drying, carbonation, plastic deformation in the fresh state, elastic deformation, and viscous creep in the early age and later on. It is really a complex behaviour which has to be studied more carefully in the future.

The large plastic deformation in the fresh state, i. e. during construction, have the great advantage that masonry could settle and could adjust itself to a state almost free of stresses due to imposed deformation. This is important because the brittle bricks did not encounter stress concentrations and did not break.

3.2 Short term loading to failure

When a layered structure is loaded it is important to know the individual properties of the layers. In the case of Byzantine masonry the joint is rather soft compared to the brick. The brick is brittle and has a low tensile strength. In this
combination it is well known that the joint expands due to Poisson's effect and imposes tensile stresses on the brick. On the other hand, the brick reduces horizontal expansion of the joint and induces compressive stresses in the joint. It is a typical interaction of two completely different materials.

Fig. 3 shows the stress-strain response of four prisms loaded after 1 year preloaded in the incremental loading test.

Prism 3/1 and 3/2 with 60 mm mortar joint show a rather weak behaviour. Prism 2/3 which was wrapped and could not dry and not carbonate is very similar to the prisms with 60 mm mortar joint. Prism 2/2 is much stiffer than the others. It can be concluded that joint thickness and environment have a distinct influence on the stress-strain behaviour of masonry.

The sequence of pictures 4a to 4c shows the failure of masonry prisms.
Fig. 4:  Cracking and failure pattern of masonry prisms
a) Prism 2/2 at $\sigma = 2.5$ MPa
b) Prism 3/1 at $\sigma = 2.9$ MPa, prepeak
c) Prism 3/1 at $\sigma = 2.8$ MPa, postpeak
Fig. 4a shows the beginning of splitting cracks in the brick due to transverse expansion of the mortar. Picture 4b illustrates a situation when splitting cracks in the brick coincide with spalling of the mortar joint. Finally, picture 4c is a typical core shaped fracture due to friction on the loading platens and expansion of the mortar joint. However, it should be mentioned that the stresses reached at failure are far beyond the stresses in a real Byzantine structure. For example, the stress in the main column of San Vitale does not reach more than 0.4 MPa. The failure stress of prism 3/1 was about 3 MPa.

The results of incremental tests and short term tests have been discussed broadly and the effects on construction and long term structural behaviour have also been shown in [FALTER, 1998].

4. CONCLUSIONS

Tests on reproduced Byzantine masonry has shown that the plastic behaviour of lime mortar with crushed brick has a rather favourable effect on the structural behaviour during construction and long term. The very good state of preservation of churches and other buildings after more than thousand years is certainly due to the ancient building technology which used thick joints in masonry structures. The reported investigation should give rise to more thorough and systematic treatment of the subject which would help understanding the structural behaviour of durable ancient structures.

ACKNOWLEDGEMENT

The cooperation with Prof. Binda and Prof. Baronio of Politecnico di Milano is gratefully acknowledged.
REFERENCES


HIGH STRENGTH CONCRETE UNDER SUSTAINED TENSILE LOADING

HOCHFESTER BETON UNTER DAUERZUGLAST

BETON DE HAUTE RESISTANCE SOUS TENSION SOUTENU

Hans-Wolf Reinhardt, Tassilo Rinder

SUMMARY

A research programme is described which aims at the investigation of the short term and long term tensile strength of high-strength concrete. The loading arrangement is illustrated and first test results are given.

ZUSAMMENFASSUNG

Im folgenden wird ein Forschungsprogramm zur Untersuchung der Kurzzeit- und Langzeitzugfestigkeit von hochfestem Beton beschrieben. Die Versuchseinrichtung sowie erste Ergebnisse werden vorgestellt.

RESUME

On décrit un programme de recherche qui vise la recherche sur le court terme et la résistance à la traction à long terme du béton de haute résistance. L'agencement de chargement est illustré et les premiers résultats d'essai sont donnés.

KEYWORDS: Tensile loading, creep, high strength concrete
1. MOTIVE AND SCOPE

Usual design formulas for structural concrete neglect tensile strength of concrete as a relevant quantity for the bearing capacity of a structure. Some codes state at least a relation between minimum reinforcement and tensile strength in order to prevent collapse of those members which do not need much reinforcement to carry dead and life load, but which may be subject to imposed deformation due to shrinkage and/or temperature. However, every structure relies on tensile strength to a certain extent. If concrete is uncracked tensile strength governs shear and torsional capacity and it is mainly responsible for bond of steel in concrete and thus for anchorage length and cover to reinforcement. Unreinforced structures such as pavements rely directly on tensile strength since traffic load, temperature and shrinkage induce tensile stresses. The serviceability state is often related to crack formation and cracks appear when the tensile strength of concrete is reached. If loads are permanent tensile stresses and they should be compared to tensile strength as determined under sustained loading.

There was some literature attainable on the tensile strength of normal-strength concrete under sustained loading [1, 2, 3, 4]. The experimental results indicate that sustained loading reduces tensile strength by about 40% depending on age at loading and environment. These are two effects which are opposite to each other: tensile stress causes formation and extension of micro-cracks which lead finally to unstable crack propagation on one hand, but there is also continuous hydration which increases strength and may even heal micro-cracks if water is available.

Which mechanism is prevailing and whether the effect of both mechanisms is the same for normal-strength and high-strength concrete is not known. One may argue that micro-cracking is retarded in high-strength concrete because the properties of matrix, aggregate and interfacial zone are closer together. On the other hand, once a crack starts to propagate crack arrest may be less probable and continuous hydration is less pronounced in high-strength concrete than in normal-strength concrete. Thus, there are reasons to believe that sustained loading
High strength concrete under sustained tensile loading

influences strength not in the same way. Since no test results could be found in the literature on this question it was decided to start a series of tests which will be presented and discussed as far as they have been performed already.

2. RESEARCH PROGRAMME

The programme contains mainly two variables: the concrete strength and the loading level. The strength classes are C55/67, C70/85, C90/105 according to prEN 206. Specimens are loaded at a sustained level of 75, 80, 90 and 95% of the mean short term uniaxial strength. Four specimens will be used per variable combination. Sustained loading is also realised as axial loading in a climate controlled room. Shrinkage will be measured on companion specimens without mechanical loading. Testing proceeds either to failure of the specimen or to two years at the longest.

3. TEST SET-UP

3.1 Loading arrangement and specimen geometry

Many years ago [5, 6] creep and relaxation tests have been performed in the Otto-Graf-Institute on concrete in compression. The loading arrangement has been changed to tensile load for steel testing after some years. It is now adjusted to concrete tensile loading by changing the clamping devices and the hinges on top and bottom of the specimen. Fig. 1 shows the schematic of the loading arrangement.

A water container with about 1 m³ volume is filled up and serves as load which acts, via a lever and an almost frictionless hinge, on the specimen. The multiplication factor can be varied between 3 and 19. The specimen is tensioned by a rod which is connected to a load cell and a ball hinge. There is another hinge on the bottom between specimen and support. When the specimen is being loaded
water can be filled stepwise into the container until the specified level is reached or, when the dead weight of the container is large enough, the container is gradually released from its fixation.

Fig. 1: Loading arrangement, dimensions in mm

To load a concrete specimen in uniaxial tension is always a delicate question. In the past, several methods have been used: mechanical grips on a dogbone shaped specimen [7], metal loading plates glued to the ends [8], inserts in the specimen [9], a threaded layer [10]. All of them try to introduce the force centrically and such that the fixing device does not lead to premature failure. In the present investigation, lifting anchors are concreted in the specimen as shown in fig. 2. The anchors have an inner thread M 24 and a cross-bar which ensures reliable anchorage. They are cheap and readily available.
The length changes of the loaded specimens and the length changes of the shrinkage specimens are recorded by LVDTs. All data, i.e. half-bridge arrangement of LVDTs and temperature and humidity from the Almemo unit are logged directly into a dedicated computer terminal and stored in a PC 486/33 with 8 MB RAM.

The specimens have a prismatic section with a cross-section of 80 mm x 100 mm and a length of 300 mm. The distance of the measuring points is 300 mm. LVDTs are fixed to these points.
3.2 Specimen preparation

The specimens are cast in a horizontal position. After concreting, they remain in the steel mould for one day. After demoulding, they are stored in a fog room (20°C) for six days and then, sealed with plastic foils, in a conditioned room (20°C, 65% RH) until beginning of testing at an age of 28 days.

Besides the tapered tension specimens there were 100 mm and 200 mm cubes fabricated and stored according to DIN 1048, i.e. 7 days wet and 21 days at 20°C/65% RH.

4. CONCRETE COMPOSITION

There are four concrete grades the composition of which are given in Table 1. The cement is a rapid hardening portland cement class CEM I 42.5 R according to DIN 1164 (ENV 197). Silica fume is added as a slurry. The water content in Table 1 is given as the sum of added water and water from silica slurry, superplasticizer and retarder.

The properties of fresh concrete and the dry density after 28 days are given in Table 2. Since the projected strength partially was not achieved or was substantially exceeded, for the further tests modified mixtures are used. Also the slumps were too low, which requires the use of much more superplasticizer. The experience showed that then an addition of retarder becomes unnecessary.
Table 1: *Concrete composition in kg/m³*

<table>
<thead>
<tr>
<th>Component</th>
<th>Concrete Mix 1</th>
<th>Concrete Mix 2</th>
<th>Concrete Mix 3</th>
<th>Concrete Mix 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement CEM I 42.5 R</td>
<td>400</td>
<td>450</td>
<td>450</td>
<td>453</td>
</tr>
<tr>
<td>Water ¹)</td>
<td>168</td>
<td>128</td>
<td>140</td>
<td>125</td>
</tr>
<tr>
<td>Aggregate</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0-2 mm</td>
<td>634</td>
<td>602</td>
<td>453</td>
<td>584</td>
</tr>
<tr>
<td>2-4 mm</td>
<td>320</td>
<td>208</td>
<td>324</td>
<td>206</td>
</tr>
<tr>
<td>4-8 mm</td>
<td>343</td>
<td>359</td>
<td>309 (Liapor)</td>
<td>374</td>
</tr>
<tr>
<td>4-8 mm</td>
<td>464</td>
<td>607</td>
<td>474</td>
<td>636 (Basalt)</td>
</tr>
<tr>
<td>Silica fume ²)</td>
<td>25</td>
<td>40</td>
<td>46</td>
<td>45</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>11.0</td>
<td>20.0</td>
<td>13.8</td>
<td>18.0</td>
</tr>
<tr>
<td>Retarder</td>
<td>2.9</td>
<td>1.8</td>
<td>4.1</td>
<td>5.4</td>
</tr>
<tr>
<td>Water-cement ratio</td>
<td>0.42</td>
<td>0.28</td>
<td>0.33</td>
<td>0.28</td>
</tr>
<tr>
<td>Water-binder ratio</td>
<td>0.40</td>
<td>0.26</td>
<td>0.30</td>
<td>0.25</td>
</tr>
</tbody>
</table>

¹) Total water, i.e. water of silica slurry, superplasticizer and retarder included
²) Dry mass of slurry

Table 2: *Properties of fresh concrete*

<table>
<thead>
<tr>
<th>Property</th>
<th>Concrete Mix 1</th>
<th>Concrete Mix 2</th>
<th>Concrete Mix 3</th>
<th>Concrete Mix 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spread table flow, cm</td>
<td>34</td>
<td>56</td>
<td>35</td>
<td>32</td>
</tr>
<tr>
<td>Air content, %</td>
<td>2.8</td>
<td>0.6</td>
<td>2.7</td>
<td>2.7</td>
</tr>
<tr>
<td>Density of fresh concrete, kg/m³</td>
<td>2260</td>
<td>2480</td>
<td>2200</td>
<td>2440</td>
</tr>
</tbody>
</table>
5. FIRST TEST RESULTS

The connection between loading device and specimen is realized via a hollow anchor bolt (lifting anchor) and a bolt. After 28 days, the specimens with the geometry as shown in fig. 2 are tested by a monotonically increasing load. During the loading the expansions are measured with two or four LVDTs (fig. 3). The specimens normally fail due to cracking in the prismatic section as shown in fig. 4.

Fig. 3: Test set-up with 2 LVDTs

The measured Young’s moduli ($E$) and tensile strengths in the tapered specimens with respect to the reduced cross-section are given in table 3. These values agree well with the formulas of Heilmann [11]:

\begin{align*}
E_{\text{modulus}} &= \text{formula} \\
t_{\text{tensile strength}} &= \text{formula}
\end{align*}
High strength concrete under sustained tensile loading

\[ f_t = a \cdot f_c^{2/3} \]  \hspace{1cm} (1)

and Remmel [14]:

\[ f_t = b \cdot \ln\left(1 + \frac{f_c}{10}\right) \]  \hspace{1cm} (2)

with \( f_t \) = tensile strength, \( f_c \) = compressive strength, and the constants \( a \) resp. \( b \). The German guideline for high-strength concrete suggests \( 0.18 < a < 0.21 \) [12].

Fig. 4: Specimens after failure in short-term and long-term tensile test
Table 3: Mechanical properties of concrete at 28 days
Some specimens were loaded with 80, 90 and 95% of their short term tensile strength and the strains were measured. The times until failure are listed in table 4. Specimens loaded with 80% of their short term tensile strength did not fail within 90 days. The measured creep behavior of one test specimen (ratio 0.90) is exemplary shown in fig. 5.

Table 4: Service lives under sustained tensile loading (single test results)

<table>
<thead>
<tr>
<th>Ratio</th>
<th>Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mix1</td>
</tr>
<tr>
<td>0.90</td>
<td>1 h</td>
</tr>
<tr>
<td>0.95</td>
<td>0 h</td>
</tr>
</tbody>
</table>

6. CONCLUSION AND OUTLOOK

The testing programme has yet started which makes that only short-term results and a few long-term test results are available. The short-term results are within the range which has been predicted by empirical formulae. The sustained loading results will be evaluated according to [13] who has based the theoretical prediction on the Griffith criterion for brittle materials. The available test results show already that high-strength concrete creeps little due to its brittleness and is able to bear comparatively higher sustained loadings than normal concrete.
Fig. 5: Tensile creep of high strength concrete (Mix 2, ratio = 0.90) with I, II, III primary, secondary, and tertiary stage

REFERENCES


GLUED-IN HARDWOOD DOWELS AS AN ALTERNATIVE TIMBER END-JOINTING DEVICE

EINGELEIMTE HARTHOLZDÜBEL ALS ALTERNATIVES VERBINDUNGSMITTEL FÜR HOLZTRÄGERSTÖSSE

GOUJONS DE BOIS DUR COLLES COMME ASSEMBLAGE ALTERNATIF POUR POUTRES DE BOIS

Kohei Komatsu

SUMMARY

In this report, as a recent research topic in the field of Japanese timber engineering, a glued-in hardwood dowels joint was introduced. Tensile strength of glued-in dowel joints was found to be controlled by two parameters, one of which is the glue line shear strength $f_{vs}$ and another is shear stiffness $\Gamma$ which was defined as a proportional coefficient between glue line shear stress $t$ and relative displacement between dowel and wood member.

From pull-out tests and push-out tests, glue line shear strength $f_{vs}$ was estimated as 7.6 to 9.4 MPa for polyurethane adhesive and 10.9 to 12.9 MPa for epoxy adhesive in the case of Japanese maple dowel and Japanese cedar main member. Shear stiffness $\Gamma$ evaluated from the two different test methods varied from 9.3 to 43.6 N/mm$^3$ for polyurethane adhesive and 45.2 to 73 N/mm$^3$ for epoxy resin adhesive. Flexural properties of glulam beams, which were end-jointed by glued-in hardwood dowels, were analysed theoretically and evaluated empirically using glued-in dowel jointed glulam beams of 100 mm x 200 mm cross section and 2700 mm total span length made of Japanese cedar. Good agreements were obtained between theoretical prediction and experimental observation.
ZUSAMMENFASSUNG

In diesem Bericht wird als aktuelles Forschungsprojekt des japanischen Ingenieurholzbaus ein mittels eingeleimten Hartholzdübeln ausgebildeter Trägerstoß vorgestellt. Hierbei wurde festgestellt, daß die Zugfestigkeit dieser geklebten Hartholzdübelverbindung von zwei Parametern bestimmt wird. Zum einen ist dies die Schubfestigkeit $f_{vs}$ der Klebefuge, zum anderen die Schubsteifigkeit $\Gamma$, die als proportionaler Koeffizient aus Schubspannung $\tau$ der Klebefuge und relativer Verschiebung zwischen Dübel und umgebendem Holz definiert ist.

Aus Auszug- und -druckversuchen wurde die Schubfestigkeit $f_{vs}$ der Klebefuge für Dübel aus japanischem Ahorn und Prüfkörper aus japanischer Zeder zu 7,6 - 9,4 MPa für Einkomponenten-Polyurethanklebstoff, und zu 10,9 - 12,9 MPa für Epoxidharzklebstoff bestimmt. Die Schubsteifigkeit $\Gamma$, die aus den Zug- und Druckversuchen bestimmt wurde, varierte im Bereich von 9,3 - 43,6 N/mm³ für den Polyurethanklebstoff und von 45,2 - 73 N/mm³ für den Epoxidharzklebstoff. Das Verformungsverhalten von Brettschichtholzträgern, die auf diese Weise mit Dübeln verbunden sind, wurde sowohl analytisch als auch experimentell an Prüfkörpern von 100 x 200 mm Querschnitt und 2700 mm Länge untersucht. Es wurde eine gute Übereinstimmung zwischen Rechnung und Versuch erzielt.

RESUME

Dans ce rapport, un assemblage de poutres formée par moyen de goujons collés de bois dur est représenté comme projet de recherche actuel de la construction de bois japonais. Par ceci, on a reconnu que la résistance à la traction de ces assemblages collés de goujons bois dur est déterminée par deux paramètres. D'une côté, c'est la résistance cisaillement du joint de collage , de l'autre côté c'est la rigidité cisaillement qui est défini comme coefficient proportionnel de contrainte cisaillement du joint collé et décalage relative entre goujon et bois entourant.

Par des essais de traction et de pression, la résistance cisaillement du joint collé pour les goujons fait d'éraie japonais et un échantillon fait de cèdre japonais de 7,6 à - 9,4 MPa pour la colle polyuréthane à 1 composante et de 10,9 à 12,9 MPa pour la colle epoxy avait été défini. La rigidité cisaillement, qui avait été défini par des essais de traction et de pression, variait entre 73 N/mm³ pour la colle epoxy . La réaction de déformation des poutres de bois lamellé-collé qui sont liés de cette manière par des goujons, avait été examiné non seulement
analytiquement, mais encore expérimentalement aux échantillons d'un diamètre de 100 x 200 mm et d'une longueur de 2700 mm. Une bonne concordance entre facture et essaie avait été atteinte.

KEYWORDS: Timber joints, glued in hardwood dowels, glued in rods

1. INTRODUCTION

Hardwood dowel might be a worth re-thinking material as an alternative jointing tool for engineered timber joints, because it can be harmonized with timber structural members more gently and naturally than such non-organic materials as steel or plastics and so on.

On the basis of above mentioned motivation, a research project team in the Institute of Wood Technology, Akita Prefectural College of Agriculture, Noshiro, Akita Prefecture, Japan has started a series of research projects in order to utilize hardwood dowels as an alternative device for end-jointing glulam beams on construction site.

In this report, as a visiting research associate of Otto-Graf-Institute, I would like to introduce some interesting research results to show how the hardwood dowel has an potential as an alternative on-site end-jointing device for glulam beams, which are now in many countries executed mainly by so-called glued-in steel bolts and/or bolted splice joints.

2. WITHDRAWAL PROPERTIES OF GLUED-IN HARDWOOD DOWEL JOINTS

A series of pull-out tests were done by [KOIZUMI ET AL, 1998a,b] using the method shown in Fig. 1.
In Table 1, some properties of the materials used in these tests are shown. For the main members, sawn timbers of Japanese cedar (Cryptomeria Japonica) having 33, 38 and 46mm x 70 and 90 mm cross section was used. For the dowels,
Japanese maple (*Acer mono*) having diameters of 8, 12 and 16mm were used. The diameter of leading hole for embedding dowels was always 1mm larger than the dowel diameters. The embedment length $l$ of dowels were varied from $2d$ to $10d$, where $d$ was the diameter of the dowel. For the adhesive, one component polyurethane adhesive, epoxy resin adhesive and resorcinol-formaldehyde adhesive were used.

<table>
<thead>
<tr>
<th>Item</th>
<th>$n$</th>
<th>Density mean $(\text{kg/m}^3)$</th>
<th>CV (%)</th>
<th>$E_f$ mean (GPa)</th>
<th>CV (%)</th>
<th>MC (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dowel</td>
<td>4</td>
<td>713</td>
<td>0.7</td>
<td>15.1</td>
<td>2.3</td>
<td>9.7</td>
</tr>
<tr>
<td>Main member</td>
<td>24</td>
<td>377</td>
<td>6.2</td>
<td>8.5</td>
<td>3.4</td>
<td>10.5</td>
</tr>
</tbody>
</table>

$n$ : Number of pieces of lumber  
$E_f$ : Dynamic Young's moduli measured by longitudinal vibration method  
MC : Moisture content

Table 1: *Properties of materials used in withdrawal tests.*

### 2.1 Summary of test results on single dowel joint

![Glued-in Hardwood dowel in wood member subjected to a tension force $Q$.](image)

Maximum pull-out force $Q_{\text{max}}$ of glued-in hardwood dowel joint shown in Fig. 2 was expressed in eq. (1), which was originally derived by [JENSEN ET AL, in press] on the basis of Volkersen-type stress analysis, with including parameters of glue line shear strength $f_{sv}$ and shear stiffness $\Gamma$ governing the pull-out strength of the joint.
\[ Q_{\text{max}} = \frac{f_{\text{vs}} \pi dl (1 + \alpha) \sinh \omega}{\omega (1 + \alpha \cosh \omega)} \]  
\[ \omega = 2 (1 + \alpha) \frac{\Gamma}{\alpha \sqrt{d \frac{E_d}{E_w A_w}}} \]  
\[ \alpha = \frac{E_w A_w}{E_d A_d} \]

where

- \( E_w \): Modulus of elasticity of wood member (see Table 1)
- \( E_d \): Modulus of elasticity of hardwood dowel (see Table 1)
- \( A_w \): Cross sectional area of wood member (see Fig.1)
- \( A_d \): Cross sectional area of hardwood dowel (see Fig.1)

As it was difficult to define the pure shear rigidity of the glue line \( G \) in the case of the timber-glue joint, shear stiffness \( \Gamma \) was defined as shown in eq. (2) in which shear stress \( \tau \) in the glue line was assumed to be proportional to the relative displacement \( d \) between hardwood dowel and the surrounding wood member.

\[ \tau = \Gamma \delta \]  

Two unknown parameters, shear strength of glue line \( f_{\text{vs}} \) and shear stiffness \( \Gamma \), could be estimated by applying a nonlinear least-squares method in eq. (1) with experimental data of maximum pull-out strength \( P_{\text{max}} \).
Shear strengths of glue line $f_{vs}$ obtained directly from push-out tests shown in Fig. 3 were close to those estimated through pull-out test.

On the other hand, shear stiffness $\Gamma$ observed in push-out tests was almost two times of those estimated through pull-out tests. This is because $\Gamma$s through pull-out tests were estimated at the maximum load level while $\Gamma$s through push-out test were defined as the initial stiffness.

Values of shear strength of glue line $f_{vs}$ and shear stiffness $G$ obtained from the two different test series are compared in Table 2.
Among the three adhesives tested, polyurethane adhesive joints showed the highest maximum pull-out strength values.

The optimum shear strength of the glue-line seemed to be about 10MPa by taking the tensile strength of dowel itself into account.

Optimum shear stiffness $\Gamma$ seemed to be about 10N/mm$^3$ by taking the limit length and tensile strength of hardwood dowels into account.

### 2.2 Preliminary Comparisons with Glued-in Steel Rod Joints

At present, a series of pull-out tests on glued-in steel rod joints are being executed [AICHER ET AL.] in the department of wood and timber engineering, Otto-Graf-Institute, as shown in Fig.4.
It is interesting for the author to compare the strength properties of similar jointing methods. Unfortunately, however, the conditions between these two test series are too different to compare them rigorously, so very rough comparisons were attempted by defining the following "apparent average shear stress $\tau_{ave}$":

$$\tau_{ave} = \frac{Q}{\pi dl}$$
Q: Applied tensile force on dowel

\[ d: \text{ diameter of dowel} \]

\[ l: \text{ embedment depth of dowel} \]

Fig. 5 shows comparisons between stress(\( \tau_{\text{ave}} \))-relative displacement (\( \delta \)) relationships for glued-in hardwood dowel joints and glued-in steel rod joints. It is interesting to see that even if quite different materials are used for glued-in dowels or rods, almost same orders of stress-relative displacement relationships were obtained. This is probably because strength and stiffness of glued-in dowel type joints might be affected much more by the mechanical properties of the adhesive used and less by those of the dowel type material and/or surrounding wood member.

![Graph showing stress-relative displacement relationships](image)

Fig. 5: *Comparisons of stress(\( \tau_{\text{ave}} \))-relative displacement(\( \delta \)) relationship.*

Wood members for glued-in steel dowel joint are European spruce glulam of 120 mm x 120 mm cross section.

2.3 Summary from multiple dowel joint tests
Withdrawal strength ought to be increased as the diameter of dowel increases, this prediction was coincident with experimental results from 8 to 12 mm dowels. In the case of 16 mm dowel, however, dowel failures were dominant.

The effect of MOE of the wood member surrounding the dowel(s) was less important in the range of experiments.

For dowel spacing, twice the dowel diameter seemed to be sufficient.

Withdrawal strength for multiple dowels joint was found to be about 80% of that for a single dowel joint.

3. BENDING STRENGTH AND STIFFNESS OF GLULAM BEAMS END-JOINTED WITH GLUED-IN DOWELS

A glued-in hardwood dowel joint was first applied as an end-joint for glulam beams. At first, theoretical analysis for predicting bending strength and stiffness of end-jointed glulam beams were done by the author [KOMATSU, 1997] and also [SASAKI ET AL].

3.1 Analytical Aspects

The process for deriving the strength of the beam was essentially based on the concept for reinforced concrete beams (Fig.6), however, slip displacement between hardwood dowels and glulam member had to be considered in order to derive a more realistic behaviour typical to a glulam beam which was semi-rigidly jointed by an elastic adhesive.
For example, a maximum bending moment of glulam beams which were end-jointed with single row of glued-in hardwood dowels at the outer tensile side as shown in Fig. 6 might be predicted by eq.(3) [KOMATSU, 1997; SASAKI ET AL].

\[
M_{\text{max}} = nQ_{\text{max}} \left( g - \frac{\lambda}{3} \right)
\]

where \( Q_{\text{max}} \) is the maximum tensile strength of a single dowel joint and could be expressed as follows by assuming \( \alpha \) (refer to section 2) to be infinite for a safety side approximation.

\[
Q_{\text{max}} = f_{vs} \pi dl \left( \frac{\tanh \omega}{\omega} \right)
\]

or simply use experimental data.

In eq.(3), \( l \) is the most important variable defined as a distance from the most outer compression side to the neutral axis, and is expressed in eq.(4) by solving equilibrium equation between resultant compression force \( C \) and resultant tensile force \( T \) as shown in Fig. 6.
\[
\lambda = \frac{1}{b E_w} \left\{ -n K_s l + \sqrt{(n K_s l)^2 + 2 b E_w (n K_s l) g} \right\}
\]  

(4)

where \( K_s \) is defined as „slip modulus“ between dowel and glulam and could be obtained also theoretically on the basis of Volkersen-type stress analysis [KOIZUMI ET AL., 1998A, B; SASAKI ET AL.]

\[
K_s = \Gamma \pi d l \left| \frac{\tan h \omega}{\omega} \right|
\]

Rotational rigidity \( R_J (= M/q) \) of half part of the end-joint is expressed in eq.(5) by assuming rigid-body rotation due to slip \( S \) of dowel as well as the equilibrium of moment \( M \) and residual forces \( T, C \) as illustrated in Fig. 7.

\[
R_J = \left| g - \frac{\lambda}{3} \right| (g - \lambda) n K_s
\]  

(5)

Fig.7: Rotation at the half part of end-joint.
Thus, a total mid-span deflection $d_0$ of the end-jointed glulam beam subjected, for example, to four points bending loading as shown in Fig. 8 was derived as eq.(6) by applying virtual work theory.

$$
\delta_0 = \delta_{\text{BENDING}} + \delta_{\text{SHEAR}} + \delta_{\text{JOINTS}}
$$

$$
= \frac{P(3I_sL^2 - 4I_s^3)}{48EI} + \frac{\kappa Pl_s}{2GA} + \frac{Pl_sL}{4R_j}
$$

(6)

Fig. 8: *Four points bending test set-up.*

### 3.2 Experimental Results

All experiments have been completed by a research group at Institute of Wood Technology, Akita Prefectural College of Agriculture by cooperating with Wood Research Institute, Kyoto University. In this article, a part of the test results is outlined. The rest of them is now being analysed and prepared for presenting to the Journal of Japan Wood Research Society.

Fig. 9 shows a cross section of glulam beam with 6 hardwood dowels embedded in a row at most outer tensile side and also shows the test set-up.
Fig. 9: Cross section of tested beam and photo of test set-up.

Figure 10 shows comparisons between observed load (P) - deflection ($\delta_0$) relationship and calculated ones using eqs.(3)-(6).
Table 3 supplies data used for calculation of maximum bending load and deflection of glulam beam, as well as some calculated results.

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Table 3a, 3b: Data used for calculation (a) and calculation results (b).

4. CONCLUSIONS

In this report, as a recent research topic in the field of Japanese timber engineering, a glued-in hardwood dowels joint was introduced.

It is clear from this research mentioned above that the tensile strength of glued-in dowel joints is controlled by two parameters, one of which is the glue line shear strength f vs and another is shear stiffness Γ which was defined as a proportional coefficient between glue line shear stress τ and relative displacement between dowel and wood member.

From pull-out test and push-out test, glue line shear strength was estimated to be in the range of 7.6 to 9.4 MPa for polyurethane adhesive and 10.9 to 12.9 MPa for epoxy adhesive in the case of Japanese maple dowel and Japanese cedar main member. On the other hand, shear stiffness varied from 9.3 to 43.6 N/mm³ for polyurethane adhesive and 45.2 to 73 N/mm³ for epoxy resin adhesive.

On the basis of these experimental data, flexural properties of glulam beams which were end-jointed by glued-in hardwood dowels were analysed. For the test specimens jointed by only one row of dowels, good agreement was
obtained between theoretical prediction and experimental observation. For multiple rows of dowels, a new analysis is being executed at present and will be presented in near future.

A possibility of using hardwood dowels as jointing device of glulam beams could be shown by a series of investigation. It is, of course, impossible to apply this jointing method for any kind of timber joints, especially for the part where high stress is being sustained. This jointing method, however, will be usable for relatively small scale timber structures in which good appearance is especially demanded.

5. ACKNOWLEDGEMENTS

The author would like to express his sincere thanks to Dr. Simon Aicher and Mr. Gerhard Dill-Langer, who gave him kind supports, discussions and encouragements during his one month stay in the department of wood and timber engineering, Otto-Graf-Institute. Mr. Michael Wolf and Mrs. Lilian Höflin would also be greatly acknowledged for their various helps to the author.

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TEMPERATURE EFFECTS ON THE STRUCTURAL BEHAVIOR OF LAMINATED SAFETY GLASS

ZUM EINFLUSS DER TEMPERATUR AUF DAS TRAGVERHALTEN VON VERBUNDSICHERHEITSGLAS

A L'INFLUENCE DE LA TEMPERATURE AU COMPORTEMENT PORTANT DU VERRE FEUILLETE DE SECURITE

Gunter Krüger

SUMMARY

Shear and bending tests were performed in the temperature range from -20°C to +40°C to investigate the characteristic viscoelastic properties of polyvinyl butyral-resin (PVB), mostly used in laminated safety glass for architectural applications. A temperature dependent coupling parameter is used to describe the structural behavior of laminated safety glass subjected to bending stresses in comparison to a monolithic plate of the same thickness.

ZUSAMMENFASSUNG


RESUME

Des essais en cisaillement et en flexion ont été effectués à la température de -20°C à +40°C afin de déterminer des valeurs caractéristiques visco-élastiques de feuille faite en polyvinyl-butyral (PVB), appliquée en verre feuilleté de sécurité. A l'aide d'un paramètre de couplage dépendant de la température, on
peut décrire le comportement portant de verre feuilleté de sécurité en flexion en comparaison avec une vitre monolithique de même épaisseur.

KEYWORDS: Laminated Safety Glass, PVB, Temperature, Shear Stress, Creep

1. INTRODUCTION

The use of laminated safety glass has considerably enlarged the bounds of possibility for construction with glass in architectural applications, in particular for safety and overhead glazings. Laminated safety glass consists of a sandwich with two (or more) sheets of glass bonded to a thin layer of a marcomolecular polymer, usually polyvinyl butyral (PVB). In comparison to monolithic glass, it has an improved impact resistance and provides substantial safety in accidents. In the case of failure laminated safety glass preserves a residual load-carrying capacity, the tough interlayer has the ability hold glass fragments in place.

Fig. 1: Schematic representation of laminated safety glass subjected to bending stresses.

The structural behavior of laminated safety glass subjected to bending stresses is considerably determined by the shear transfer in the interlayer. Fig. 1 exemplary shows the deformation of a sandwich beam with rigid faces (glass plates) and a thin, low modulus core (PVB). Flexural moments are mainly carried by the glass plates. The stiffness of the core determines the amount of shear transferred between the glass plates.
Since PVB shows a distinctive temperature dependence of its mechanical properties, the consideration of a favourite shear transfer in structural design calculations is not permitted in Germany [DIBt, 1998]. At a temperature \( T = 23^\circ \text{C} \) the PVB resin for architectural applications requires a tensile strength of \( \geq 20 \) N/mm\(^2\) and an elongation at break of \( \geq 250\% \).

At room temperature PVB behaves viscoelastic. Relaxation and a dependence on the duration of loading is observed. Under long-term loading PVB has the tendency to creep. With falling temperature the shear modulus increases. Below the glass transition temperature the material becomes elastic with low damping and high tensile strength at break.

The experimental investigations reported in this paper, provide the temperature dependence of the material data of PVB and the structural behavior of laminated safety glass required as input data for calculations and for practical engineering purposes.

2. TESTS

2.1 Creep Tests in Simple Shear

The test setup is schematically shown in fig. 2. The test specimen with a cross-sectional area of 12mm x 20mm was cut from a laminated safety glass with the construction 4 mm glass / 0.76 mm PVB / 4 mm glass. For a time period of 1000 s a load \( F = 50 \) N produced a constant shear stress \( \tau = 0.2 \) N/mm\(^2\) in the interlayer. The displacement \( u(t) \) between the glass plates was measured with a LVDT during the load was applied and during further 1000 s of free relaxation after the load was released. The time-dependent shear \( \gamma(t) \) was calculated with the nominal thickness \( t_f = 0.76 \) mm of the PVB resin.

The decrease of the shear stiffness during the load $F$ is applied (time interval $0 \leq t \leq 1000$ s) can be described by

$$\gamma(t) = \tau \cdot f(t) \quad (1)$$

with the 4-parameter creep function

$$f(t) = a + b \cdot t + c \cdot [1 - \exp(-\lambda \cdot t)] \quad (2)$$

The parameter $a$ represents the time independent elastic part of the shear that follows instantaneously the external load. The shear modulus resulting from this pure elastic deformation is given by $G = \tau a$.

The linear term describes irreversible creep with a creep velocity $v = b/\tau$. The relaxation time $t = 1/\lambda$ gives the time span within the reversible part of the
shear deformation recovers. A dependence on the load level is not concerned by equ. 1.

![Graph showing shear deformation over time for different temperatures.](image)

**Fig. 3:** Measured shear $\gamma(t)$ as a function of time.

Fig. 3 demonstrates the quite different behavior of PVB at temperatures $T = \pm 0^\circ C$ and $T = +40^\circ C$. With increasing temperature the shear stiffness decreases rapidly and creep becomes more significant. The parameters $a$, $b$ and $\lambda$ were evaluated by a fit of equ. 1 to the experimental data in the interval $0 \leq t \leq 1000$ s. Although the tests are not suitable for a precise determination of the shear modulus, the principal tendency as a function of temperature can be evaluated.

The shear modulus decreases about two orders of magnitude in the considered temperature range (fig.4), while the creep velocity $v(T)$ increases (fig. 5) from $v = 0$ to the about of $10^{-3}$ mm$^3$/Ns. The relaxation time $t = 1/\lambda$ was about 50 s to 200 s.
Fig. 4: Shear modulus $G(T)$

Fig. 5: Creep velocity $v(T)$

2.2 Bending Test
Fig. 6 shows the test setup for 3-point bending. A concentrated load $F=20$ N was applied at the midspan if a beam (span $L=400$ mm, width $B=30$ mm) consisting of laminated glass (4 mm glass / 0.76 mm PVB / 4 mm glass). The center deflection $u(t)$ was measured by a LVDT. The temporal procedure was the same as described in cap. 2.1.

The center deflection can be calculated by

$$u = \frac{F}{48 \cdot (E \cdot I)_{beam}} \cdot L^3$$  \hspace{1cm} (3)

E and I are the Young’s modulus and the moment of inertia of the beam. The moments of inertia of the glass plates $I_g$, of the core $I_f$ and the coupling term $I_c$ contribute to the total amount of the moment inertia I. The cross section area of the beam is shown in fig.6.

$$(E \cdot I)_{beam} = E_g \cdot I_g + E_f \cdot I_f + E_c \cdot I_c \approx E_g \cdot I_g + E_c \cdot I_c$$  \hspace{1cm} (4)

$$I_g = 2 \cdot B \cdot t_g^3/12, \quad I_f = B \cdot t_f^3/12, \quad I_c = \kappa \cdot B \cdot t_g \cdot (t_g + t_f)$$  \hspace{1cm} (5)

Since $I_f \ll I_g$, the moment of inertia of the core can be neglected. The amount of coupling by the core is given by the parameter $\kappa$ with $0 \leq \kappa \leq 1$. In the
limiting case $\kappa = 1$ the total moment of inertia becomes equal that of a monolithic beam with the thickness $t = 2 \cdot t_g + t_f$. $\kappa = 0$ describes the case of zero coupling with $I = I_g$.

Fig. 7: Cross section

The measured displacements $u(t)$ at temperatures $T = \pm 0^\circ C, +10^\circ C, +20^\circ C$ and $+40^\circ C$ are given in fig. 8. At low temperatures PVB and from that laminated glass behaves mostly elastic. Due to the decrease of the shear stiffness of the PVB interlayer with rising temperature the displacement $u(t)$ increases.

At temperatures above $40^\circ C$ the coupling between the glass plates is very small and the deformation of the laminated glass beam is determined mainly by the elastic deformation of the glass plates. It is obviously that creep becomes mostly significant at mid temperatures about $+10^\circ C$ to $20^\circ C$. Due to this, the bending stiffness of laminated glass is mostly influenced by the duration of loading in this temperature range.
Fig. 8: Measured center deflection \( u(t) \) as a function of time for different temperatures between 0 °C and 40 °C
The fig. 8 gives the parameter $\kappa(T)$ that was calculated by means of the equ's. 4 and 6 and the measured displacement $u(t = 10s)$ and $u(t = 1000s)$. Not the nominal, but the actual values of $t_g$ and $t_f$ were used. The actual Young's modulus $E_g$ was determined from a comparative test with a monolithic glass plate.

Fig. 9: Coupling parameter $\kappa(T)$ for a load duration $t = 10$ s and $t = 1000$ s
CONCLUSIONS

A 4-parameter viscolestic model was used to explain the structural behavior of PVB under long-term loading in simple shear. The shear modulus of PVB evaluated from the experimental data decreases about two orders of magnitude in the temperature range between -20°C and +40°C. The creep velocity increases with increasing temperature to about $10^{-3} \text{mm}^3/\text{Ns}$ at +40°C.

The structural behavior of laminated glass subjected to bending stresses is considerably determined by the shear transfer in the PVB interlayer. According to the temperature dependence of the shear modulus of PVB, laminated safety glass behaves as a monolithic plate at temperatures below ±0°C, at high temperatures above +40°C the behavior comes close to the limiting case with no coupling between the glass plates. Load-duration effects are mostly important at temperatures about 10°C to 20°C.
THE PERFORMANCE OF THERMALLY SPRAYED ZINC COATINGS AS ANODE FOR CATHODIC PROTECTION ON REINFORCED CONCRETE

WIRKUNG VON THERMISCH GESPRITZTEN ZINKÜBERZÜGEN ALS ANODE FÜR DEN KATHODISCHEN KORROSIONSSCHUTZ IM STAHLBETONBAU

L´EFFICACITE DE LA METALLISATION AU ZINC SUR LE BETON ARME POUR LA PROTECTION CATHODIQUE ANTI CORROSIVE

Manuela Zecho, Klaus Menzel, Ulf Nürnberger

SUMMARY

Studies regarding the effectiveness of thermally sprayed zinc as galvanic and/or as impressed current anode showed good results under favourable conditions such as high zinc/steel area ratio and sufficient direct moistening. Cathodic protection with sprayed zinc on chloride containing concrete is achieved at least 2 years. With unfavourable zinc/steel area ratio and with temporarily inadequate moistening after one year a marked decline in the protective effect (galvanic anode) was observed. Subsequent operation with external current required steadily increasing voltage due to the increase of the internal resistance of the zinc/concrete interface.

ZUSAMMENFASSUNG

Untersuchungen zur Wirkung von auf Beton thermisch gespritzten Zinküberzügen zum Zwecke des kathodischen Korrosionsschutzes haben gezeigt, daß unter günstigen Bedingungen wie hohes Flächenverhältnis Zink/Stahl und genügende Befeuchtung ein ausreichender Schutz möglich ist. Bei einem ungünstigen Flächenverhältnis Zink/Stahl und mit zumindest zeitweise ungenügender Befeuchtung war nach einem Jahr ein deutliches Nachlassen der Schutzwirkung zu beobachten. Der anschließende Betrieb mit Fremdstrom
erforderte wegen des ansteigenden Widerstandes an der Phasengrenze Zink/Beton zunehmend höhere Spannungen.

**RESUME**

Des études sur l'efficacité de la métallisation au zinc sur le béton pour la protection cathodique anti corrosive ont montré, qu'une protection suffisante est possible dans de bonnes conditions telles qu'une proportion importante de surface zinc/acier et d'humidification satisfaisante. Quand on avait une proportion défavorable de surface zinc/acier et une humidité temporairement insuffisante, on observait, au bout d'un an, une réduction de l'efficacité de protection. L'opération ultérieure avec le courant externe nécessitait une tension croissante due à l'augmentation de la résistance interne de l'interface de zinc/acier.

**KEYWORDS:** cathodic protection, thermally sprayed zinc, reinforced concrete, galvanic anode, impressed current anode, zinc

1. **CATHODIC PROTECTION OF REINFORCED STEEL IN CONCRETE STRUCTURES**

Corrosion of steel in concrete can be stopped or prevented by cathodic protection (CP). In civil engineering CP has been used for over 20 years [BAECKMANN, SCHWENK, 1989; POLDER, 1998]. The advantage of cathodic corrosion protection as compared with other protective measures consists in the fact, that concrete must be removed only in areas of advanced destruction. Chloride-contaminated zones do not need to be removed.

1.1 **Protective Criteria for Cathodic Corrosion Protection on Reinforced Concrete**

**Protective Potential**

As protective potential in DIN 30676 [DIN 30676, 1985] a potential of $U_{\text{NHE}} = -0.43$ V ist stated. The protective potential is limited downwards in order to avoid a loss of adhesion between reinforcement and concrete due to hydrogen
evolution at lower potential values. However it is still uncertain whether commercial concrete types are actually affected [MENZEL, 1989]. In any case the possible danger of hydrogen embrittlement has to be considered in the case of prestressed structures. At a pH > 12 hydrogen evolution starts at about -0.7 V/NHE. Therefore, the protection potential should not exceed -0.65 V/NHE [BAECKMANN, SCHWENK, 1989; SHAW, 1965; PEDEFERRI, BERTOLINI, 1995]

**Protective Current Density**

The protective current density is limited to 20 mA/m² (steel surface). Usual current densities are between 1 and 15 mA/m² (steel surface).

**100 mV-Criterion**

A usual method for checking the effectiveness of cathodic protection is the 100 mV criterion. If, within four hours after the current has been switched off, the potential rises by at least 100 mV, sufficient protection is assumed [NACE Standard RP 0290-90]. However, studies show that in wet concrete and/or after some years of operation the 100 mV rise takes more than 4 hours [MIETZ, ISECKE, 1993] without loss of protection.

**1.2 Anodes for Cathodic Corrosion Protection**

In connection with reinforced concrete in most cases so far cathodic protection units with external current have been used. The anode is usually a conductive net (activated titanium wire mesh or conductive plastic cables with a copper core [BAECKMANN, SCHWENK, 1989]), embedded in a surface layer of shotcrete. In order to counter the problem of inhomogenous current distribution, repeated attempts have been made to apply conductive coatings [APOSTOLOS, 1983; WARNE, 1986; APOSTOLOS, CARELLO, 1985; CARELLO, 1986; MANNING, SCHELL, 1986; SCHELL, 1987; SEMINAR ON CORROSION IN CONCRETE, 1987; APOSTOLOS, 1987; MANNING, SCHELL, 1987; MANNING, 1990].

**1.3 Zinc as an Anode**
A possibility, already frequently used in practice for obtaining conductive concrete surfaces, consists in the application of sprayed zinc. As the zinc/concrete interface changes in the course of time zinc tends to passivate [HILDEBRAND, SCHWENK, 1986]. On the other hand, in concrete containing chloride, activation is possible so that, even without external current, cathodic protection will be successful [BAECKMANN, SCHWENK, 1989].

In the USA, field tests have already been carried out on bridges to test the effect of sprayed zinc films on concrete as a sacrificial anode (e.g. Florida Keys and Tampa Bay). The electric contact between zinc and reinforcing steel was partly assured by direct spraying onto exposed steel. It has been reported that the protective systems are still successfully working after 5 years in operation and that protective current densities of 1 µA/cm² are regularly achieved. The 100 mV criterion is still fulfilled also, however decreasing with time. As a limiting factor for adequate protection insufficient moistening is reported in some of the cases. In the vicinity of the sea splash water and frequent nebulosity ensures moistening for many years of protection. The effectiveness is directly related to the electric resistance of the concrete. In areas of high resistivity (very dry concrete) the protective effect is lower than in areas with a lower concrete resistance. [SAGÜÉS, POWERS, 1995; FUNAHASHI, DAILY, YOUNG, 1997].

Coatings of sprayed zinc have a number of advantages as compared with individual anodes or mesh anodes [SCHELL, 1987; APOSTOLOS, PARKS, CARELLO, 1987; MANNING, SCHELL, 1987]:

- They make possible a very good current distribution adapted to the particular conditions.
- They can be easily applied to surfaces with a complex design, in any direction.
- They only insignificantly change the appearance of the concrete.
- The film thickness can be adjusted to the particular conditions. After consumption of the zinc it can be renewed.
Mentioned as a disadvantage is the fact that, for example in the case of inadequately low concrete cover in the case of protection with external current, short circuits are possible between zinc and reinforcing steel which lead to increased anode consumption and to an acid attack on concrete [Warne, 1986; Manning, Schell, 1986; Seminar on Corrosion in Concrete, 1987; Apostolos, Parks, Carello, 1987; Manning, Schell, 1987]. However, if these circumstances are considered, sprayed coatings (applied by flame spraying and electric arc spraying) can be successfully used, as proved on pillars and bridge decks in marine environment [Carello, R.A., 1986; Apostolos, Parks, Carello, 1987]. In particular, the good long-term behaviour of the anode material is emphasised [Manning, Schell, 1987].

As an anode, zinc is consumed. However, via the film thickness the protection duration as a function of the corrosive medium can be adjusted. On the basis of the studies for concrete structures exposed to chloride, film thicknesses of 200 µm are recommended [Carello, R.A., 1986].

In the case of protection with external current anodes, in [Carello, R.A., 1986] a protective current density of about max. 25 mA/cm² steel surface (during wet periods) and of min. 2 mA/cm² (in dry periods) is recommended by the same author. As the resistance of the zinc/concrete-interface increases with time due to growing layers of corrosion products, the voltage must be increased (adjusted) after a few years.

Initial problems of adhesion of zinc on concrete were improved by changing the process parameters of spraying (spraying distance, angles) and the preparation of the concrete surface. Thus tests and engineering applications already available showed that sprayed zinc films adhere very well to dry, blasted surfaces. Adhesion on "old" concrete proved to be best. Further studies showed that an increased surface temperature of the concrete (60-150 °C) during spraying markedly increases the adhesion of zinc. [Apostolos, 1983; Manning, 1990; Baldock, Brousseau, Arnot, Evraire, 1993; Brousseau, Feldmann,
2. INVESTIGATIONS / TEST METHODS

2.1 Physical Protective Effect

2.1.1 Water Penetration Test

Regarding the physical protection, water penetration tests with zinc-sprayed and bare concrete surfaces were performed. Epoxi-coated concrete cylinders with metallized and bare bases were manufactured and stored in a water filled bath. The test conditions are shown schematically in fig. 1.

Fig. 1: Water penetration test.
2.1.2 Chloride Penetration Test

For testing the effectiveness of zinc as a barrier against chloride penetration metallized and bare concrete surfaces were sprayed with a 3-% sodium chloride solution once a week. The depth of chloride penetration was determined by analyzing drilled samples taken from different depth.

2.2 Cathodic Protection

2.2.1 Specimens

Outdoor-Tests

In outdoor exposure-tests the influence of the following parameters has been taken into consideration:

- coverage ratio (metallized/bare surface)
- zinc-to-steel-surface-ratio,
- concrete cover.

To characterize the effectiveness of sprayed zinc as sacrificial and impressed current anode three concrete slabs of 1m x 1m x 0.1m with three or two layers of reinforcement were manufactured. The slabs were either totally or partially coated with zinc. Details are given in figure 2.

The specimens were made of concrete class B25, containing 3 Wt.-% of chloride per cement weight. The chloride was added as sodium chloride. The water cement ratio amounts to 0.75.

The above figured concrete slabs were used for outdoor tests in Stuttgart. Concrete slab 1 was used as reference slab for measuring the free corrosion potential of zinc and steel. Zinc and reinforcement have never been shortcircuited in this case. The effectiveness of zinc as galvanic anode and as impressed current anode was tested on slab 2 and 3.
Fig. 2: Concrete specimens for outdoor tests.

**Laboratory Tests**

Small specimens (10cm x 7cm x 6cm) were manufactured for tests under constant environmental conditions (20°C, ca. 88% RH). One batch of specimens was chloride-containing (3% Chloride/cement weight), the other specimens were artificially carbonated after concreting and free of chloride. Afterwards on each of the samples two opposite surfaces were thermally sprayed with zinc (zinc-to-steel...
area ratio: 10:1). These specimens were used for testing the effectiveness of zinc as galvanic and impressed current anode.

2.2.2 Zinc Metallizing

Prior to metallizing the concrete surface was sand blasted. The blasting provides the surface roughness necessary to develop adequate bond strength between zinc and concrete. Prior to metallizing, the concrete surface was heated to 60 to 70°C by means of a propan gas burner. The zinc (flamesprayed) was sprayed to a thickness of about 450 - 550 µm.

2.2.3 Electrochemical Tests

Potentials were measured using a calomel electrode (+242 mV to NHE), contacted to the concrete either by means of a wet sponge or (for long term measurements) with silica gel inserted in a drilled hole. Short-circuit current was measured by means of a zero-resistance-ammeter.

3. RESULTS

3.1 Physical protective effect

3.1.1 Water penetration test

The diagram in figure 3 shows the results up to a testing period of 160 hours. Obviously the zinc-coating obstructs the water penetration to some extent.
3.1.2 Chloride penetration test

The results of the chloride analysis vs. concrete cover after 246 days are shown in figure 4. In case of metallized concrete the chloride content is significantly lower than in case of the bare surface.
3.2 Cathodic Protection

3.2.1 Outdoor Tests - Zinc as Galvanic Anode

*Free Corrosion Potential of Zinc and Iron (no short-circuit)*

The potential run of zinc not contacted to steel shows a primary decrease in potential up to -780 mV SCE, followed by a potential rise to values close to the free corrosion potential of the reinforcement (-400 mV SCE) after about 600 days (fig. 5).

![Graph showing corrosion potential of zinc and reinforcement (no short-circuit).](image)

Fig. 5: *Free corrosion potential of zinc and reinforcement (no short circuit).*

*Corrosion Potential of Fe-Zn-Short-Circuit-Couples*

![Graph showing potential of the reinforcement after short-circuiting with zinc.](image)

Fig. 6: *Potential of the reinforcement after short-circuiting with zinc.*
Figure 6 shows the potential of the reinforcement prior to and after short circuiting with zinc. Short-circuiting leads to a potential drop of the reinforcement of about 200 mV to values near the free corrosion potential of zinc. With time the potential rises to values about -400 mV SCE.

**Polarization Decay Measurements**

Polarization decay measurements show good results if evaluated by the 100 mV criterion for a period of about 9 months. The potential shift exceeds 100 mV within a few hours after opening the short-circuit between zinc and steel (fig. 7).

![Polarization decay measurement](image)

Fig. 7: *Polarization decay measurement 220 days after short-circuiting.*

From winter 1995 on the 100 mV-criterion was not fulfilled any more (fig. 8).

![Polarization decay measurement](image)

Fig. 8: *Polarization decay measurement 636 days after short-circuiting.*

**Galvanic Current Densities**
Current densities also show a time dependency. With increasing time the current density decreases (fig. 9). At the beginning the current density is about 2.6 µA/cm² (steel surface). 2.5 years later values about only 0.18 µA/cm² (steel surface) were reached.

Fig. 9: Galvanic current densities.

### 3.2.2 Outdoor Tests - Zinc as Impressed Current Anode

After decreasing of the effectiveness of zinc working as galvanic anode tests with zinc as impressed current anode were started. By means of a potentiostat the potential of the reinforcement was regulated to -750 mV SCE and -800 mV SCE respectively. The required protective current was measured (fig. 10).

30 days after operating with external current the impressed current shows values of about 2 and 4 µA/cm² (steel surface). 10 weeks later the potential of the reinforcement could not be regulated to -800 mV SCE anymore. Obviously operation with external current required inadequately high voltage due to the increase of the internal resistance of the zinc/concrete interface.
3.2.3 Laboratory Tests - Zinc as Galvanic Anode

Chloride Containing Concrete

Two years after short-circuiting the potential shows values about -750 mV SCE in chloride containing concrete, indicating sufficient protection (fig. 11). This can be confirmed by polarization decay measurements (fig. 12).
Fig. 11: *Potential run of the reinforcement short-circuited with zinc in chloride containing concrete.*

![Graph showing potential run of reinforcement short-circuited with zinc in chloride containing concrete.](#)

Fig. 12: *Polarization decay measurement 342 days after short circuiting (chloride containing concrete).*

**Carbonated Concrete**

The potential of the short-circuited couple (zinc-reinforcement) in carbonated concrete shows no constant run (fig. 13). The 100 mV criterion is barely fulfilled (fig. 14).

![Graph showing potential run of reinforcement short-circuited with zinc in carbonated concrete.](#)

Fig. 13: *Potential run of the reinforcement short-circuited with zinc in carbonated concrete.*

![Graph showing potential run of reinforcement short-circuited with zinc in carbonated concrete.](#)
3.2.4 Laboratory Tests - Zinc as Impressed Current Anode

**Chloride Containing Concrete**

Under constant environmental conditions cathodic protection with zinc as impressed current anode was assured for 1.5 years at least. Figure 15 shows the required impressed current for regulating the potential of the reinforcement to -800 mV SCE.

![Graph showing impressed current for chloride containing concrete](image)

**Carbonated Concrete**
For regulating the potential to -800 mV SCE an impressed current of 0.2 to 0.7 µA/cm² is required in carbonated concrete (fig. 16).

![Graph showing current density over time for carbonated concrete samples](image)

Fig. 16: Impressed current (carbonated concrete, two identical samples).

4. DISCUSSION

The exposure and laboratory-tests reveal both limitations and possibilities of CP with zinc sprayed anodes.

By outdoor exposure in rural/town atmosphere full protection (according to potential- or 100mV-decay criteria) is restricted to a period of about one year as far as the parameters chosen in this experiment (zinc-to-steel ratio 1:0.4 to 1:0.6; 3% chloride) are realistic. The loss of effectiveness is due to zinc passivation (in case of galvanic anode) and increase of the ohmic resistance of the zinc-concrete interface (in case of impressed current anode). Delamination or significant loss of adhesion of the zinc cover was not observed during the three years of exposure.

The results of the laboratory experiments (high and constant humidity, high zinc-to-steel area ratio and chloride contaminated concrete) confirm the good results reported from marine environment. Until now the electrochemical parameters are almost stable and in the range of full protection.
In carbonated concrete, CP does not perform satisfactory because of zinc passivation and the high internal resistance of the circuit.

5. ACKNOWLEDGEMENTS

The Project, No. 9590, was funded by the German Federal Ministry of Trade and Commerce through AiF Arbeitsgemeinschaft industrieller Forschungsvereinigungen und supervised by the Forschungsgemeinschaft Zink e.V. The exposure tests will be continued.

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HEAT FLOW IN A GLULAM JOIST WITH A GLUED-IN STEEL ROD SUBJECTED TO VARIABLE AMBIENT TEMPERATURE

WÄRMEFLUß IN EINEM BRETTSCHICHTHOLZTRÄGER MIT EINER EINGEKLEBTEN STAHLSTANGE BEI VERÄNDERLICHERN TEMPERATUREINWIRKUNGEN

FLUX DE TEMPERATURE DANS UNE POUTRE DU LAMELLE COLLE AVEC UNE GOUJON COLLE SOUMIS AUX TEMPERATURES AMBIENTES VARIABLES

Simon Aicher, Michael Wolf, Gerhard Dill-Langer

SUMMARY

It is reported on some experimental and numerical investigations concerning temperature distributions in a glulam joist with a protruding axially glued-in threaded steel rod subjected to variable ambient temperatures. The temperature evolution in the two specimens investigated was monitored by thermo-elements attached to the rod along anchorage length and in the wood. In detail, the thermo-elements were placed in the interior of the rod along an axial groove. The groove had been planed into two rods of half circle cross-section which were glued together subsequently. The temperature load over a period of six hours consisted of three intervals each lasting two hours of warming up, steady state and cooling with temperatures between 20 and 50°C. The measurements revealed significant temperature gradients along anchorage length being more pronounced for the rod with the higher slenderness ratio \( \lambda = 20 \). The transient numerical simulation was performed in an approximation with an axial-symmetric finite element model. The effective thermal diffusivity of a specifically introduced steel thread/adhesive layer showed the highest sensitivity for fitting the numerical results to the empirical temperatures. Within the frame of the chosen approach a very good agreement of experimental and theoretical data was obtained.
ZUSAMMENFASSUNG


RESUME

Il est rapporté des premières investigations experimentelles et numériques aux distributions de température dans du lamellé-collé avec une tige filetée resortant de la section transversale soumis a une température ambiente variable. L’évolution du température dans les deux épreuves étudiés était enregistré par des thermocouples qui étaient montés le long de l’ancrage de tige filetée et dans le bois. En détail, les thermocouples ont été appliqués dans une entaille axiale à l’entérieure du goujon métallique. L’entaille intérieure a été fait par fraisage de deux gougons métalliques demi-circulaires; après les deux parts étaient collés. Le chargement thermique dans une temps de six heures comprenait trois phases de deux heures de l’échauffement, de la température stationaire et de refroidissement avec de températures entre 20 et 50°C. Les mesures montraient des gradients du température le long de l’ancrage qui était plus fort en cas d’élancement $\lambda=20$. La simulation transitoire numérique était conduite en approximation avec une
modélisation par éléments finis axisymétriques. La conductibilité des éléments d’une couche spéciale combinant l’adhésive et le métal a montré la plus grande sensibilité à l’ajustement des résultats numériques et expérimentaux. Dans le cadre de l’approche choisie on a obtenu une très bonne accord entre les résultats expérimentaux et théorétiques.

KEYWORDS: glued-in steel rods, metric thread, glulam, transient heat transfer, variable temperature loads, thermo-elements

1. INTRODUCTION

Glued-in rods in wood resp. glulam represent a very promising connecting method in timber engineering and have already been employed in several large scale constructions outside Germany. Especially in joints where the steel rods are not entirely hidden in the timber but are screwed or welded to connecting devices of steel (i.a. [RIBERHOLT, 1986]; [BUCHANAN AND TOWNSEND, 1990]; [FAIRWEATHER, 1992]; [AICHER ET. AL. 1998]) which are in contact to the ambient air the impact of varying temperatures is of interest. The extremely different thermal conductivities resp. diffusivities of wood and steel obviously lead to time dependent temperature gradients which result in eigenstrains resp. -stresses. Finally the eigenstress state is influenced by a 3 to 5 times higher temperatur elongation coefficient of steel as compared to wood.

A limited number of investigations have been addressed to the stated problem so far; an extensive experimental test program has been carried out by [EHLBECK ET AL., 1992]. In the cited investigations the length of the stepped temperature cycles was two days with a temperature difference of 50°C at different relative humidities of the air. Thermo-elements and strain gauges were mounted in a side groove along the rod axis then filled by an epoxy resin necessitating special attention to delayed temperature hardening of the adhesive at tests with elevated temperatures. Conclusions bound to the investigated temperature cycles state a rather uniform temperature distribution along
anchorage length within several hours after the temperature change and hence constant strains.

Theoretical investigations on the transient temperature evolution in the regarded hybrid material joint are not stated in literature. The recent FMPA investigation on the temperature issue described here had essentially two aims:

- to investigate the experimental implications and results with threaded test rods having an interior groove where all measuring devices are applied. This method has been first adopted by [AMSTUTZ, 1955] in investigations on the bond behaviour of deformed steel bars in concrete,
- to make a first attempt of a numerical simulation of the problem.

This paper covers exclusively the temperature evolution aspect; the associated eigenstrain - stress issue will be forwarded separately.

2. EXPERIMENTAL INVESTIGATIONS

2.1 Specimen built-up

The investigations so far were performed with two glulam specimens No. 1 and 2 with a square cross-section of 160 x 160 mm, incorporating a threaded steel rod glued-in parallel to the fiber direction at one end cross-section. Figure 1 depicts the general built-up of the specimens. The difference of both similar specimens mainly concerned the rod diameter d and the anchorage length l_a, thus the rod slenderness ratio \( \lambda_a = l_a / d \). For specimens No. 1 and 2 quantities d, l_a, \( \lambda_a \) were 200, 20, 10 and 320, 16 and 20, respectively. Table 1 contains a compilation of all relevant dimensions. Figures 1b, c reveal the loading conditions of the specimens when mounted into the loading rig. In order to measure the heat flow, the specimens were equipped with thermo-elements mounted to the threaded rods along anchorage length and in the wood. The employed thermo-
elements consisted of copper constantan wires, fixed to the rod and wood with epoxy adhesive.

Fig. 1 a-c: *General built-up of investigated specimens No. 1 and 2*

In order to avoid the cited problems in experimental data interpretation experienced by [EHLBECK ET. AL. 1992], it was decided to manufacture test rods which contain all measuring devices - thermo-elements, strain gauges - in an
interior axial groove. Figure 2 gives a schematic view of the built-up of the rods and of the application of the thermo-elements.

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>unit</th>
<th>specimen No 1</th>
<th>specimen No 2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>glulam</strong></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>cross-section a x b</td>
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<td>160 x 160</td>
<td>160 x 160</td>
</tr>
<tr>
<td>total length l</td>
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<td>1185</td>
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<td>anchorage length l_a</td>
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<td>320</td>
</tr>
<tr>
<td>intermediate length l_m</td>
<td>mm</td>
<td>400</td>
<td>640</td>
</tr>
<tr>
<td>slot length l_s</td>
<td>mm</td>
<td>225</td>
<td>225</td>
</tr>
<tr>
<td>hole diameter dh</td>
<td>mm</td>
<td>21</td>
<td>17</td>
</tr>
<tr>
<td><strong>steel rod, metric thread</strong></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>total length l_a + l_u</td>
<td>mm</td>
<td>200 + 50</td>
<td>320 + 50</td>
</tr>
<tr>
<td>nom. diameter d=d_nom</td>
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<td>20</td>
<td>16</td>
</tr>
<tr>
<td>core diameter d_c</td>
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<td>13,3</td>
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<tr>
<td>slenderness l_a /d_nom</td>
<td>-</td>
<td>10</td>
<td>20</td>
</tr>
<tr>
<td>groove cross-section agr x lgr</td>
<td>mm</td>
<td>6 x 6</td>
<td>4,8 x 4,8</td>
</tr>
<tr>
<td><strong>thermo-elements</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>along glued-in rod l_1</td>
<td>mm</td>
<td>30</td>
<td>42</td>
</tr>
<tr>
<td>l_2</td>
<td>mm</td>
<td>46</td>
<td>75</td>
</tr>
<tr>
<td>l_3</td>
<td>mm</td>
<td>46</td>
<td>75</td>
</tr>
<tr>
<td>l_4</td>
<td>mm</td>
<td>46</td>
<td>75</td>
</tr>
<tr>
<td>l_e</td>
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<tr>
<td>in wood</td>
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<tr>
<td><strong>substitute dimensions for</strong></td>
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</tr>
<tr>
<td><strong>axial-symmetric analysis</strong></td>
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</tr>
<tr>
<td><strong>glulam</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( (a \cdot b = \pi \frac{d_w^2}{4}) )</td>
<td>mm</td>
<td>180,1</td>
<td>180,1</td>
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<tr>
<td><strong>rod-groove</strong></td>
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<td>( (a_{gr} \cdot b_{gr} = \pi \frac{d_{gr}^2}{4}) )</td>
<td>mm</td>
<td>6,77</td>
<td>5,41</td>
</tr>
</tbody>
</table>

Table 1: Dimensions of investigated specimens No. 1 and 2, locations of thermo-elements and substitute dimension for approximate axial-symmetric finite element analysis
The manufacturing of the rods was conceived to obtain surfaces of the threaded rods resembling best possible the periphery of an unprocessed rod. This was achieved for each test rod by planing the cross-sections of two rods to a half circle. Then the rectangular grooves were planed along the axis of both rod halves. After application of the thermo-elements the halves were glued together by means of an epoxy adhesive of low viscosity. The curing of the rod bond took place in clamped conditions at a slightly elevated temperature of 40°C. Both ends of the rods were sealed with air tight isolation material.

When manufacturing a half circle shaped rod by planing away one half of the cross-section of the original rod, the half circle shaped rod tends to bend due to freed eigenstresses which are introduced into the periphery of the rod during cold tensioning. As the eigenstresses and hence the bow increase significantly with higher steel qualities, threaded rods of rather low steel quality of 4.6 were used.

Before bonding the rods into the specimen the functioning of the measuring devices was tested extensively. The adhesive used to bond in the rods was a 2component epoxy (Ciba AW 139 / HV 953 U).
Apart from the thermo-elements applied to the interior of the rod a thermo-element was positioned in the wood in the middle of the cross-section at 20 mm distance from the bonded-in end of the steel rod. The moisture content of the glulam specimens at the time of gluing-in the threaded rods was about 10%.

Figure 3 shows a photograph of one of the specimens with the thermo-wires and the wires of the strain gauges sticking out of the protruding end of the glued-in rod.

Fig. 3: Photograph of the end face of specimen No. 1 with the protruding rod end and sticking out thermo- and strain gauge wires

2.2 Applied temperature load at quasi constant moisture conditions

Climate recordings under roof at FMPA showed that temperatures in summer time may easily vary by about $\Delta T = 20$ to $30^\circ C$ during one day. Hereby temperatures increase during noon from about $20 - 25^\circ C$ to $40 - 50^\circ C$; the elevated temperatures last for about 1.5 to 3 hours and then gradually decrease. In
Heat flow in a glulam joist with a glued-in steel rod subjected to variable ambient temperature

In more Southern European areas the daily temperature differences supposedly may well be 10 to 15°C higher.

The tests conducted so far consisted of a simple temperature cycle, lasting for six hours (Fig. 4). The cycle consists of three stages: a first degressive increase of the ambient temperature from 20 to 50°C in 2 hours, then a 2 hours period of constant temperature of 50 °C and third a degressive cooling phase to 20°C within 2 hours.

![Course of applied temperature and relative humidity of ambient air](image)

Fig. 4: Course of applied temperature and relative humidity of ambient air

Due to the stepwise temperature changes in combination with the crude control mechanism of the climate chamber the temperature course in the warming up phase however was rather linear (see Fig. 10). In order to keep the moisture content of the wood quasi constant during temperature variations the relative humidity of the ambient air of the chamber was adjusted at every temperature change accordingly to maintain a moisture content of the wood of 10%. For the tests the specimens were mounted in vertical position to the test rigs installed in the climate chamber; in the pure temperature test the self weight of the specimens was compensated. All side and end grain faces of the specimens were fully exposed to the ambient air.
2.3 Test results

Figures 5a,b reveal the measured temperature evolutions of both specimens at the locations of the thermo-elements along anchorage length during the heating, steady state and cooling phase.

Fig. 5 a-b: Measured temperatures for specimens No. 1 and No. 2

a) No. 1 \((l_a/d=10, a/d=9)\)
b) No. 2 \((l_a/d=20, a/d=11.25)\)
Both figures show distinct differences of the temperature along the anchorage length of the rod (thermo-elements 1 - 4) and between ambient temperature and the temperature of the wood in the center of the cross-section close to the rod end. It can be seen further that the temperature gradients decrease as anticipated with sustained constant elevated temperature. The temperature gradient along the rod is more pronounced for specimens No. 2 with the higher slenderness ratio of $\lambda = 20$. The absolute differences of the temperatures between the protruding and the embedded end of the rods at the end of the heating period are considerably smaller for specimen No. 1 with $\lambda = 10$ compared to specimen No. 2 with $\lambda = 20$. In the cooling phase the temperature differences are, as anticipated, contrary to the warming up phase. The influence of the slenderness ratio on the rod temperature distribution is revealed in Fig. 6.

![Graph showing temperature evolution over time for different slenderness ratios](image)

*Fig. 6: Influence of slenderness ratio on the temperature evolution at both ends of the anchorage length*
3. HEAT FLOW ANALYSIS

3.1 Basic equations

The Fourier equation of heat conduction in a cylindrically anisotropic material (rotation axis z here coinciding with the grain direction) is

\[
D_{rr} \left[ \frac{1}{r} \frac{\partial T}{\partial r} + \frac{\partial^2 T}{\partial r^2} \right] + D_{\phi\phi} \frac{1}{r^2} \frac{\partial^2 T}{\partial \varphi^2} + D_{zz} \frac{\partial^2 T}{\partial z^2} = \frac{\partial T(r, \varphi, z, t)}{\partial t}
\]  

(1)

where

\[ T \text{ and } t \] temperature and time,

\[ D_{ii} = \frac{k_i}{\rho C_p} \left( \begin{array}{c} \text{mm}^2 \\ \text{m} \\ h \end{array} \right) \] thermal diffusivities,

\[ k_i \left( \begin{array}{c} \text{W} \\ \text{m} \text{K} \end{array} \right) \] thermal conductivities,

\[ C_p \left( \begin{array}{c} \text{Ws} \\ \text{kg} \text{K} \end{array} \right) \] specific heat,

\[ \rho \left( \begin{array}{c} \text{kg} \\ \text{m}^3 \end{array} \right) \] mass density.

The boundary condition for the investigated heat conduction problem was assumed to be of the convective type (Newton’s law of cooling), so

\[
\frac{\partial T}{\partial n} = - \frac{h_n}{k_n} (T_s - T_B)
\]  

(2)

where

\[ h_n \left( \begin{array}{c} \text{W} \\ \text{m}^2 \text{K} \end{array} \right) \] convection heat transfer coefficient in direction n of the outward normal of the boundary

\[ T_s, T_B \] temperature of the surface resp. of the ambient air.
Symbol $\partial / \partial n$ denotes differentiation in the direction of the outward normal of the boundary surface (here $n = z, r$). For heat transfer coefficients $h_n \rightarrow \infty$ the boundary condition (2) tends to the situation where the temperature distribution is prescribed at the boundary surface, say is for instance equal to the ambient temperature.

### 3.2 Modelling details, material properties

The numerical simulation of the temperature evolution in the experimentally investigated specimens was performed by means of finite element analysis (FEA). In an approximation the actual 3D geometry was replaced by an axial symmetric model. Figures 8a,b give the geometry; the dimensional quantities are listed in Table 1. Figure 7 shows the finite element discretization. The model was built by 8 node axial symmetric thermal elements with cylindrical anisotropic thermal conduction capability. The employed thermal element can be replaced by an equivalent structural element performed in a second step of the analysis not discussed here.

It can be seen from Fig. 8 that the simulation model of the actual three component compound – wood/adhesive/steel - contained a fourth "material" being the thread/adhesive layer primarily used for modelling of the contact conductance of the steel/adhesive interface (see below).

Fig. 7: Finite element mesh of the employed axial-symmetric model
Fig. 8 a-b: Geometry of employed axial-symmetric finite-element model  
a) length section  b) cross-section
The material properties employed in the thermal analysis are compiled in Table 2. With respect to the thermal material properties, it has to be reminded that the primary aim of the transient temperature modelling was to obtain computational temperature fields which agree best possible with empirical pointwise temperature measurements. The computational temperature fields are applied as loads for the mechanical problem, i.e. for the numerical assessment of the temperature induced eigenstrains and eigenstresses to be compared with test results. For this reason the absolute values of the employed material properties are of secondary interest. Nevertheless mainly literature based properties (thermal conductivities, specific heats) were used. However heat transfer coefficients and the thermal diffusivity of the thread/adhesive layer (see below) were regarded as parameters adjustable in a certain range for the fitting of the analysis to the empirical data.

The specific heat of wood depends pronouncedly on the moisture content \( u \); the quantity given in Table 2 for the specific heat of glulam is based on [Kollmann, 1982]

\[
C_p(u) = \frac{u + 0.324}{1 + u} \cdot 4.19 \quad \text{kJ/kg K}
\]  

(3)

using a moisture content of \( u = 0.12 \).

3.3 **Issue of contact conductance**

At the interface of two materials with different thermal conductivities in general a sudden jump of the temperature profile is encountered unless perfect thermal contact exists and hence continuity of temperature and heat flux. The temperature discontinuity in the interface is due to micro voids filled with air. Similarly to the convection coefficients of the outer boundary of the continuum, contact conductances \( h_c \) [W/m\(^2\)K] should be prescribed for the interior interfaces of the regarded problem, here especially for the interface steel/adhesive. However the employed FE code ANSYS does not enable the input of contact
conductances. The problem was solved in such manner that the contact conductance of the steel/adhesive interface was incorporated into the effective thermal diffusivity of the thread/adhesive layer.

Table 2: Compilation of thermal analysis properties of all materials of the investigated specimen built-up.

<table>
<thead>
<tr>
<th></th>
<th>units</th>
<th>wood parallel to grain</th>
<th>steel</th>
<th>adhesive</th>
<th>thread/adhesive layer</th>
</tr>
</thead>
<tbody>
<tr>
<td>mass density</td>
<td>ρ</td>
<td>450</td>
<td>7850</td>
<td>1400</td>
<td>4625</td>
</tr>
<tr>
<td>thermal conductivity</td>
<td>k</td>
<td>0,29 1)</td>
<td>0,13 1)</td>
<td>60 1)</td>
<td>0,28 2)</td>
</tr>
<tr>
<td>specific heat</td>
<td>C_p</td>
<td>1,66 3)</td>
<td>0,4</td>
<td>1,5 4)</td>
<td>1</td>
</tr>
<tr>
<td>thermal diffusivity</td>
<td>D</td>
<td>1400</td>
<td>620</td>
<td>68800</td>
<td>480</td>
</tr>
<tr>
<td>(convection) heat transfer coefficient 5)</td>
<td>h</td>
<td>50</td>
<td>25</td>
<td>100</td>
<td>50</td>
</tr>
</tbody>
</table>

1) acc. to DIN 4108, part 4  
2) acc. to [N.N., 1990]  
3) acc. to eq. (3) with u = 0,12  
4) acc. to [Batzer, 1985]  
5) literature data for forced convection of gas media vary roughly between 10 to 100 W/(m² K) [GRÖBER, 1963]; a value of 25 W/(m² K) is assumed for convection at exterior walls in DIN 4108, part 4
Compared to heat transfer coefficients the thermal diffusivity of the thread/adhesive layer revealed to be the most sensitive parameter for adjustment of the theoretical and empirical temperature distribution. A diffusivity value of $D = 31.1 \, \text{mm}^2/\text{h}$ gave the best agreement with empirical temperature measurements.

### 3.4 Results of heat flow analysis

![Temperature distribution of specimen No. 1 at different times of the applied temperature history](image-url)

Fig. 9: Temperature distribution of specimen No. 1 at different times of the applied temperature history
Figures 9a - g show the spatial evolution of the temperature field in the wood and along the steel rod for different times of the warming up, steady state and cooling phase. The figures reveal a temperature exchange in the wood occurring faster than anticipated.

4. COMPARISON OF EXPERIMENTAL AND MODELLING RESULTS

Figures 10 and 11 depict the high agreement between the experimentally obtained temperature evolution and the numerical analysis. The empirical observation of pronounced temperature gradients along rod length was confirmed by the numerical calculations. Furthermore the results of FE-analysis also revealed the quantitative differences between the transient temperature distributions of the two specimens with different slenderness ratios. With respect to material parameters used in the calculations it shall be reminded that the primary objective of the numerical modelling was the fitting of the measured data.

Fig. 10: Comparison of experimental- and FEA-results in the case of specimen No. 1
5. CONCLUSIONS

Empirical tests in conjunction with FE-analysis were conducted to investigate the heat conduction problem of a glulam joist with a protruding steel-rod bonded in glulam. The results of experimental data and numerical calculations proved that the transient temperature distributions are more complex than anticipated including pronounced temperature gradients along rod length. Thus more simple ideas of the temperature distributions (steel rod always in equilibrium with the surrounding air; wood as perfect isolating material) turned out to be too crude. So, temperature variations in the time domain of hours cause complex uneven temperature distributions in the regarded compound and hence eigenstrains and stresses. The latter aspect will be dealt with separately.

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INFLUENCE OF SPECIMEN GEOMETRY ON STRESS DISTRIBUTIONS IN PULL-OUT TESTS OF GLUED-IN STEEL RODS IN WOOD

EINFLUß DER PRÜFKÖRPERGEOMETRIE AUF SPANNUNGSVERTEILUNGEN BEI AUSZUGSVERSUCHEN VON IN HOLZ EINGE-KLEBTEN GEWINDESTANGEN

INFLUENCE DE GEOMETRIE DE L’EPROUVETTE SUR LA DISTRIBUTION DES CONTRAINTES AUX ESSAIS DE TRACTION DES GOUJONS METALLIQUES COLLES EN BOIS

Simon Aicher, Lilian Höfflin, Michael Wolf

SUMMARY

It is reported on some numerical investigations concerning the influence of specimen geometry on the stress distribution in a joist like glulam specimen with two steel rods glued-in axially parallel to fiber at the two opposite end faces. Hereby the question arises to what extent the intermediate distance between the two opposite glued-in rod ends affects the stress distribution in the specimen. The concern is on the tension stresses in the wood and especially on the stresses in the contact area between steel rod and wood along anchorage length.

The parameter study was conducted in an approximation with an axial symmetric finite element model. The computation showed that intermediate distances of more then two times of anchorage length have no influence on the regarded stress distributions in the nearfield of the rod. Distances between the two opposite rods smaller then the anchorage length result in increasing changes of the stress distributions.
ZUSAMMENFASSUNG

Es wird über rechnerische Untersuchungen zum Einfluß der Prüfkörpergeometrie auf die Spannungsverteilung in stabförmigen Prüfkörpern aus Brettschichtholz, in die an den gegenüberliegenden Querschnittsenden Gewindestangen axial faserparallel eingeklebt sind, berichtet. Es erhebt sich hierbei die Frage, in welchem Maße der Abstand zwischen den beiden gegenüberliegenden eingeklebten Stahlstangenenden die Spannungsverläufe im Prüfkörper beeinflußt. Von Interesse sind die Zugspannungen im Holz und insbesondere die Spannungen im Kontaktbereich von Gewindestange und Holz längs der Verankerungslänge.


RESUME

Une étude numérique est présenté sur l’influence de la géométrie aux distribution des contraintes dans une éprouvette parallélépipède de lamellé-collé avec deux tiges filetés collés parallèlement au fil de bois à chaque extrémité de l’éprouvette. Le problème posé est à quelle distance l’influence des goujons collés devient négligeable sur la distribution de contraint de l’éprouvette. L’étude concerne les contraintes de traction dans le bois et spécifiquement les contraintes dans la surface du contact entre le goujon métallique et le bois le long de l’ancrage.

L’étude paramétrique a été simplifiée par une modélisation par éléments finis axisymétriques. Les résultats ont montré si la distance séparant l’extrémité des goujons est supérieure a deux fois la longueur de l’ancrage elle n’a plus d’influence sur les contraintes au voisinage du goujon métallique. Si ce distance est inférieure a une fois la longueur de l’ancrage la résultat est une augmentation du changement.
KEYWORDS: Wood, glulam, glued-in steel rod, specimen geometry, stress distribution, axialsymmetric FE-analysis

1. INTRODUCTION

In the framework of the European Research Project „Glued in rods in timber structures“ one of several work items consists in the derivation of an empirical data base on the withdrawal resistance of axially loaded glued-in threaded steel rods in pull-out tests [JOHANSSON ET AL., 1998]. For assessment of the influences of anchorage length $l_a$, rod diameter $d$ and adhesive type (Phenolic Resorcinol, Polyurethane, Epoxy), a number of 270 short term and 130 DOL tests are to be performed with rods glued-in parallel to fiber direction centrally into glulam specimens of square cross-section. This test configuration is regarded subsequently. It should also be mentioned that additionally to the above test configuration further 85 pull-out tests are performed with different angles between load and grain direction.

After evaluation of several possible solutions for the lay-out of the specimens (i.a. [AICHER AND HERR, 1997]) it was decided to use a test set-up as shown in Fig. 1 with two rods, one being the actual test rod ($l_a = \text{anchorage length}$, $d = \text{nominal diameter}$), the other one being the support rod.

The embedment length $l_s$ and diameter $d_s$ of the support rod were chosen as $l_s = 1,2 \times l_a$ and $d_s = 1,5 \times d$ in order to definitely achieve a failure at the test rod where the load displacement measuring devices are mounted. One of several reasons for choosing a test configuration with unequal rods was to create a highest possible degree of conformity in test layout of ramp load and related DOL tests. In case of the DOL tests, due to limited number of available LVDT’s, only one rod per specimen can be equipped with deformation measuring devices.

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1 The authors gratefully acknowledge the funding of the research project „Glued in rods in timber structures“ by European Commission through grant No. SMT4-CT97-2189.
The dimensions of the specimens, covering a very large range of rod slenderness ratios $\lambda_a = \ell_a / d = 5$ to 40, had to be chosen as a sound compromise between material resp. manufacturing effort and avoidance of falsifying boundary conditions. With respect to the latter the specimen ratios $D/d$ and $L_m/\ell_a$ were of primary interest ($L_m = l_m/2$). The ratio of cross-sectional width $D$ to rod diameter $d$ was chosen in the range of $7 - 7.5 - 8.75$, what at a first view could seem rather small ($7$ resp. $7.5$) with respect to splitting failures. However, testing of more than 50% of the specimens so far revealed only very rare cases of splitting failure modes. This issue shall not be deepened here. In the following the implications of the length between test and support rod $l_m = 2L_m$ resp. the ratio $L_m/\ell_a$ is discussed in more detail.
2. FINITE ELEMENT ANALYSIS OF SPECIMEN GEOMETRY

2.1 Modelling aspects

The effect of the mentioned geometry ratios was investigated by means of a linear elastic finite element analysis. In an approximation of the real geometry the 3D problem was analysed as an axial-symmetric problem with the rotation axis coinciding with the rod axis. In this approach the specimen diameter $D^* = (4D^2/\pi)^{0.5}$ was determined from the equivalent square area. The material properties of the glulam were assumed to be homogeneous in the cross-section, i.e. radial and tangential stiffness quantities were smeared. For the parameter study the following stiffness values were employed for glulam ($l = \text{longitudinal} = \text{parallel to fiber}, r = \text{radial}, t = \text{tangential}$):

\[
\begin{align*}
E_{ll} &= 12000, & E_{rr} &= E_{\varphi\varphi} = 600, \\
G_{rl} &= G_{l\varphi} = 680, & G_{r\varphi} &= 45 \\
\nu_{rl} &= 0.45, & \nu_{l\varphi} &= 0.056, & \nu_{r\varphi} &= 0.31.
\end{align*}
\]

All given $E_{ii}$, $E_{ij}$ values are in N/mm$^2$. In case of $\nu_{ij}$ first and second index denote deformation and stress direction, respectively. The glue line was not explicitly modelled as it was verified that this approximation is of minor influence. In the parameter study on the influence of intermediate length $l_m$ the dimensions of the support rod were assumed equal to the test rod, so only one quarter of the length section of the approximate axial symmetric specimen was modelled (Fig. 2). Figure 3 shows the employed finite element mesh.

With respect to the conditions at the embedded end of the rod $\xi = l_a$, the following has to be considered. Depending on the type of adhesive there can be a well bonded connection between both areas, but for other adhesives the bond can be very poor, if existent at all. In the lower loading range this bond, if existent, will contribute to load transfer, however for strength considerations a tension cut-off zone should be assumed. In the presented calculations an element layer of 1 mm thickness representing the adhesive was arranged between the rod end face...
and the end grain face of the bottom of the drilled hole; the MOE of the isotropic adhesive layer was set to 2000 N/mm$^2$.

![Diagram of specimen with sections](image1)

**Fig. 2:** Dimensions of finite element discretized part of the specimen and locations of sections for stress evaluation

![Finite Element discretization of the axial symmetric model](image2)

**Fig. 3:** Finite Element discretization of the axial symmetric model
2.2 Simulation results

The specimen geometry was analysed in general for six different intermediate length ratios \(L_m/l_a = 0.05; 0.125; 0.25; 0.5; 1\) and 2 at constant ratios \(D/d\) and \(l_a/d\). Two slenderness ratios \(l_a/d = 10\) and 20 and two ratios \(D/d = 5\) and 7.5 were investigated. All results presented here strictly apply to \(D/d = 7.5\) and \(L_a/d = 10\); they apply qualitatively very well to the other parameter configurations, too.

2.2.1 Axial normal stresses in the wood

Figure 4 depicts the cross-sectional distribution of the normal stress in the wood in axial direction of the specimen in four sections 2-5 with increasing relative distance (see Fig. 2) from the embedded end of the glued-in rod; the stresses are given for intermediate length ratios of \(L_m/l_a = 0.25; 0.5; 1\) and 2.

Generally speaking the graphs reveal the following: In case of a large distance between the two rods a very even stress distribution occurs in the middle part of the intermediate length \(l_m\). With gradual approach of the regarded cross-section to the embedded end of the rod the normal stress distribution becomes increasingly uneven due to the high stiffness of the rod attracting the stress flow. In section 1, right at the rod end, a high stress peak occurs close to the rod. In case of small intermediate distances between the rods the normal stress distribution in the wood is never even; the stress flow is throughout highly concentrated in the interior of the cross-section.
Fig. 4: Axial normal stress in the wood in some cross-sections (see Fig. 2) of intermediate length $l_m$ for different ratios $l_m/l_a$

a) section 2  

b) section 3  

c) section 4  

d) section 5
Influence of specimen geometry on stress distributions in pull-out tests of glued-in steel rods in wood

Fig. 5 a-c: Axial normal stresses in the wood in three cross-sections close to the embedded end of the rod
a) at end of rod  b) 20 mm from the rod  c) 40 mm from the rod
All mentioned aspects were anticipated qualitatively. With respect to very short distances before the embedded end of the rod it is necessary to look at sections with discrete absolute distances to the rod end. Figures 5a-c show the normal stress distribution for different $L_m/l_a$ ratios in three sections with equal absolute distances of 1, 20 and 40 mm to the embedded rod end. It is interesting to note that the stress peak in front of the rod end remains quasi constant in all three sections for intermediate length $l_m \geq 2l_a$. Contrary, for values $l_m \leq l_a$ the stress concentration increases significantly, and these differences remain also in the cross-section of the rod end. So, it can be stated that for intermediate length $\leq 2$, quantity $l_m$ has a non neglectible influence on the normal stress distribution in the wood at the location of the embedded rod end. More important with respect to falsifying boundary conditions is the question whether intermediate distance $l_m$ has a significant impact on the bond stresses along anchorage length.

2.2.2 Bond stresses along anchorage length

Figures 6a,b and 7a,b depict the shear and axial stresses resp. the stresses in radial and tangential direction in the steel-wood-interface along anchorage length for a large parameter span $l_m/l_a = 0,1$ to 4. The following conclusions can be drawn from the given stress distributions. The shape of the stress distribution is only marginally affected by intermediate length ratio. Intermediate length has a certain influence on the peak values of the stresses at both ends of the anchorage length. Shorter intermediate lengths result in higher peak values what conforms to above presented axial normal stress distribution in the wood. The interface shear stresses in the wood, as anticipated, show the highest sensitivity to a reduction of intermediate length. Significant changes in the shear stress distribution however are confined to ratios of $l_m/l_a < 1$. 
Fig. 6 a-b: Stresses of wood in the steel-wood-interface along bond line depending on intermediate timber length \( l_m \) for \( D/d = 7.5 \) and \( l_a/d = 10 \)

a) shear stress  

b) axial stress
Fig. 7 a-b: Stresses of wood in the steel-wood-interface along bond line depending on intermediate timber length $l_m$ for $D/d = 7.5$ and $l_a/d = 10$

a) radial stress

b) tangential stress
3. CONCLUSIONS

The parameter study on the effect of intermediate length of two steel rods glued parallel to fiber into the end grain faces of a glulam joist was conducted in an approximation with an axial symmetric finite element model. The computation showed that intermediate distances of more than two times of anchorage length have no influence on the regarded stress distributions in the nearfield of the rod. Distances between the two opposite rods smaller than the anchorage length result in increasing changes of the stress distributions.

ACKNOWLEDGEMENTS

The presented problem was subject of several intensive discussions with the project partners (Swedish National Testing and Research Institute, University of Lund, TRADA Technology Ltd., University of Karlsruhe and associated industry partners). This co-operation is gratefully acknowledged.

REFERENCES


CORROSION INDUCED FAILURES IN PRESTRESSED CONCRETE STRUCTURES AND PREVENTATIVE MEASURES

KORROSIONSBEDINGTE SCHÄDEN IN SPANNBETONKONSTRUKTIONEN UND HIERAUS ABGELEITETE PRÄVENTIVMASSNAHMEN

DOMMAGES EN CONSEQUENCE DE CORROSION DANS DES CONSTRUCTIONS EN BETON PRECONTRAINT ET DES MESURES PREVENTIVES

Ulf Nürnberger

SUMMARY

During the past 40 years in Germany some serious damages in post- and pre-tensioned components have been occurred because of stress corrosion cracking of prestressing steel. Especially those problems caused a great stir, where the attendant circumstances of design, execution and building materials were not unusual. But the prestressing steel was unsuited to resist the inevitable conditions on construction site and inside the construction.

Therefore some important measures and corrections were introduced to reduce number of damages related to stress corrosion cracking. They include improved standards and recommendations for planning and executions of new constructions and for strengthening of older ones. Research was done to develop non-destructive measuring techniques to investigate older prestressed constructions. With regard to building materials such materials which not guarantee durable protection of the embedded steel had been forbidden and a steady control prevents that unwelcome materials can come on the market. As well as hot rolled bars, the quenched and tempered wires and the cold deformed wires and strands had been improved in the last 40 years.
ZUSAMMENFASSUNG


RESUME

Pendant les derniers 40 ans, il y avait en Allemagne quelques dommages assez graves dans des éléments de construction en béton précontraint avec adhésion immédiate ou ultérieure (post-tendu), qui pouvaient être ramenés à une corrosion fissurante. L'attention attireraient en particulier ces cas, où les conditions concernant la planification, la réalisation et les matériaux de construction n'étaient pas extraordinaires. Toutefois, l'acier ne pouvait pas résister aux influences inévitables du chantier en général et dans la construction en particulier.

Pour diminuer les dommages à cause de la corrosion, quelques mesures et innovations essentielles étaient introduites. Celles-ci comprennent des normes et des recommandations perfectionnées pour la planification et la réalisation de nouvelles constructions et la mise en état des constructions plus âgées. On a exécuté des recherches pour développer des méthodes de mesure non-destructives, qui permettent l'examen des constructions précontraintes plus âgées. Les matériaux de construction, qui n'étaient pas capables de protéger
l'acier à une longue vue, sont interdits; des contrôles permanents empêchent la distribution des matériaux inconvenables. En ce qui concerne l'armature de pré-contrainte, il faut constater, que pas seulement les fils et les torons cylindrés à chaud ou écouissés, mais aussi les fils et torons traités par trempe et revenu, étaient améliorés continuellement pendant les derniers 40 ans.

KEYWORDS: prestressed concrete, prestressing steel, stress corrosion cracking, failures, durability, corrosion protection

1. INTRODUCTION

In prestressed concrete the purpose of prestressing lays in exerting pressure on the low tensile strength concrete in that areas, where the concrete normally is exposed to tensile stresses and threatened by cracking and failure. Therefore in prestressed concrete structures the high strength prestressing steel performs essential bearing action.

In posttensioned concrete members high strength steel wires, strands or bars are arranged in ducts. After casting and hardening of concrete the prestressing reinforcement is tensioned and compressive stresses are generated in concrete. After that ducts are grouted with cement mortar, in order to protect the steel against corrosion and to guarantee a permanent load capacity.

During the past 40 years in Germany some serious damages in post-tensioned and pre-tensioned components have been occurred because of stress corrosion cracking of the prestressing steel [1-3]. These failures happened due to onsite conditions and were favoured of the sensitiveness of the used prestressing steel.

Stress corrosion cracking and the subsequent failure of steel and construction may occur

• if the protection is not guaranteed from the beginning as a result of poor workmanship,
• or it is lost because of deterioration of the construction in the course of the time,
• or the prestressing reinforcement is predamaged during handling.

Also an application of unsuitable materials for prestressing steel, injection mortar or concrete can alone or in combination with other factors favour SCC.

2. THE BASIS OF STRESS CORROSION CRACKING

Fractures of prestressing steel as a rule can be referred to hydrogen induced stress corrosion cracking (H-SCC) [4,5]: It may happen during erection of the construction or during the later use. The following conditions are necessary for H-SCC:

• a sensitive material or state,
• a sufficient tension load,
• at least a slight corrosion attack.

During the corrosion process hydrogen atoms have to be set free and get absorbed by the steel. In sensitive steels the hydrogen under the effect of mechanical stresses can create precracks in critical structural areas such as grain boundaries. These cracks may grow and result in material fracture.

Special conditions have to exist to activate the formation of adsorbable hydrogen atoms. To understand the correlations between procedure on site and development of damage, the chemical reactions of corrosion should be considered (table 1).
Table 1: *Chemical reactions of corrosion*

<table>
<thead>
<tr>
<th>Reaction Type</th>
<th>Chemical Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anodic iron dissolution</td>
<td>$\text{Fe} \rightarrow \text{Fe}^{2+} + 2e^-$</td>
</tr>
<tr>
<td>Cathodic reactions</td>
<td>$\frac{1}{2} \text{O}_2 + \text{H}_2\text{O} + 2e^- \rightarrow 2\text{OH}^-$</td>
</tr>
<tr>
<td>If pH &lt; 7</td>
<td>$\text{H}^+ + e^- \rightarrow \text{H}_{ad}$ (hydrogen discharge)</td>
</tr>
<tr>
<td>Rivalry reaction with regard to 5</td>
<td>$2\text{H}_{ad} \rightarrow \text{H}_2$ (recombination)</td>
</tr>
<tr>
<td>Is prevented in the presence of promotors</td>
<td></td>
</tr>
<tr>
<td>If oxygen is present or air access</td>
<td>$2\text{H}_{ad} + \frac{1}{2} \text{O}_2 \rightarrow \text{H}_2\text{O}$</td>
</tr>
</tbody>
</table>

Harmful hydrogen can arise only

- if the steel surface is in the active state or depassivated (this is expressed by reaction 1),
- if the cathodic reaction of corrosion is discharging hydrogen (this is described by reaction 3),
- if the adsorbable atomic hydrogen is not changed into the molecular state (see reaction 4).

Therefore at the surface of corroding steel amount of adsorbable hydrogen atoms rises

- with increasing hydrogen concentration (reaction 3 is accelerated),
- in the presence of so-called promotors (reaction 4 is hindered).
From the practical point of view one can say that hydrogen induced damages are only possible

- in acid mediums
- or in the presence of promotors such as sulphides, thiocyanates and compounds of arsenic or selenium.

In concrete structures the attacking medium is mostly alkaline and acid solutions are limited to exceptions. Nevertheless, in natural environments the pitting induced H-SCC can take place (Fig 1). Pitting induced H-SCC means crack initiation within a corrosion pit. In the corrosion pits the pH-value falls down because of hydrolysis of the Fe$^{3+}$-ions. Pittings or spots of local corrosions can be explained by differential aeration or concentration cells. Especially effective is the attack of condensation water or salt enriched watery solutions when erecting the constructions. In prestressed construction carbonation of concrete and mortar as well as chloride contamination are responsible of local corrosion attack.

![Pitting induced stress corrosion cracking](image)

Fig. 1: *Pitting induced stress corrosion cracking*
3. PRESTRESSING STEEL

In case of sensitive prestressing steel already minimal contents of hydrogen can lead to irreversible damages. Therefore the steel quality and the susceptibility to hydrogen of the applied steel melt is of enormous importance [6,7]. In Table 2 one can find a survey of the different steel types. World wide dominates the application of cold deformed wires and strands. They yearly production of cold deformed material is 1 million tons. The strength of commonly used cold deformed prestressing material amounts with falling diameter of the wire from 1570 up to 2060 N/mm². In the case of strands the upper strength limit is higher.

Table 2: Survey about produced prestressing steel

<table>
<thead>
<tr>
<th>type</th>
<th>shape, surface</th>
<th>diameter</th>
<th>anchorage system</th>
<th>strength class</th>
<th>production (world wide) tons/year</th>
</tr>
</thead>
<tbody>
<tr>
<td>cold deformed</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>•wire</td>
<td>round-smooth</td>
<td>4-12,2 mm</td>
<td>wedge or button heads</td>
<td>1570-1860¹ (N/mm²)</td>
<td>1,000,000 (world wide)</td>
</tr>
<tr>
<td></td>
<td>round-profiled</td>
<td>5-5,5 mm</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>•strand</td>
<td>round-smooth</td>
<td>9,3-15,3 mm</td>
<td></td>
<td>1700-2060¹ (N/mm²)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(7 wires)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>hot rolled</td>
<td>round-smooth</td>
<td>26-36 mm</td>
<td>thread (ends)</td>
<td>1030-1230 (N/mm²)</td>
<td>50,000 (Germany, UK)</td>
</tr>
<tr>
<td>•bar</td>
<td>round-ribbed</td>
<td>26,5-36 mm</td>
<td>thread (full length)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>quenched and tempered</td>
<td>round-smooth</td>
<td>6-14 mm</td>
<td>wedge</td>
<td>1570 (N/mm²)</td>
<td>5,000 (Germany, Japan)</td>
</tr>
<tr>
<td>•wire</td>
<td>round-ribbed</td>
<td>5-14 mm</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>oval-ribbed</td>
<td>40-120 mm²</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

¹in Germany max.1770 N/mm²

The larger diameter hot rolled bars with 50,000 tons-a-year production have a considerably lower strength from 1030 to 1230 N/mm². The production of quenched and tempered steel wires is with yearly 5,000 tons significantly lower. Their strength corresponds with those of the larger diameter deformed wires. Hot rolled and quenched and tempered steel are produced only in few countries.
Table 3 summarises the advantages and application of the different steel types. Cold deformed steel may be applied for all types of prestressing and in all types of members. This material is economical to produce and relatively high strength can be reached. The strand has an advantage of easy transport and storage, flexibility, easy installation and good bond behaviour. Nowadays the hot rolled bars and quenched and tempered steel have their special areas of application. Bars, which are tension members with high load-bearing capacity and easy handling, are given the preference if transverse prestressing and earth anchors are required. Ribbed quenched and tempered wire with oval cross section and very good bond behaviour are needed for prefabricated elements and sleeper for the railway.

Table 3: Advantages and application of prestressing steel

<table>
<thead>
<tr>
<th>type</th>
<th>especial advantage</th>
<th>application</th>
</tr>
</thead>
<tbody>
<tr>
<td>cold deformed</td>
<td>• economically to produce</td>
<td>for all types of prestressing</td>
</tr>
<tr>
<td>• wire</td>
<td>• high strength</td>
<td>and all types of elements</td>
</tr>
<tr>
<td></td>
<td>• low coil diameter, high coil weight (strand)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• flexible tension members (strand)</td>
<td></td>
</tr>
<tr>
<td>• strand</td>
<td>• easy to install (strand)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• good bond behaviour (strand)</td>
<td></td>
</tr>
<tr>
<td>hot rolled</td>
<td>• tension members with high load</td>
<td>transverse</td>
</tr>
<tr>
<td>• bar</td>
<td>• simple to anchor</td>
<td>prestressing, earth anchors</td>
</tr>
<tr>
<td></td>
<td>• easy handling</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• effective bond of ribs</td>
<td></td>
</tr>
<tr>
<td>quenched and</td>
<td>• very good bond behaviour</td>
<td>prefabricated</td>
</tr>
<tr>
<td>tempered</td>
<td></td>
<td>elements, sleeper</td>
</tr>
<tr>
<td>• wire</td>
<td></td>
<td>(railway)</td>
</tr>
</tbody>
</table>
4. REASON OF DAMAGES

In connection with special circumstances all types of prestressing steels may suffer SCC. Making a diagnosis of defective building can help to find solutions to avoid future problems. Reasons of damages of prestressed concrete structures can be classified as [3]:

- insufficient design,
- incorrect handling of building materials,
- incorrect execution of planned design,
- unsuitable mineral building materials,
- unsuitable (sensitive) prestressing steel.

The reason 1 to 4 are responsible for lack or time dependent loss of passivation and a promotion of SCC.

Damages as the consequence of these influences should not be the main topic of this contribution. However, especially those problems caused a great stir throughout the world of prestressed concrete, where the attendant circumstances of design, execution and building materials were not unusual. But the prestressing steel was unsuit to resist the inevitable conditions on construction site and inside the construction. During the last 10 years in Germany some serious damages in pre-tensioned and post-tensioned components have been occurred, which were strongly favoured by the sensitivity of the used prestressing steel. In the following two cases will be discussed and we will end up with conclusions respectively consequences.

Some cases steel fractures in the yet ungrouted ducts of post tensioned structures as well as serious collapses of building components can be attributed to the presence of aggressive water in the ducts which results from bleeding [6,8]. Bleeding is a separation of fresh concrete, where the solid content sink down and the displaced water rises or penetrates in inner hollows. In the bleeding water significantly high contents of sulphates and increased quantities of chlorides may
be accumulated (Table 4) by leaching of the construction materials cement, aggregates and water. The high amounts of potassium-sulphate result from the gypsum in the cement. The watery phase of fresh concrete penetrates into the ducts through anchorages, couplings and defects in the sheet and accumulates at the deepest points. Already in the not grouted and not pre-stressed condition the steel may suffer from strong pitting. If the steel is sensitive to hydrogen pitting induced stress corrosion cracking takes place (Fig 2).

Table 4: Analysis of bleeding water

<table>
<thead>
<tr>
<th>Analysis</th>
<th>Range</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>sulphate</td>
<td>1.90 - 5.20</td>
<td>mg/l</td>
</tr>
<tr>
<td>chloride</td>
<td>0.13 - 0.18</td>
<td>mg/l</td>
</tr>
<tr>
<td>calcium</td>
<td>0.06 - 0.09</td>
<td>mg/l</td>
</tr>
<tr>
<td>sodium</td>
<td>0.18 - 0.37</td>
<td>mg/l</td>
</tr>
<tr>
<td>potassium</td>
<td>3.60 - 7.30</td>
<td>mg/l</td>
</tr>
<tr>
<td>pH-value</td>
<td>10 - 13</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 2: Fracture starting at a corrosion pit
In some cases construction elements of buildings failed after more than 25 years as a consequence of SCC [2,3,5,6,8]. For example the beams of the roof of a laboratory had been affected causing the roof collapse (Fig 3). In an other case the beams over a factory building were affected. It was found, that in both cases the used quenched and tempered steel was very sensitive to hydrogen induced cracking. Investigations showed, that in the time of erecting the building the steel had suffered precorrosion and precracking probably owing to contact with normal influences such as condensing water and bleeding water. A conformation of this hypothesis gives this X-ray pattern photograph with many cracks in the steel (Fig 4). And it was concluded from the research, that the pre-cracks, initiated in a very early stage of the building, could grow discontinuously in the grouted state over a very long period of time.

Fig. 3: Broken beam as a result of stress corrosion cracking
Fig. 4: *X-ray pattern photograph of a precracked steel, below: cut of the wire*

Fig. 5 illustrates the typical intercrystalline fracture of the steel wires with crack propagation pattern on the grain faces. Such a mechanism evidently can occur only on the condition, that the prestressing steel has a very high sensitivity to SCC, that means it react to very small amounts of hydrogen.

Fig. 5: *Intercrystalline fracture with crack propagation pattern on the grain faces*
The wires of the bundle were broken in one section. The ducts were grouted completely and fractures have occurred within an alkaline and chloride free mortar.

The problem of very high sensitivity of prestressing steel is not restricted to special steel types. A cold deformed wire was involved in another case. Numerous circularly wrapped and prestressed concrete pressure water pipes in the ground exploded after some years in service (Fig 6). As a consequence an extensive expanse has been flooded. During service life concrete cover had suffered loss of bond and microcracking by sulphate action. Responsible for this effect was the production technology of the pipes, namely the accelerated hydration by heat curing [5]. As a consequence the concrete around the steel, that means the concrete in the contact zone steel / concrete, was carbonated.

Fig. 6: Stress corrosion cracking of a circularly wrapped pipe

In this case the prestressing steel proved to be extremely sensitive to hydrogen influence. The strength of the steel was higher than 2000 N/mm$^2$ and as
Corrosion induced failure in prestressed concrete structures and preventative measures

a result of radial shear stresses firstly longitudinal and later also transverse cracking took place (Fig 7).

Fig. 7: Cracking of the cold deformed wires

5. CONSEQUENCES

Table 5 summarises the most important measures introduced in Germany after some periods with increased number of damages related to SCC.
Table 5: *Reasons of damages of prestressed concrete structures and consequences*

<table>
<thead>
<tr>
<th>reason of damage</th>
<th>consequences</th>
</tr>
</thead>
<tbody>
<tr>
<td>insufficient design</td>
<td>• strengthening of construction</td>
</tr>
<tr>
<td></td>
<td>• improved standards and recommendations for planning and execution of work</td>
</tr>
<tr>
<td></td>
<td>• inspection of other constructions of the same kind</td>
</tr>
<tr>
<td></td>
<td>• demolition of elements or construction</td>
</tr>
<tr>
<td>incorrect execution of planned design and incorrect handling of building materials</td>
<td>• see above</td>
</tr>
<tr>
<td></td>
<td>• training and careful education of personnel</td>
</tr>
<tr>
<td></td>
<td>• supplementary reinforcement to avoid unannounced failure</td>
</tr>
<tr>
<td>unsuitable mineral building materials</td>
<td>• limitation of chlorides in water, cement and aggregates</td>
</tr>
<tr>
<td></td>
<td>• prohibition of cements, additives and accelerators which favour corrosion and hydrogen evolution</td>
</tr>
<tr>
<td>unsuitable (sensitive) prestressing steel</td>
<td>• prohibition of qualities, which suffer SCC under on-site conditions</td>
</tr>
<tr>
<td></td>
<td>• development of steel types which are more resistant to hydrogen</td>
</tr>
<tr>
<td></td>
<td>• limitation of maximum strength</td>
</tr>
<tr>
<td></td>
<td>• long-time SCC-test (2000 h) under practical conditions</td>
</tr>
</tbody>
</table>

Corrections as a result of insufficient design are the concern of design engineer and of constructional experts. They include improved standards and recommendations for planning and execution of new constructions and for strengthening of older ones. Focus of the last years were measures during erection the building related to

- investigation of concrete technologies, which favour bleeding of fresh concrete.
- In this connection it was found, that addition of additives to fresh concrete such as retarder and liquefier may increase bleeding [6].

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• In other cases it was recommended to flush out the salt rich bleeding water after concreting [4].
• A far reaching decision was the limitation of time between prestressing of steel and injection of ducts in German standard DIN 4227.

All measures had the aim to hinder corrosion and cracking before protecting reinforcement with injection mortar.

Much research was done to develop nondestructive measuring techniques to investigate older prestressed constructions. There exist numerous buildings where formerly produced prestressing steel was used. Nowadays we know because of other damages, that some of the older steel types are very sensitive to H-SCC. Therefore a magnetic methods was developed to assess cracks and fractures before a possible catastrophic collapse [9,10]. The prestressing steel is magnetised straight through the concrete cover by a strong magnet. Deep cracks and fractures are to detect by an alteration of the flow of the magnetic lines (Fig 8). To examine the magnetic flow, characteristically changed by cracks and fractures, a very sensitive Hall-analysers is used. It is carried on the concrete surface along the tension members. If a multitude of broken steel wires are detected and the further bearing capacity is not sure a demolition of the construction may be recommended.
Further incorrect execution of the planned design and incorrect handling of the building materials may be found. For instance, the concrete cover is too low and of no adequate quality or the ducts are not injected well. In such case the improper dealing with the prestressed concrete is responsible for problems.

In Germany the application of a supplementary reinforcement is under discussion [3]. In the case of loss of capacity a failure without warning should be avoided. This measure is very controversial discussed because it represents a turning away from the principle of prestressed concrete and induces economic disadvantage for the German construction industry.
With regard to building materials in the past strict instructions led to an essential reduction of failures [11]. Already 30 years ago the content of chlorides in water, cement and aggregates was limited to a very low level. Cements which not guarantee durable protection of the embedded steel had been forbidden, and a steady control prevents, that unwelcome materials can come on the market. That concerns all types of cements and additives, which are continuously investigated electrochemically with regard to corrosion and hydrogen evolution.

But the most radical change took place on the field of prestressing steel. In Germany a great variety of steel types is in use (section 3). New types find their way into the prestressing technique, other types are excluded because they not proved to be reliable. That means they suffered SCC under onsite conditions [1 - 3]. As well as hot rolled bars, the quenched and tempered and the cold deformed steel had been improved in the last 40 years.

Concerning the failures with quenched and tempered steel a so called old type of steel was involved [12]. In the beginning these material was carbon-silicon-manganese alloyed (Table 6). Because of a non sufficient stress corrosion cracking behaviour, already an attack of condensating water resulted in hydrogen induced problems, the chemical composition was changed in 1965 [13]. The carbon content was lowered and chromium was alloyed. Because of this precaution the full quenching and subsequent tempering was improved and the retained martensite and detrimental residual stresses were reduced. Further the manganese content was decreased and silicon content was increased. As a consequence absorption, solubility and diffusivity of hydrogen was considerably diminished [14].
Table 6: Analysis of quenched and tempered prestressing steel

<table>
<thead>
<tr>
<th></th>
<th>old type</th>
<th>new type</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>0.65</td>
<td>0.48</td>
</tr>
<tr>
<td>Si</td>
<td>1.19</td>
<td>1.80</td>
</tr>
<tr>
<td>Mn</td>
<td>0.88</td>
<td>0.62</td>
</tr>
<tr>
<td>P</td>
<td>0.014</td>
<td>0.012</td>
</tr>
<tr>
<td>S</td>
<td>0.020</td>
<td>0.014</td>
</tr>
<tr>
<td>Cr</td>
<td>0.04</td>
<td>0.46</td>
</tr>
</tbody>
</table>

As mentioned at the beginning the old type quenched and tempered steel was involved in numerous and serious damages. But nearly no failures happened after application of the new steel type.

In a similar action the sensitive hot rolled bar with bainitic structure 1985 was replaced by a new type pearlitic steel [5].

Based on experiences with damages and on laboratory testing we know, that the susceptibility to H-SCC increases greatly with increasing strength [5,6]. An interpretation of numerous SCC-tests conducted according to the FIP-standard showed, that with an increase of the strength of cold deformed steel from 1700 to 2000 N/mm² the service life drops by a factor of 100. Therefore the upper strength is limited in Germany to about

- 1400 N/mm² for hot rolled steel,
- 1700 N/mm² for quenched and tempered steel,
- 1950 N/mm² for cold deformed steel

and there is no tendency to release this requirement. In the European standard EN 10138 an upward extension of the strength of cold deformed wire and strand is planned. In the case of strands the strength range is to be extended up to 2060 N/mm² which complies essentially with the wish of prestressing steel makers in France. Essential German objections to this European standard [15] are related to the extension of the strength limit for cold deformed material because a
such high strength steel seems not to be sufficiently safe under practical conditions.

At long last also the development and application of an improved corrosion testing in Germany is a consequence of damages caused by using unsuitable prestressing steel [4,16,17]. In most cases SCC-tests are carried out according to the so called FIP-standard [18] in a highly concentrated thiocyanate solution. The result of the FIP-testing is a brittle fracture after hydrogen charging and general embrittlement of the whole cross section. The newly used test solution is adopted to practical mediums over a testing time up to 2000 hours. The advantage of this standard testing procedure is, that the mechanism of cracking agrees with that observed in the practice. It is a pitting induced SCC where the crack initiation is connected with corrosion processes on the steel surface. In Table 7 the conditions of the FIP- and of the German long time test are compared. Besides a mixture of neutral salts the new solution contains also 0,5 g/l thiocyanate as a promoter in order to provide hydrogen in an amount which can occur in defective concrete constructions. The new test is applied in steel production quality control and also in examining causes of damages. The main observation is all steels or steel melts that caused difficulties on practice failed in standard test within 2000 hours. This observation is valid for cold deformed material, quenched and tempered wire and hot rolled bars.

Table 7: Parameters and criteria of SCC-test of prestressing steels

<table>
<thead>
<tr>
<th>standard</th>
<th>concentration</th>
<th>temperature</th>
<th>stress</th>
<th>lifetime (request)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FIP</td>
<td>20 mass.-% NH₄SCN</td>
<td>50°C</td>
<td>0,8 Rₘ</td>
<td>hot-rolled &gt; 30-50 h, quench., temp. &gt; 10-15 h, cold-deformed &gt; 2-3-h</td>
</tr>
<tr>
<td>DIBT</td>
<td>0,5 g/l Cl⁻, 5 g/l SO₄²⁻, 1 g/l SCN⁻</td>
<td>50°C</td>
<td>0,8 Rₘ</td>
<td>&gt;2000h</td>
</tr>
</tbody>
</table>
In Fig 9 the results of tests with cold deformed wires in comparison with those of the FIP-test are shown. The strength of the steel increases from left to right. In the FIP-test lifetime decreases steadily with increasing strength. In the long time test only steel with a very high strength failed. We may have stress corrosion fractures within 2000 hours, if the strength exceeds about 2000 N/mm². Similar results on quenched and tempered steels are published in [7].

Fig. 9: Results of stress corrosion tests of cold deformed wires after FIP- and (German) DIBT-guidelines

REFERENCES

Corrosion induced failure in prestressed concrete structures and preventative measures


   Siehe auch: Vorträge der DBV-Arbeitstagung "Forschung" am 07.11.1996 in Wiesbaden. Deutscher Beton-Verein e. V.


[12] Technische Mitteilungen Hüttenwerk Krupp - Rheinhausen, 1953


Corrosion induced failure in prestressed concrete structures and preventative measures
Table 1: Chemical reactions of corrosion

<table>
<thead>
<tr>
<th>anodic iron dissolution</th>
<th>1 Fe → Fe $^{2+}$ + 2$e^-$</th>
</tr>
</thead>
<tbody>
<tr>
<td>cathodic reactions</td>
<td>1 $\frac{1}{2}$ $O_2$ + H$_2$O + 2$e^-$ → 2 OH$^-$</td>
</tr>
<tr>
<td>if pH &lt; 7 5:</td>
<td>5 $H^+$ + $e^-$ → $H_{ad}$ (hydrogen discharge)</td>
</tr>
<tr>
<td>rivalry reaction with regard to 5</td>
<td>5 $2H_{ad}$ → H$_2$ (recombination)</td>
</tr>
<tr>
<td>is prevented in the presence of promotors</td>
<td>5 $2H_{ad}$ + $\frac{1}{2}$ $O_2$ → H$_2$O</td>
</tr>
<tr>
<td>if oxygen is present or air access</td>
<td></td>
</tr>
</tbody>
</table>

Table 2: Survey about produced prestressing steel

<table>
<thead>
<tr>
<th>type</th>
<th>shape, surface</th>
<th>diameter</th>
<th>anchorage system</th>
<th>strength class</th>
<th>production (world wide)</th>
</tr>
</thead>
<tbody>
<tr>
<td>cold deformed</td>
<td>round-smooth</td>
<td>4-12,2 mm</td>
<td>wedge or button heads</td>
<td>1570-1860$^{(1)}$ (N/mm$^2$)</td>
<td>1.000.000 (world wide)</td>
</tr>
<tr>
<td>• strand</td>
<td>round-smooth</td>
<td>9,3-15,3 mm</td>
<td>wedge or button heads</td>
<td>1700-2060$^{(1)}$ (N/mm$^2$)</td>
<td></td>
</tr>
<tr>
<td>hot rolled</td>
<td>round-smooth</td>
<td>26-36 mm</td>
<td>thread (ends)</td>
<td>1030-1230 (N/mm$^2$)</td>
<td>50.000 (Germany, UK)</td>
</tr>
<tr>
<td>• bar</td>
<td>round-ribbed</td>
<td>26,5-36 mm</td>
<td>thread (full length)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>quenched and tempered</td>
<td>round-smooth</td>
<td>6-14 mm</td>
<td>wedge</td>
<td>1570 (N/mm$^2$)</td>
<td>5.000 (Germany, Japan)</td>
</tr>
<tr>
<td>• wire</td>
<td>round-ribbed</td>
<td>5-14 mm</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• oval-ribbed</td>
<td>40-120 mm$^2$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$^{(1)}$ in Germany max.1770 N/mm$^2$
Corrosion induced failure in prestressed concrete structures and preventative measures

Table 3: *Advantages and application of prestressing steel*

Table 4: *Analysis of bleeding water*

<table>
<thead>
<tr>
<th>Substance</th>
<th>Concentration (mg/l)</th>
</tr>
</thead>
<tbody>
<tr>
<td>sulphate</td>
<td>1.90 - 5.20</td>
</tr>
<tr>
<td>chloride</td>
<td>0.13 - 0.18</td>
</tr>
<tr>
<td>calcium</td>
<td>0.06 - 0.09</td>
</tr>
<tr>
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<td>0.18 - 0.37</td>
</tr>
<tr>
<td>potassium</td>
<td>3.60 - 7.30</td>
</tr>
<tr>
<td>pH-value</td>
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</tr>
</tbody>
</table>
Table 5: Reasons of damages of prestressed concrete structures and consequences

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<th>new type</th>
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</thead>
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<tr>
<td>carbon</td>
<td>0.65</td>
<td>0.48</td>
</tr>
<tr>
<td>silicon</td>
<td>1.19</td>
<td>1.80</td>
</tr>
<tr>
<td>manganese</td>
<td>0.88</td>
<td>0.62</td>
</tr>
<tr>
<td>phosphorus</td>
<td>0.014</td>
<td>0.012</td>
</tr>
<tr>
<td>sulphur</td>
<td>0.020</td>
<td>0.014</td>
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</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>quench., temp. &gt; 10-15 h</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>cold-deformed &gt; 2-3-h</td>
</tr>
<tr>
<td>DIBT</td>
<td>0.5 g/l Cl⁻, 5 g/l SO₄²⁻, 1 g/l SCN⁻</td>
<td>50°C</td>
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<td>&gt;2000h</td>
</tr>
</tbody>
</table>

Fig. 1: Pitting induced stress corrosion cracking
Fig. 2: Fracture starting at a corrosion pit

Fig. 3: Broken beam as a result of stress corrosion cracking
Corrosion induced failure in prestressed concrete structures and preventative measures

Fig. 4: X-ray pattern photograph of a precracked steel, below: cut of the wire

Fig. 5: Intercrystalline fracture with crack propagation pattern on the grain faces
Fig. 6: Stress corrosion cracking of a circularly wrapped pipe

Fig. 7: Cracking of the cold deformed wires
Fig. 8: Magnetic field of a cracked wire embedded in concrete (Sawade)
Fig. 9: Results of stress corrosion tests of cold deformed wires after FIP- and (German) DIBT-guidelines