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New Methods in the Rehabilitation of Prestressed Concrete Structures

Nouvelles méthodes pour l'assainissement des ouvrages en béton précontraint

Neue Verfahren bei der Instandsetzung von Spannbetonbauwerken

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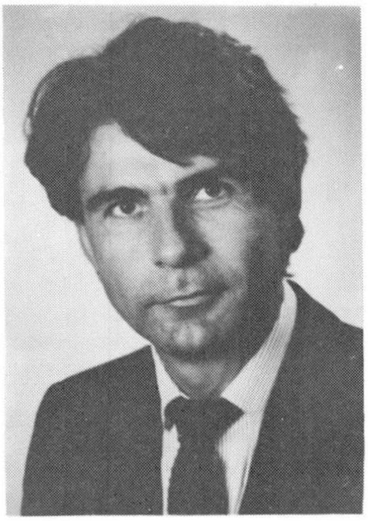

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SUMMARY

There are two major requirements when repairing prestressed concrete structures: effective analysis methods must be available and one has to be well acquainted with the technology to be used. The present paper deals with the following recent methods:

— locating of steel tendons covered by conventional reinforcement by applying the method of radar technology
— opening of prestressing ducts without any danger of damage being caused to the steel tendons
— measuring and grouting of cavities in ungrouted prestressing ducts
— reprofiling of repaired sections by using a new, high-quality wet shotcrete-method.

RÉSUMÉ

L'assainissement des ouvrages en béton précontraint demande avant tout des méthodes d'analyse efficaces et en parfaite adéquation avec la technologie utilisée. Les succès récents sont les suivants:

— recherche des câbles de précontrainte situés sous les fers à béton passifs, par l'emploi de la technologie du radar
— ouverture des gaines en tôle d'acier sans risque d'endommager les câbles de précontrainte
— mesure de volume et injection des cavités des gaines mal injectées
— fermeture des sections réparées des ouvrages avec un nouveau béton projeté à malaxage de haute qualité.

ZUSAMMENFASSUNG

Für die Instandsetzung eines Spannbetonbauwerks sind vor allem leistungsfähige Analyseverfahren und beherrschte Instandsetzungstechniken gefordert. Jüngste Erfolge, über die das Referat berichtet, sind:

— das Orten von Spanngliedern, die von schlaffer Bewehrung überdeckt sind, mit der Radartechnik
— das Öffnen von Hüllrohren ohne Verletzungsgefahr für Spannglieder
— das Messen und Verpressen von Hohlräumen in unverpressten Hüllrohren
— das Verschliessen instandgesetzter Bauwerksabschnitte mit einem neuartigen, qualitativ hochwertigen Nasspritzbeton.

1. Introduction

Post-injection of uninjected sheathings in a prestressed concrete structure may effect prolongation of its lifespan or bring about its complete destruction.

A drill point hitting a tendon under tension may possibly cause its bursting with the effect of an explosion. This would not only endanger the structure but those doing the job, too. Execution of rehabilitation work must be quality-assured. Subcontractors must prove, prior to contract award, that they are organizationally and technically in a position to fulfill the specified quality requirements. To this end, they must submit a detailed description of how they intend to solve the problem. This is what quality assurance means in control.

Easy accessabilitiy to critical points, simplicity of structural design, testability of conditions and rapid detectability of errors are criteria that mean safety and economy in the rehabilitation of existing structures. The designer of new constructions must recognize the above mentioned features as a quality criterion.

The realization of construction projects always encompasses different domains. They cannot be carried out by individual domains alone, such as structural design or production or development or analysis techniques. Technically outstanding rehabilitations require in-house organizational support for planning and testing. This strategic task involves decision-making on the company's management level.

In addition, changes in works-techniques - and changing requirements as to the knowledge of employees require selective training. The mentioned quality-assured rehabilitation measures refer essentially to the organization of planning and of job sequence.
Relevant advances in the techniques of analysis and execution are described in the following.

2. Radar technique

The radar detector developed for geological soil investigations can be used to locate the reinforcement in reinforced concrete and prestressed concrete structures. Within structural components radar detects material interfaces at which the dielectric characteristics of the material change. Such changes occur for instance with density differences. The radar technique competes with other non-destructive test methods allowing insight into the concrete. It stands out for the rapidity of its functioning, is easy to handle, yields safe localization results and is economical.

The radar equipment consists of the antenna, control unit and recording unit. A monitor, computer and data logger can be added for the purpose of documenting and evaluating the radar signals. The size of the control and recording units correspond approximately to a DIN A 3 plotter. The antenna used to detect prestressed reinforcement can be held in one hand and weighs about 2 kgs.

When handling the antenna for detection, it should sweep over the surface of the test piece at possibly constant speed. The depth and spread of a fault area or an embedded reinforcing bar are determined through calculation, the wave velocity of the radar impulse and the relative dielectric constant of the material entering into the calculation. The impulse timing is taken from the graphic radar signal recording. This shows a characteristic graph of printer lines as shown in Fig. 1. With a motionless antenna the printer lines run parallel and straight. As soon as the antenna is moved when sweeping over the surface of the test piece, the blackened ribbons produce more or less accentuated oscillations indicating the detection result to the expert.

Formerly, the radar technique could only be used for the detection of reinforcement near the concrete surface. Meanwhile, the measuring technique has advanced to a point where it is possible to detect in bridges the transverse prestressed reinforcement beneath the non-prestressed reinforcement through the bridge cap as well as through the asphalt surfacing layer.

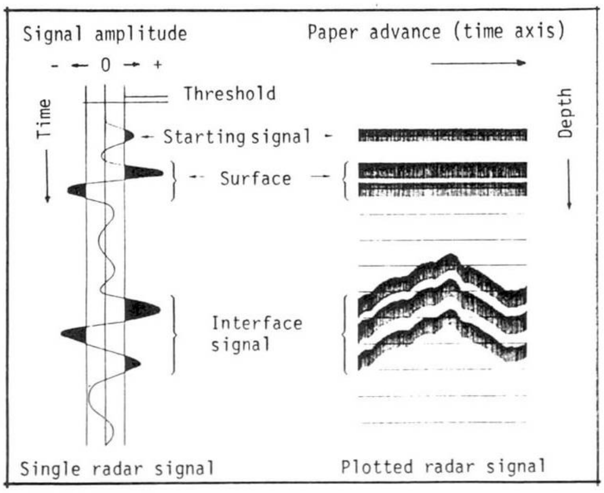


Fig.1 Graphic recording of a radar signal.

Fig.2 Guiding the radar antenna for the detection of transverse prestressed reinforcement

Latest references on radar detection of prestressed reinforcement within bridges in Germany pertain to the Köhlbrand bridge and the Kattwykdamm bridge in Hamburg as well as the Neuenkamper bridge in Remscheid.

3. Opening of sheathings

Safe and non-destructive hitting of a sheathing may just be regarded as a routine procedure. In wet drilling, a reduced electrical resistance shortly before the sheathing is reached, ensures that the drilling machine is automatically switched off. In dry drilling, an automatic switch-off device is inserted into the low-voltage circuit between drill hammer and the reinforcement. At the least contact between drill hammer bit and metal the drill hammer is switched off.

Fig.3 Drill hammer with automatic switch-off

Fig.4 The opened tendon seen through the borescope. The cavity is proof of insufficient injection.

The far greater risk of damage lies not in drilling but in the opening of the sheathing itself. Therefore the opening must be carefully surveyed. This is done by means of the borescope. This accessory, too, is meanwhile considered as belonging to the state of the art in rehabilitation works.

What is seen through the borescope can be recorded photographically and so be documented. This is a new development with opening the sheathings, because it allows everyone involved to check that the tendon was not damaged when opening the sheathing.

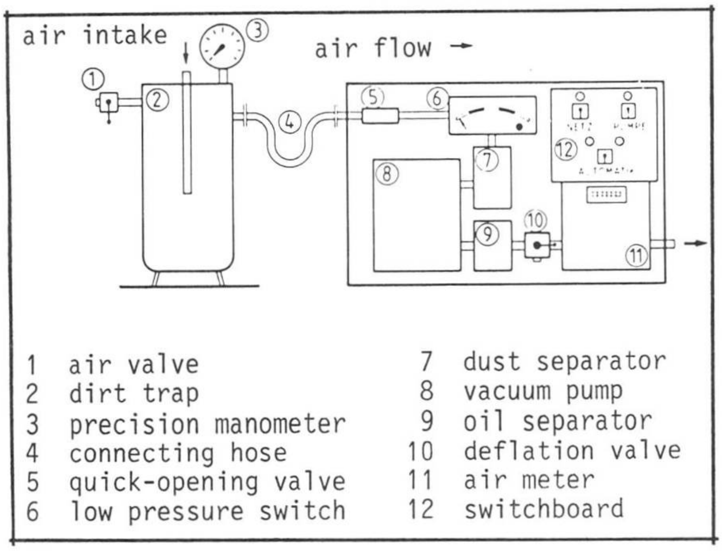
After opening of the sheathing, first the found situation as to voids must be described. Subsequently the bores are closed with injection hoses and sealing caps as protection against humidity.

4. Vacuum injection of un-grouted prestressed concrete sheathings

The patented vacuum injection procedure described below serves to fill a cavity which is accessible only through one opening with injection material.
The equipment used sucks the air out of the cavity and subsequently, via a relay valve, presses the injection material into the evacuated cavity.

The efficiency of the injection is examined by the following procedure.

Prior to the injection, the volume of the cavity is determined by measuring the pumped-off quantity of air down to a pressure of 400 mbar behind the vacuum pump. During an earlier check, the hose connections and the void were leakage-tested at 100 mbar. Leakages localized through hissing noises were sealed. In preparation of the injection, the underpressure is lowered to 50 mbar. Post pump injection assists the subsequent filling of the cavity.



Figs. 5 + 6 The vacuum procedure.

After injection the quantity of grout introduced and the earlier determined volume of the cavity are to be compared to verify the degree of fill. Suitable injection materials are epoxy resin and grout.

5. Re-profiling with "synthetic-Silica" shotcrete

Shotcrete is particularly suited for reconstituting worn concrete courses and for rehabilitating plane concrete layers protecting the reinforcing steel against corrosion. Dry and wet mix shotcretes are suitable likewise.

Wet mix shotcrete has specific quality advantages in comparison with dry mix shotcrete:

- The water-cement ratio as the most important concrete quality parameter can be selectively adjusted and maintained constant.

- Wet mix concrete production renders a homogeneous fresh concrete mix.

- Conveyance of the wet mix shotcrete involves less material and wear-and-tear costs than of dry mix shotcrete.

- The high degree of homogeneity of wet mix shotcrete counteracts differential deformations and thus non-uniform conditions.

Wet mix shotcrete requires the admixture of an accelerating agent at the spray nozzle in order to produce adequately thick and well adhering layers and to reduce rebound. Water glass accelerates setting, but if incorrectly batched, the concrete may suffer a loss of quality.

A process is being developed under which the setting of wet mix shotcrete is accelerated by physical means, so avoiding chemically effective additives.
The effect is based on the addition of powdery synthetic precipitation silica which has an extremely large specific surface in the magnitude of almost 200 m²/g. The large surface binds the surplus mixing water of the shotcrete and thus improves the cohesion of the concrete. The synthetic silica is added to the wet mix shotcrete together with the compressed air only in the spray nozzle.

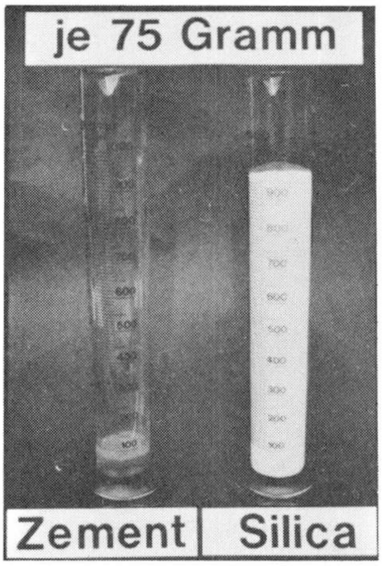


Fig.7 Comparison of volume

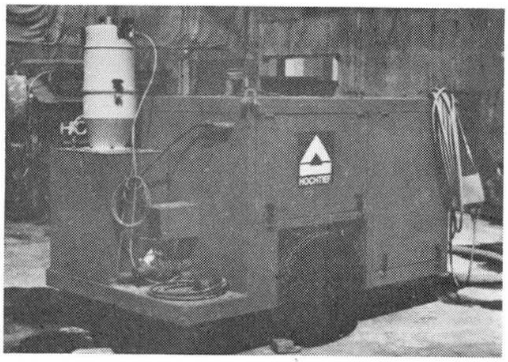


Fig.8 Batching device for loading the compressed air flow with synthetic silicic acid of 200 m²/g fineness.

The following advantages of adding synthetic silica to wet mix shotcrete are particularly important.
- No loss of concrete strength
- Extremely small rebound (less than 5 %).
- High early and final strength.
- Due to the additional silica reactions within the micro range of the hardened cement paste high resistance against chemical attack.
- Denser concrete matrix through additional formation of CSH (calcium silicate hydrates).
- Small carbonization due to high density (improves corrosion protection of reinforcing steel).

Since the silica reaction is substantially physical, the hardening acceleration does not depend upon the degree of cement used. Thus, particular requirements for applying specific cement types can be easily and safely conceded.

The silica quantities to be added to the concrete are small and amount to only appoximately 3 % of the mass of cement.
We use synthetic amorphous silica, with a degree of purity of more than 98 %.

Ultrasonic Testing of Concrete

Contrôle du béton par ultrasons

Ultraschallprüfung von Beton

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SUMMARY

A research project concerning the use of ultrasonic technology for detection of deterioration in concrete is reported. The development possibilities for the technology in detecting damage on the basis of excitation of broadband sound pulses and with evaluation based on digital signal analysis are discussed. Results are presented from a series of tests on concrete cylinders with fatigue-induced cracks.

RÉSUMÉ

Un projet de recherche concernant la détection de défauts dans le béton au moyen d'ultrasons est présenté. L'article expose les possibilités de développement de cette technique pour la détection d'anomalies, par la mise en oeuvre d'impulsions sonores à bande large et par l'évaluation des signaux au moyen d'analyse numérique. Il présente également les résultats d'une série d'essais réalisés avec des cylindres en béton présentant des fissures induites par fatigue.

ZUSAMMENFASSUNG

Es wird über die Forschung bezüglich der Schadensortung in Beton mit Ultraschalltechnik berichtet. Die Entwicklungsmöglichkeiten für diese Technik zur Kontrolle bei diffusen Schadentypen auf Grundlage der Erzeugung von breitbandigen Schallimpulsen und Auswertgung mit digitaler Signalanalyse werden diskutiert. Gleichzeitig werden die Ergebnisse einer Versuchsreihe an Betonzylindern mit Ermüdung ankündigenden Rissen untersucht.

1. INTRODUCTION

Ultrasonic measurements on concrete have in the past been used primarily for strength determinations. New opportunities for using ultrasonic technology are revealed when interest is expanded to include mapping of damage on the basis of relative assessments within a given structure. A completely new type of control is made possible through computerization of measurements and through digital storage and processing of measured ultrasonic signals.

Analysis of the transmitted sound enables not only detection of deterioration having an effect on the velocity of sound but also of deterioration that mainly affects the propagation and attenuation of the sound wave. Development of ultrasonic technology from this point of view is in progress at the Department of Structural Engineering at the Royal Institute of Technology in Stockholm.

The work is being carried out in cooperation with other research institutes in Sweden which are active in the field of non-destructive testing of concrete, see Ingvarsson [1].

2. ULTRASONIC DETECTION OF DETERIORATION

2.1 Measuring technique

Ultrasonic technology for detection of deteriorated concrete comprises a group of possible methods of measurement and analysis. The fundamental characteristic is that a ultrasonic pulse is introduced into the concrete, the signal subsequently being received after passing through the material.

Different parameters can be used to characterize investigated material of concrete. The most common is measurement of the ultrasonic velocity. The attenuation of the transmitted signal can also be used. As a measure of the attenuation of the material, it is customary to use the amplitude of the first wave in the received signal. Further studies of the received signal also make it possible to investigate those parts of the signal which have not been transmitted via the fastest route.

2.2 Deterioration criteria

The transmission of the ultrasonic signal is influenced by the changes to the acoustic properties which may be caused by deterioration. From these changes, it is therefore possible to establish criteria for the material in question. A frequently used damage criterion is low sound velocity in the concrete. A long measured transmission time, giving a corresponding low velocity, can originate from either the signal passing through a deteriorated area or from bypassing it. On transmission of a ultrasonic signal through a structure, the differences in impedance between sound and damaged concrete can cause a larger amount of energy to be reflected at the boundary surface of the damage and, in consequence, the attenuation of the transmitted signal is greater than in the case of homogeneous concrete.

Investigations of specimens with fictive damage that do not permit any transmission have for instance been carried out by Knab, Blessing and Clifton [2]. This investigation comprised velocity measurements in concrete specimens provided with fictive cracks. Moreover, Sansalone and Carino [3], have reported on work on impact-echo measurements of fictive cavities.

Measurements giving an estimate of the dispersion of the sound wave caused by a partially transmitting damage can also be of interest. In many cases, deterioration of a concrete structure occurs in the form of load- or frost-initiated cracking. Development of methods for attempting to measure and quantify this type of damage on the basis of transfer functions for concrete, will be carried out.

In establishing damage criteria which can be utilized to assess significant differences within a structure, it is also of great interest to study the magnitude of the changes in a measured signal that can be caused by deterioration in comparison with the variations arising on account of the natural inhomogeneity of the concrete. The problems caused by the test reproducibility must also be analyzed.

3. DETECTION OF CRACKS

3.1 Specimens

A series of concrete cylinders were tested with the ultrasonic technique in order to investigate the possibility of detecting cracks in concrete on account of fatigue .

Cylinders with a diameter of 100 mm were drilled out of a cast concrete slab with a height of 300 mm. These cylinders were sawn into specimens with an approximate height of 90 mm. The concrete mix consisted of Portland cement and natural aggregate with a maximum size of 16 mm. The mixing ratio by weight was: 1.00 part cement, 2.1 parts fine aggregate and 1.6parts of coarse aggregate. The water- cement ratio was 0.50 and the air content 8 %.

3.2 Performance of ultrasonic measurements

In the test series, ultrasonic measurements were carried out with direct transmission in the axial direction of the cylinders . Piezo-electrical transducers with a high frequency response in the band from 40- 160 kHz were used as sending and receiving probes. In all measurements the electric input, shown in Fig. 1, consisted of one cycle of a sinusoidal wave in 130 kHz with a voltage of 250 V. The received signal was stored in digital form with a sampling frequency of 2 MHz.

The transducers were applied to the concrete specimen in each separate ultrasonic measurement according to Fig. 1. In order to bring about a reproducible coupling the probes were mounted against a spring- loaded holder which produced a constant contact pressure. A coupling medium was applied between the probes and the concrete.

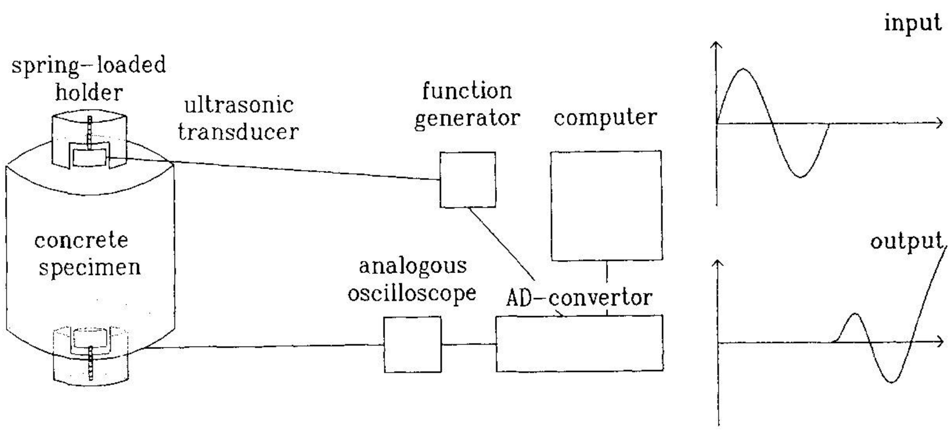

Fig. 1: Schematic diagram of test setup

The ultrasonic measurements were performed on unloaded specimens at roughly each 400,000th load cycle. The test was discontinued after 2 million load cycles if failure had not occurred. Each measurement was done with eight independent couplings of the transducers. All eight separately stored readings consisted of a time mean value of four transmitted impulses. On the occasions when ultrasonic measurements were performed, any cracks visible on the surface of the cylinder were registered.

3.3 Results from crack detection tests

In the study of concrete cylinders exposed to a uni-axial fatigue load, ultrasonic velocities and signal amplitudes were registered in order to study if these parameters can give an indication of crack development.

The changes in amplitude were studied by calculation of a coefficient of reduction, defined as

$$D_a = 1 - \frac{A}{A_0}$$

where A is the average of the amplitudes of the first received wave in eight registered signals from a specific specimen and test occasion and A_0 is the corresponding value prior to loading.

The reduction in velocity was determined in a corresponding manner.

A comparison between λ, the total length of visible cracks relative to cylinder height and calculated reductions in amplitude, D_a, and velocity, D_v, for the specimens is presented in Table 1.

Number of load cycles		Specimen 1	2	3	4	5	6	7	8	9	10	11
	Da	0	0	0	0	0	0	0	0	0	0	0
0	Dv	0	0	0	06 0	0	0	0	0	0	0	
	λ	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Da	23	34	1	22	5	5	90	23	-	16	52
400000	Dv	9	13	6	9	7	10	15	9	-	10	19
	λ	-	1.0	0.5	2.0	0.8	1.0	4.3	2.0	-	1.3	5.6
	Da	30	42	20	30	7	15	96	30	-	28	78
800000	Dv	10	13	7	11	9	13	22	11	-	11	24
	λ	-	4.3	0.5	2.0	1.3	2.0	10.1	2.0	-	2.3	7.5
	Da	35	37	83	36	9	16	-	61	-	31	97
1200000	Dv	10	14	13	12	9	13	-	11	-	14	31
	λ	-	4.6	5.9	3.2	1.3	2.5	-	2.0	-	3.4	11.5
	Da	33	36	88	36	3	59	-	39	-	30	-
1600000	Dv	11	14	14	12	9	18	-	12	-	13	-
	λ	-	4.6	5.9	4.6	1.3	7.2	-	2.0	-	3.4	-
	Da	31	40	-	37	3	-	-	35	-	32	-
2000000	Dv	10	14	-	12	9	-	-	12	-	13	-
	λ	3.5	5.3	-	4.8	1.3	-	-	2.0	-	4.0	-

Table 1: Coefficients of Amplitude Reduction, D_a (%), Coefficients of Velocity Reduction, D_v (%), and total lengths of visible cracks relative to cylinders heights, λ for specimens 1-11.

Mean values of velocities and amplitudes from signals measured with renewed coupling are presented in Fig. 2 together with the corresponding standard deviations.

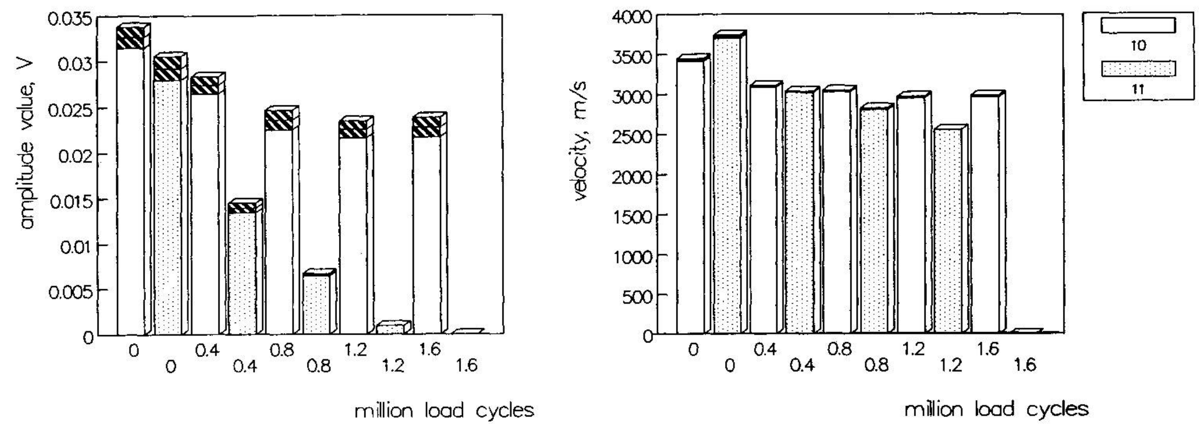

Fig. 2: Mean values with standard deviations of eight repeated measurements from specimens 10 and 11, at different stages of loading.
Amplitude of first received sound wave, left and Velocity, right.

Apart from the deviations that can be calculated for selected analysis parameters, the reproducibility can be studied by calculation of the averaged coherence function between input and output signals. By this means a picture is obtained of how well the included frequencies are reproduced in all of the received signal. The coherence of the repeated measurements was therefore registered in each measurement stage. Calculated coherence functions for one of the cylinders are shown in Fig. 3 referring to two different occasions.

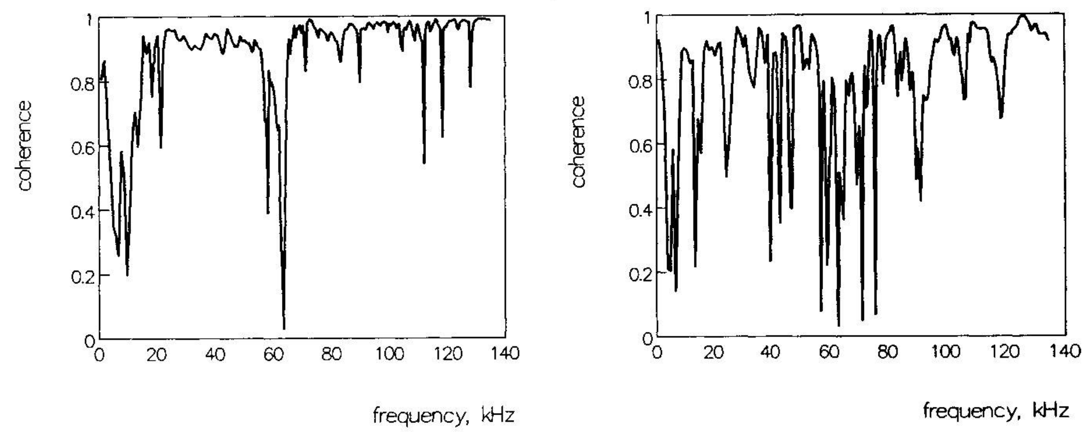

Fig. 3: Coherence functions for measurements on specimen number 11 at the initial stage, left, and after 0.8 million load cycles, right.

The duration of the first received wave was shown to be changed due to cracks. The appearance of this wave is shown in Fig. 4 for one of the specimens at different degrees of cracking.

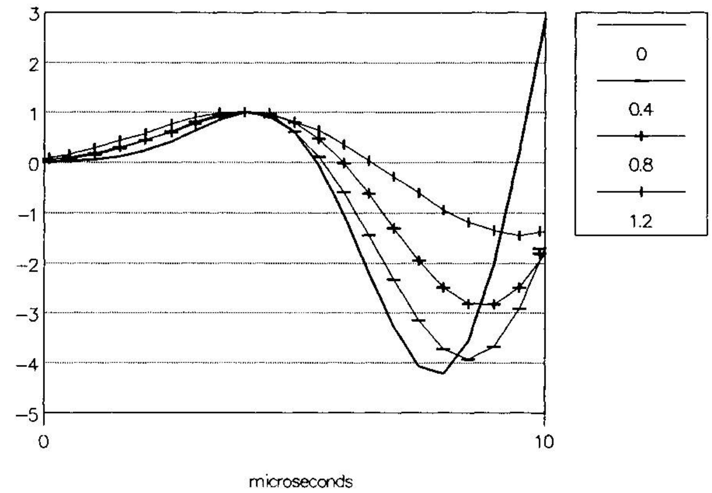

Fig. 4: First parts of received waveforms from specimen 11 at 0, 0.4, 0.8 and 1.2 million cycles. Amplitude values are normalized with respect to the first received peak.

4. CONCLUSIONS

This investigation shows that determination of ultrasonic parameters can be a meaningful way to analyse deterioration. Severe cracking is shown to give significantly greater effects in the values recorded than those related to the natural lack of homogeneity of the material.

The changes upon deterioration will be greater for amplitude measurements than for velocity measurements. The variation in values recorded with repeated measurements is however greater for amplitude than for velocity measurements. Several repeated measurements are therefore required to obtain a reproducible amplitude parameter. It is also shown that there are increasing difficulties to reproduce the entire signal when the concrete is deteriorated. A lack in coherence can in itself be an indication of an increased number of cracks in the concrete specimen.

The duration of the first wave of the received signal will also be longer and thus contain lower frequencies with increased damage. The changes in the frequency contents of the received signal due to deterioration is therefore interesting.

Signal analytic studies of the concrete frequency response due to ultrasonic stress wave exitation combined with control of the coherence of the measurement are thus very interesting to carry out for deteriorated concrete. Further research on detection of internal cracking due to fatigue and frost-cycles, using ultrasonic signals and digital signal analytic evaluation, is thus considered to be of considerable interest and will be further studied.

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Acoustic Emission Evaluation of Concrete Structures

Evaluation de l'émission acoustique des structures en béton

Bewertung der akustischen Emission einer Betonstruktur

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SUMMARY

This paper presents an experimental study to evaluate the degree of structural integrity using acoustic emission measurement as a method of nondestructive inspection. According to the results, the following possibilities of acoustic emission measurement were confirmed: to predict the generation of cracks during a month after placing of mass concrete; and to evaluate the deterioration degree of aged concrete structures.

RÉSUMÉ

Ce document présente une étude expérimentale pour évaluer le degré d'intégrité structurale à l'aide de mesures d'émission acoustique en tant que contrôle non destructif. Les résultats obtenus ont confirmé les possibilités suivantes de la mesure d'émission acoustique: afin de prévoir la formation de fissures dans les jours suivant la mise en place du béton, et afin d'évaluer le degré de détérioration d'anciennes structures en béton.

ZUSAMMENFASSUNG

Diese Arbeit stellt eine Experimentalstudie zur Bewertung der strukturellen Unversehrtheit vor, die als zerstörungsfreie Untersuchung die Messung der akustischen Emission benutzt. Die Ergebnisse bestätigten, dass die Messung der akustischen Emissionen folgende Möglichkeiten bietet: auf die Rissbildung in den ersten Tagen der Aushärtung zu schliessen, und auf den Alterungsgrad einer Betonstruktur zu schliessen.

1. INTRODUCTION

A large number of reinforced concrete(RC) structures constructed during the high-growth age of Japanese economy are now known to be approaching their service limit and have to be repaired or reinforced. Recently, in addition, the short-term deterioration of RC structures due to poor quality of construction, thermal stress, salt damage, alkali-aggregate reaction, and freezing and thawing action have been reported and generated widespread problems. Therefore the development of techniques which enable to readily evaluate the degree of deterioration of aged RC structures and to restore them are urgently required. Acoustic emission(AE) measurement is a well-known method for detecting microscopic cracks generated in the solid. Thus, basic experiments were performed to confirm AE properties in concrete and to apply AE measurement as one of the evaluation method of the structural integrity of RC structures. This paper reports the results of the basic experiments and presents some inspection methods of AE measurement as one of nondestructive testings.

2. INSPECTION METHODS OF ACOUSTIC EMISSION MEASUREMENT

Applications of AE measurement for the evaluation of the structural integrity of concrete are devided roughly into three categories.

(A) Quality controll of new construction.
(B) Evaluation of deterioration degree of existing structures.
(C) Evaluation on accelerated deteriorating tests.

Inspection methods of AE are closely associated with these applications, and would be devided into three categolies.

(a) To predict the progress of microscopic cracks by continuous or intermittent monitoring of AE signal in RC structures.
(b) To evaluate the structural integrity of RC structures by detecting AE signal from the structures driving external forces or elastic waves.
(c) To evaluate the material integrity of concrete by performing AE measurement during uniaxial compressive testing and to analyze the effects of existing cracks.

The choice of inspection methods is dependent upon the target of applications. Table 1 indicates possible combination for the purpose of applied fields. For more accurate evaluation, of cource, combination of other nondestructive testings and AE is desirable. In this study, basic experiments connected with the applications of AE measurements (A) and (B) were carried out.

3. ACOUSTIC EMISSION BEHAVIOR ON MASS CONCRETE

An experiment was performed to help predict the generation of cracks during one month after placing of mass concrete using AE measurement in the laboratory. The mix proportion of the concrete are shown in Table 2. Dimensional data of the mass concrete specimen and the locations of sensors installed in it are shown in Fig.1. Measurement items are temperature, longitudinal stress, longitudinal displacement and AE to obtain the information about the generation of microscopic cracks in the specimen. AE was detected using six transducers through wave-guides. In Fig.1, dotted points A,B,C,D,E, and F show the locations of six transducers to detect AE signals. The conditions of AE signal detection decided by preliminary examinations, are shown in Table 3. The effects of noise can be eliminated because the range of frequency of detected AE is about 100-300kHz and much higher than that of noise in the laboratory. Fig.3 shows the relationship between the time elapsed after placing of concrete and the temperature, longitudinal stress, logitudinal deformation and event counts of AE per 2 hours. Temperature at the central portion of the specimen rose to the peak(76C) at 1.5 days. At that time, compressive stress and tensile stress were set up at the central and the upper portion respectively. After removal of forms at 2 days, the temperature fell rapidly and both stresses changed to the opposite behaviors. Shrinking deformation of the specimen occured during the falling of its temperature, and the deformation at the upper portion was larger than that at the bottom portion. A large number of AE were detected during about 3 days after placing of concrete, but after that, the number of AE events decreased with time elapsed and finally became constant. A large number of AE were detected one day before the first discovery of cracks at the upper surface of the specimen. The generation of AE continued until one month after placing, and at that time, hair-cracks became visible. According to the results, it is confirmed that it is possible to predict the generation of cracks by AE measurement during one month after construction. It is considered that the causes of generation of cracks in the mass concrete specimen are as follows.

1) The cause of generation of the first crack at 2 days after placing of concrete is subsidence of concrete.
2) The cause of generation of cracks at one month is the combination of thermal shrinkage and drying shrinkage of the specimen.

4. TEST OF THE CORE SPECIMENS EXTRACTED FROM THE AGED CONCRETE STRUCTURE

The Shinomiyajuku bridge(Photo.1) was crossing Shakujii river and had been in service for 20 years after construction. Dimensional data of the bridge are as follows.

Height of abutment	:	4.2 m
Width of abutment	:	9.0 m
Width of girder	:	11.0 m
Span	:	10.0 m

Concrete cores were extracted from the abutment of the bridge. Coring(cylinderϕ 10cm×50cm) was performed as shown in Fig.4 and two sample specimens(ϕ10cmx20cm) were made from each core. The specimens were curred in water for one month and cured in the moisturized condition for one week. AE measurement was performed to evaluate the material integrity of the core specimens during a test of uniaxial compressive strength. Fig.5 show the uniaxial compressive loading system and the AE measurement system emplyed. A compressive loading test was carried out using two teflon sheet(thickness:2mm) laid between the loading plates and the specimen. Loading rate was set at 550N/sec. Table 3 shows the conditions of AE signal detection. AE measurement system includes the one-dimensional location system using two transducers installed on the side of the specimens in the axial direction. Determination of AE source location was performed only at the center portion(10cm) of the specimens in the axial direction as shown in Fig.5, in order to eliminate the effects of noise generated by the friction between the specimen and the loading plates. The measurement error of the source location was confirmed within several millimeters by preliminary examinations. Test results of uniaxial compressive strength and static Young's modulus of the specimens are shown in Fig.4. Compressive strength is lower at the surface portion and higher at the inner portion of the abutment. Static Young's modulus shows a similar tendency. The relationship between the compressive stress and AE events detected during uniaxial compressive testing are shown in Fig.6. According to these results, in the specimens made from the surface portion of the core, many AE signals were detected at a low stress level. On the other hand, in the specimens made from the inner portion, the number of AE signals detected at a low stress level was small. In addition, the speed of longitudinal wave through the specimens was about 3500 m/sec at the surface portion and that was about 4500 m/sec at the inner portion. The reason is considered to be the degree of internal defect in the concrete specimens.

5. CONCLUSION

Conclusions of these basic experiments are as follows.

(1) AE measurement is effective to predict the generation of harmful cracks during one month after placing of mass concrete.

(2) AE measurement is effective as well as compressive testing for evaluating the material integrity of core specimens extracted from existing structures, bcause the degree of internal defects can be estimated by it.

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Table 1 Applications and Inspection Methods of AE Measurement

Applications	Inspection Methods
(A)	(a) (b)
(B)	(a) (b) (c)
(C)	(a) (b) (c)

Table 3 Conditions of AE Detection

Wave Speed	2500-4500	(m/sec)
Attenuation	35	(dB/m)
Pre-gain	40	(dB)
Main-gain	40	(dB)
Threshold Level	1.0	(V)

Table 2 A Mix Proportion of Concrete

G_{max} (mm)	Slump (cm)	Air (%)	W/C (%)	S/a (%)	Unit Weight (kg/m^3) W	C	S	G	Ad.
25	12	4.0	43.5	42.7	171	400	749	994	1.00

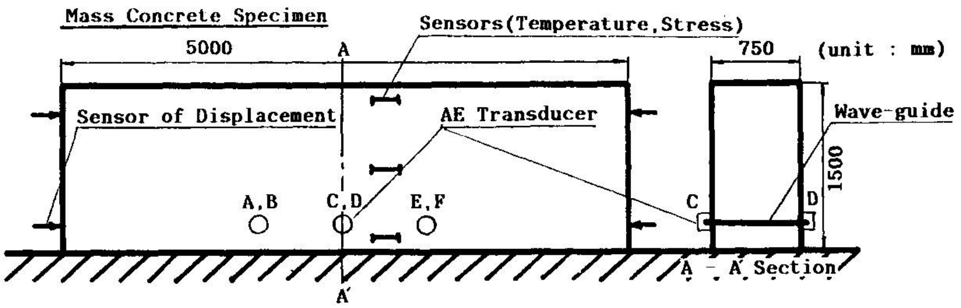

Fig. 1 Mass Concrete Specimen and Locations of Sensors

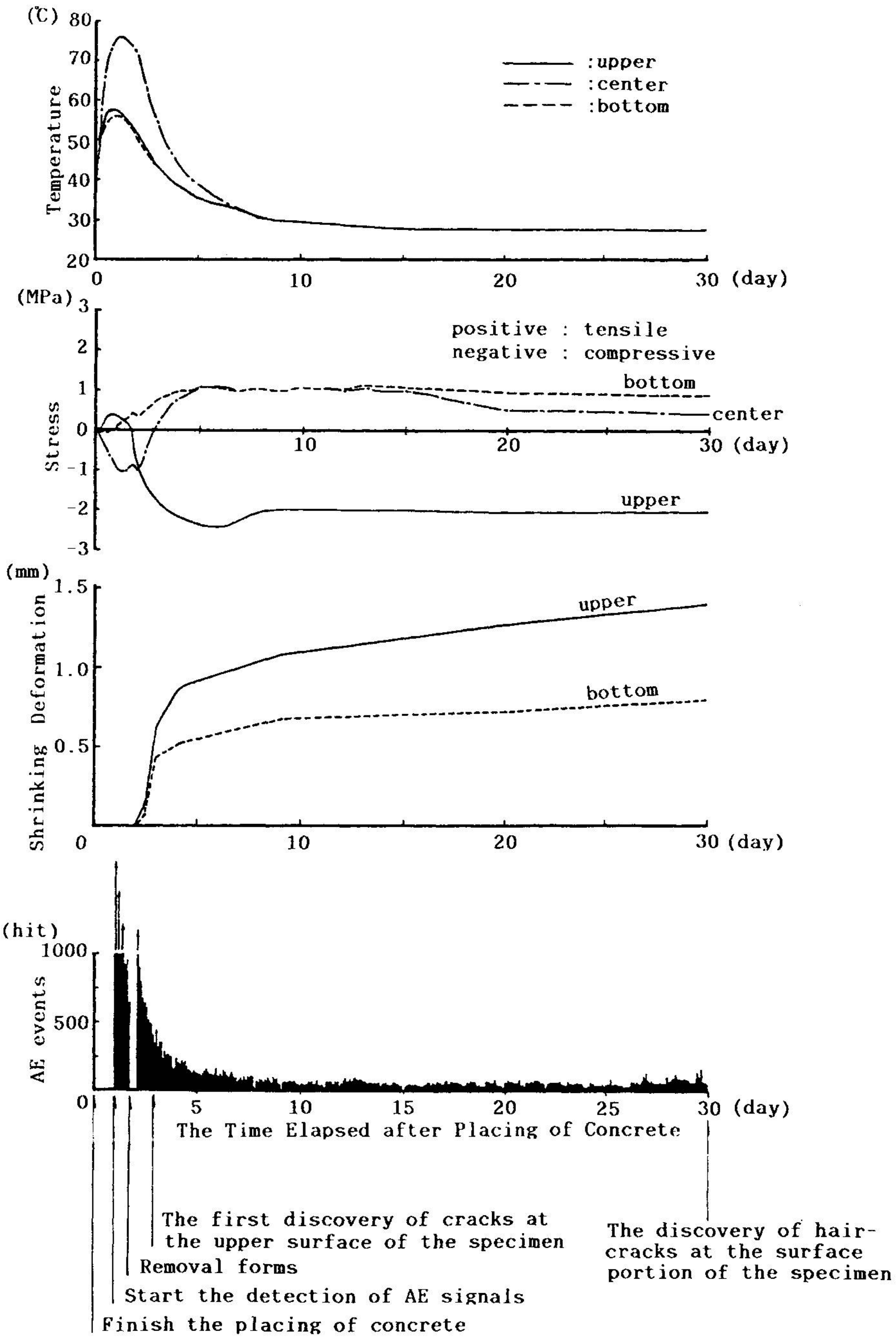

Fig. 3 The Time Elapsed versus Temperature, Longitudinal Stress, Longitudinal Deformation and AE Events

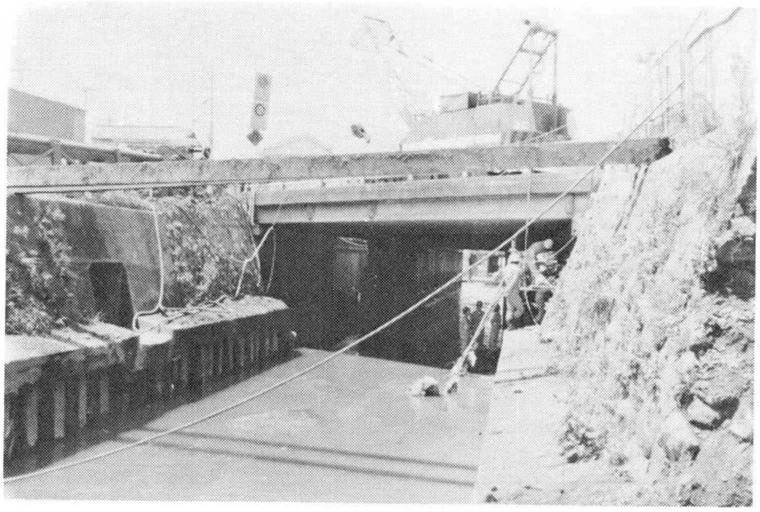

Photo. 1 Scene of the Shinomiyajyuku Bridge

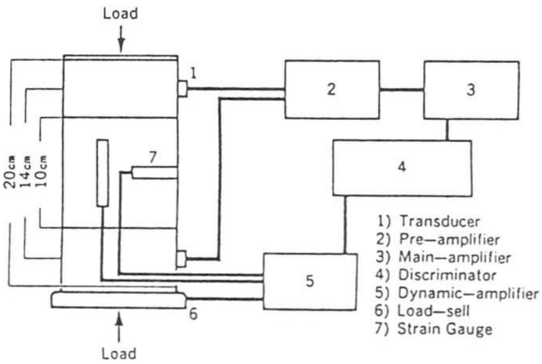

Fig. 5 Uniaxial Compressive Loading System and AE Measurement System

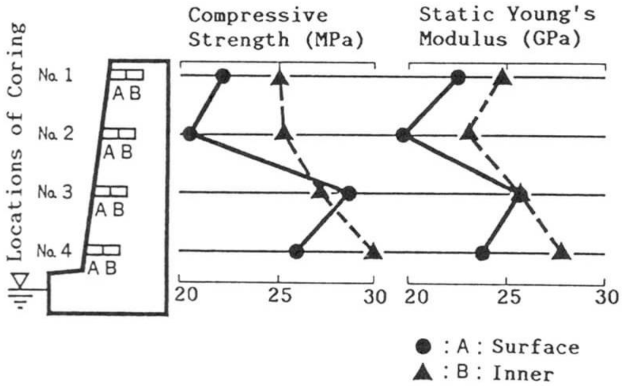

Fig. 4 Compressive Strength and Static Young's Modulus of The Core Specimens

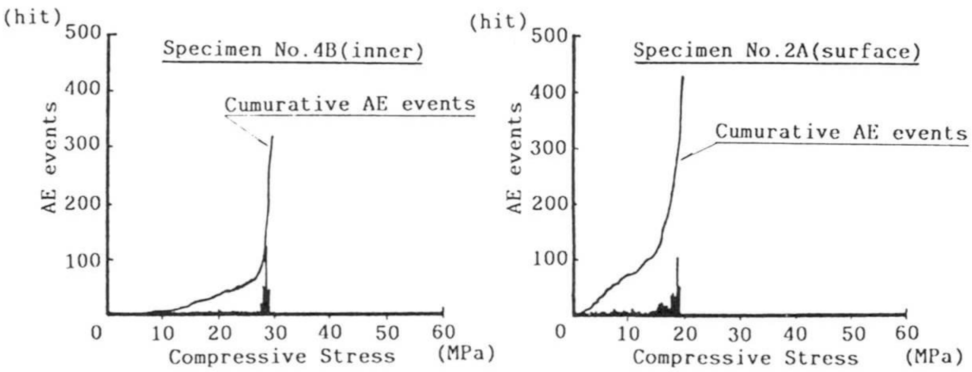

Fig. 6 Relationship between Compressive Stress and AE Events during Uniaxial Compressive Testing

Magnetoelastische Kraftmessung im Spannbeton

Magnetoelastic Force Measurement in Prestressed Concrete

Mesure magnéto-élastique de la force dans le béton précontraint

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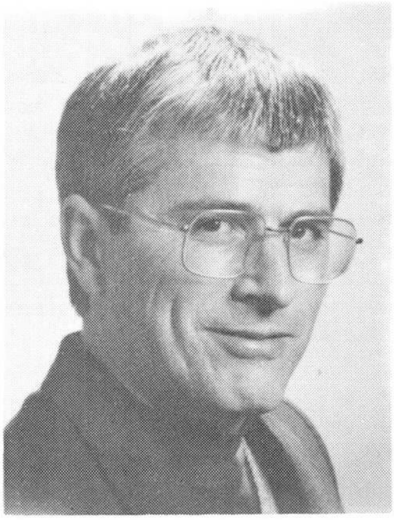

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ZUSAMMENFASSUNG

Die magnetischen, magnetoelastischen und andere damit im Zusammenhang stehenden physikalischen Eigenschaften von Spannstahl als Grundlage für die magnetoelastische Kraftmessung in den Spanngliedern von Spannbeton werden behandelt. Ein neuartiges empirisches Gesetz, welches die Modellierung der magnetoelastischen Messdaten gestattet, wird angegeben.

SUMMARY

This paper discusses the magnetic, magnetoelastic properties and other related properties of prestressing steel as a basis for magnetoelastic force measurement in the tendons of reinforced concrete. A novel empirical relationship for interpreting the magnetoelastic measurement data is presented.

RÉSUMÉ

Cet exposé traite des propriétés magnétiques, magnéto-élastiques et autres propriétés physiques liées aux aciers de précontrainte qui servent de base à la mesure magnéto-élastique de la force dans les câbles tendus dans le béton précontraint. Une nouvelle loi empirique permettant une modélisation des valeurs de mesures magnéto-élastiques est présentée.

1. INTRODUCTION

Since World War II, the number of architectural applications of prestressed concrete has been increasing continually. Today, countless structures throughout the world have been erected using this technology. Unfortunately, these structures are not always free of errors in design and execution. Combined with today's severe environmental conditions, this has led to increased evidence of damage. More than ever, the task of investigating exact damage causes within the framework of required maintenance work is of topical interest.

Although prestressed concrete behaves basically the same as untensioned reinforced concrete concerning the damage mechanism, the damage becomes especially significant in this type of construction in relation to the tendons, resp. the tensile force in them. Thus, knowledge of this tensile force is extremely important, above all where the condition of the structure is regarded as especially critical. Unfortuantely, at present there is hardly any possibility of determining the force occurring in a tendon of a prestressed concrete structure without entirely or partly destroying the tendon, unless special measures had been undertaken when the structure was erected. Methods to permit such a measurement without weakening the tendon were therefore explored.

One possibility is the magnetoelastic force measurement. The basic principles of this method and early experience with it are reported here.

2. FUNDAMENTALS

The magnetoelastic effect is based on the principle that the magnetic properties of a ferromagnetic wire change under the influence of a tensile or compressive force. The (J,H) curve which describes the magnetic behavior of a ferromagnetic material is deformed under application of a tensile force (see Fig.1).

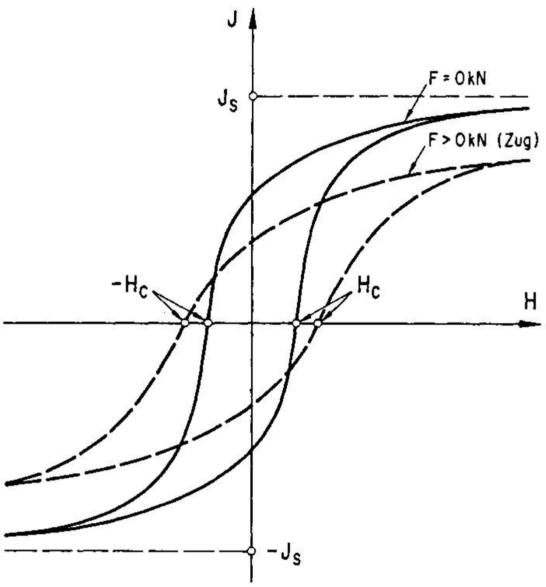

Fig.1 Magnetoelastic effect on prestressing steel

This magnetomechanical effect was discovered by E. Villari [Vil], who published his magnetoelastic observations in the middle of the last century. Closely related with the magnetoelastic effect is magnetostriction, discovered and published approx. 20 years before Villary by J.P.Joule. In this effect, the length of a ferromagnetic wire is changed by magnetizing it with a current-carrying coil. If the wire becomes longer, the effect is described as positive magnetostriction; otherwise as negative. Magnetostriction is also sometimes referred to as the JOULE effect. In everyday life, the effect can be observed in the buzzing of electrical transformers. Likewise, the magnetoelastic effect is known as VILLARI-effect.

Since changes of force and length on the same specimen are related through HOOK's law, the two effects can be regarded as inverse to one other. The idea to use the magnetoelastic effect for the force measurement originated with the originally Swedish firm ASEA (now ABB). The magnetoelastic stress gauge, designated as PRESSDUCTOR [Dah] proved to be very robust and suitable for severe industrial applications, such as the measurement of crane loads. Based on this idea the Belgian firm BEKAERT-COCKERILL attempted to construct an instrument for measuring the tensile force in the tendons of prestressed concrete. However, the force measurement was not based upon the principle of the stress gauge. Instead, it was attempted to utilize the prestressing steel itself as the measurement probe.

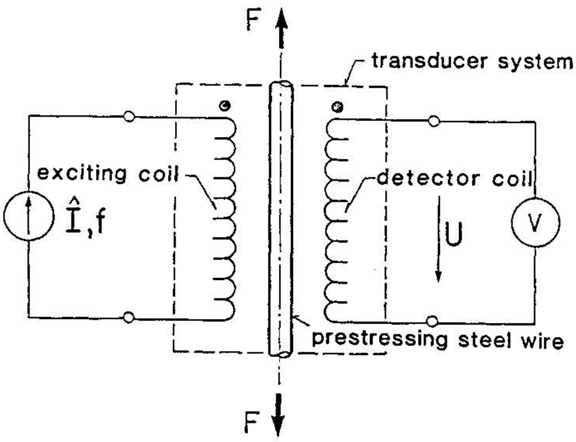

Fig.2 Magnetoelastic F-measurement

The technique of using stress gauges has the disadvantage that a measurement can be carried out only on the ends of the tendon, that is at the anchorages. This limitation can be avoided when stress gauges are not employed. Instead of the stress gauges, a measurement coil system is employed. This can be installed at any given position along the tendon. Offsetting this major advantage is the disadvantage that the force measurement is dependent upon the chemical composition and grain structure of the tendon. Since prestressing steel is a closely controlled material in regard to its mechanical strength (1500-1800 N/mm^2), its chemical composition lies within close tolerances.

3. THE MAGNETOELASTIC MEASUREMENT TECHNIQUE

The technique developed by the firm BEKAERT with the help of the Université Catholique de Louvain [Hal] is based on the following principle:

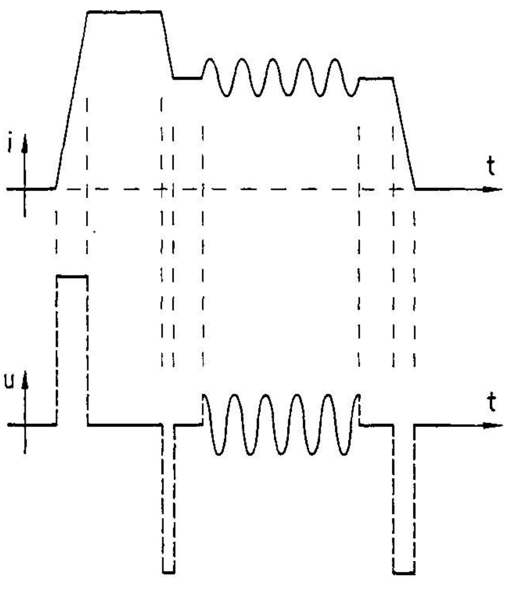

Fig.3 Excitation- and detectionsignal

An alternating current measurement impulse with a superposed alternating current burst as shown in Fig. 3 is applied to the exciting coil (see Fig. 2). The finite flanks of the measurement pulse limit the amplitude of the induced disturbing impulses in the detector winding. The current impulse at the onset drives the magnetization of the prestressing steel into saturation. This creates a defined working point on the (J,H) curve and serves to improve the repeatability of the measurement. The amplitude is then reduced to a second lower level. The actual measurement begins with the onset of the superimposed sinusoidal oscillation. This sinusoidal current induces a sinusoidal measurement voltage in the detector winding. This is measured with an RMS-voltmeter. The amplitude of the AC voltage at the output of the detector winding will be smaller of larger as a function of the magnitude of the mechanical tensile force in the prestressing steel.

The calibration curves of the firm BEKAERT were successfully repeated by the

Section Measurement Techniques of the EMPA and their linearity was confirmed. Nevertheless the following difficulties arose:

1. Since the measurement frequency of the currrent impulse is lower than 50 Hz, the low-pass filter of the RMS-voltmeter has an influence on the measurement result. However, this influence can be evaluated and compensated numerically.

2. By repeated measurement on the same specimen, the prestressing steel becomes warm due to eddy currents. Furthermore, the exiting coil heats up strongly due to copper losses.

3. In the flanks of the measurement impulse, large voltage peaks occur in the detection winding.

4. A visual inspection indicated that the quality of the electronic circuitry was deficient.

These observations led to an internal development at the EMPA within the framework of a research and development project to improve bridge maintenance.

4. MEASUREMENT OF THE ELCTRICAL, MAGNETIC AND MAGNETOELASTIC PROPERTIES OF PRESTRESSING STEEL

Since civil engineers are generally not interested in the electrical and magnetic properties of the material prestressing steel, technical data of this kind is not available. With respect to the electrical properties the electrical conductivity is of interest. This is significant in connection with eddy currents in the prestressing steel wire. To measure this quantity, three 7mm-diameter prestressing steel specimens from 3 different manufacturers were utilized (see Table 1). The ohmic resistances were determined with a Thomson bridge. The measurement of the (J,H) curves was carried out with a permeameter [IEC]. Thus, the magnetic data of three prestressing steel specimens were established for the unloaded condition (F = 0 kN). In order to be able to execute magnetoelastic measurements on wire specimens under tension, the construction of a tensioning frame was necessary. For the frame material,the nonmagnetic materials wood or unreinforced concrete were considered. Eventually wood was chosen, since it offers various advantages:

1. In dry condition, wood is an electrical insulator,
2. Wood is magnetically neutral,
3. Wood has a low specific density

and

4. Wood is naturally fiber-reinforced.

The wooden tensioning frame was dimensioned for a tensile force of 80 kN. The frame was designed to allow measurements with the permeameter as well as with measurement coils. With this frame, measurements were performed at various tensile stresses in 5 kN steps up to 30 kN. The same measurements, on the same prestressing steel specimens, were carried out as in the unloaded condition (F = 0 kN).

5. MEASUREMENT RESULTS

This section presents all of the data pertinent to the magnetoelastic force measurement, obtained on the three prestressing steel specimens from the three manufacturers.

5.1 Mechanical Properties

The 7 mm prestressing steel specimens from the three suppliers were coded with colors (see Table 1). The table also gives the mean strengths (determined from the mean values from the material specifications), as well as the minimum strength values, which are important for the dimensioning of structures. The mean diameter of the prestressing steel specimens is (7.00 $\pm$ .02)mm.

Supplier:	Color code	Min. value N/mm^2	Mean value N/mm^2
VOGT	RED	1670	1777$\pm$15
BEKAERT	YELL	1670	1757$\pm$17
F + G	BLUE	1670	1732$\pm$19

Table 1 Mechanical Strength Values of the Prestressing Steel Specimens

5.2 Chemical Composition

Since one of the prestressing steel specimen suppliers did not supply data on the chemical composition these measurements were performed in the EMPA Laboratories (Table 2). The majority of the EMPA values agreed with those of the suppliers within the measurement accuracy of the analysis instruments.

Supplier:		VOGT	BEK'RT	F + G
C	M.-%	.80 (.82)	.83	.77 (.82)
Si	M.-%	.31 (.29)	.27	.19 (.25)
Mn	M.-%	.61 (.55)	.65	.62 (.57)
S	M.-%	.013 (.016)	.016	.040 (.018)
P	M.-%	.013 (.017)	.026	.003 (.008)

Table 2 Chemical Composition of the Prestressing Steel Specimens, (.nn) = respective data from the supplier

5.3 Electric and magnetic Measurements

From the electrical measurement, only the resistance measurement are mentioned here, since they are significant for the eddy-current heating. The mean specific electrical resistance is calculated from the measurement data. It lies in the range (200 $\pm$ 20%)mOhm(mm^2/m). The influence of the Si-content on the specific electrical resistance of electrical steels is known. The higher the Si-content, the higher is the specific electrical resistance of the prestressing steel.

The magnetic behavior of the prestressing steel can be described through four quantities, namely the initial permeability μ_{ar}, the coercive field strength H_c, the residual induction B_r and the saturation polarization J_s. The measurement data of these four magnetic quantities are given in Table 3.

Supplier:	Color code	μ_{ar}	H_c	B_r	J_s
		1	A/cm	T	T
VOGT	RED	57	13.2	1.26	2.03
BEKAERT	YELL	62	13.4	1.24	2.03
F + G	BLUE	63	12.4	1.22	2.06

Table 3 Magnetic Properties of Prestressing Steel

5.4 Magnetoelastic Measurements

In these measurements with the tensioning frame, another independent quantity the force comes into play. It appears reasonable to attempt to reduce the comprehensive measurement data. This is accomplished by modelling the (J,H) curve. Since the linearity of the magnetoelastic quantity $\mu_{\Delta r} = \mu_{\Delta r}(F)$ improves with increasing field strength, it appears reasonable for F = 0 to assume a magnetization law for high field strength given by WEISS [Boz.,p.484]

$$\frac{J}{J_s} = \left(1 - \frac{\alpha H_o}{H}\right).$$

With the measurement data of the (J,H)-magnetization curve for F=0, the J_s and αH_o values can be determined. This law can now be extended:

$$\frac{J}{J_s} = \left(1 - \frac{\alpha H_o}{H}\left(1 + \frac{F}{\beta F_o}\right)\right),$$

so that the magnetoelastic behavior can be described quantitatively. In an analogous fashion, the parameter βF_o can be determined with the help of the data for F>0.

For the specimen VOGT(RED) the following values are obtained:

$$a = \alpha H_o = 19.9 \text{ A/cm} \quad \text{und} \quad b = \beta F_o = 40.1 \text{ kN}.$$

Using these values, a very good agreement between measurement and model was achieved for the higher field strengths.

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Test Methods for On-site Assessment of Durability

Essais pour l'estimation in situ de la durabilité

In situ Prüfung der Dauerhaftigkeit

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ZUSAMMENFASSUNG

Zur Dauerhaftigkeitsprognose sind Methoden zur Prüfung der Betondichtigkeit erforderlich. Die Möglichkeiten und Grenzen einiger Methoden werden erläutert.

SUMMARY

Prediction of durability requires on-site test methods for the assessment of tightness of concrete cover. Potentials and limits of several methods are discussed.

RÉSUMÉ

Pour la prévision de la durabilité des méthodes d'essai pour mesurer la perméabilité du béton sont nécessaires. Les possibilités et limites de quelques méthodes sont discutées.

1. INTRODUCTION

Tools for the assessment of durability of reinforced concrete members become increasingly important. The necessity for such tools arises already for the just completed structure and for the older one as well: quality control for the just completed structure; assessment of durability and decision for repair for the older structure.

In a climatic region such as the FRG, the most common damage is the corrosion of the reinforcement adjacent to the exposed surface. Corrosion commences once the surface of steel has been depassivated and if certain electrochemical prerequites coexist. Depassivation of steel and initiation of corrosion depend on the transport of gases and fluids through the pore structure of concrete and on the ensueing chemical reactions [1]. Thus, durability depends on depth, tightness and chemical composition of concrete cover.

This fact led to the development of test methods to assess the perviousness of the cover. The description and appraisal of these methods, their potentials and limits, will be dealt with.

2. SURVEY OF TEST METHODS

2.1 Necessity for on-site test methods

For the assessment of the condition several test methods are available. These are usually laboratory methods for the determination of: carbonation depth, chemical composition and pore structure of concrete, coefficients of gas and water transport, etc. These methods usually are destructive ones because they require core extraction. If a statistically sound appraisal of condition is needed, high costs arise.

Of advantage are on-site, non-destructive tests for the determination of the cover. Also available are on-site test methods of the perviousness of cover. These tests may substitute laboratory tests. They can be performed repeatedly, economically, and expediently.

2.2 Existing methods

Most methods for the on-site testing of the tightness have been developed for quality control of the just fabricated precast element and less for the assessment of durability of the aged structure. The principal aim of all methods is a qualified information on the influence of curing or the combined influence of curing and water/cement ratio on the diffusivity of pore structure for CO_2 and water vapour.

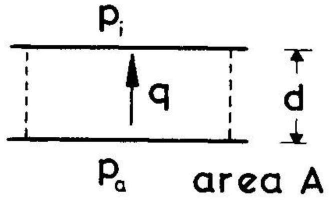
A suitable method must meet certain standards with respect to: evidential strength, selectivity, repeatability, simplicity and cost. The methods dealt with here represent a selection, they meet these standards. The following methods are discussed: the air permeability test methods by Schönlin and Hilsdorf [2] and by Figg [3], and the initial surface water absorption test ISAT [4].

All methods presuppose that a low initial perviousness will render a high durability. This assumption is necessary, but not sufficient; furthermore it is essentially not verified. In course of the specific test, only the first millimeters of cover are permeated. This fact raises the question of representativity of test results for the total cover's protective quality [7].

2.3 Transport mechanisms and test methods

For carbonation and steel corrosion the diffusion of oxygen, carbon dioxyde and water vapour as well as the sorption of aqueous solution are the relevant transport mechanisms. Diffusion at natural conditions is extremely slow. This fact and unsurmountable experimental difficulties discard diffusion's measurement on-site. As substitute for gas diffusion other transport mechanisms are chosen for on-site testing, such as the permeation of gas or fluid and the absorption of water.

Permeability testing: These tests are used for the assessment of concrete quality and curing [2], [5]. The driving potential is a pressure difference either below or above the atmospheric pressure (medium: air or nitrogen). The permeability K for the stationary flow of nonsorbent gas is described by the Hagen-Poiseuille law (Fig. 1). The principal set-up of the permeability test of Schönlin-Hilsdorf (a, [2]) and Figg (b, [3]) are shown in Fig. 2. Both methods work in the unsteady pressure range. The lapse of time Δt is measured during which a defined initial pressure p_{io} is relaxed by a definend difference Δp of the known gas volume V. The time difference Δt is used to express the permeability index [2].



$$q = KA \frac{p_a - p_i}{d}$$

q flux of gas
K permeability m²/s
d layer of material

Fig. 1: Hagen-Poiseuille's Law

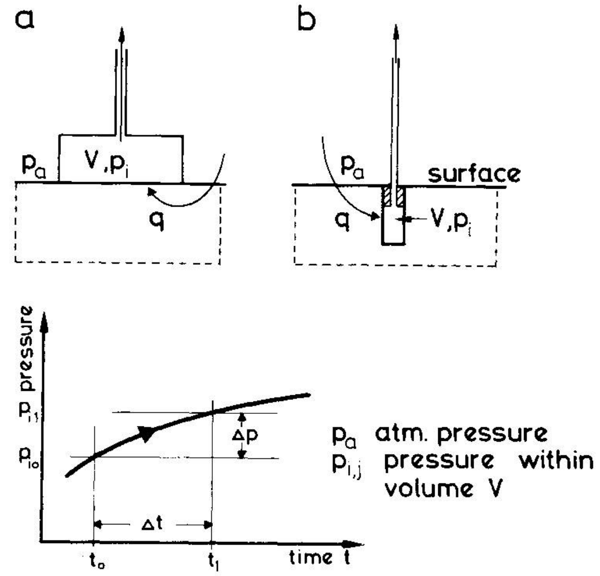
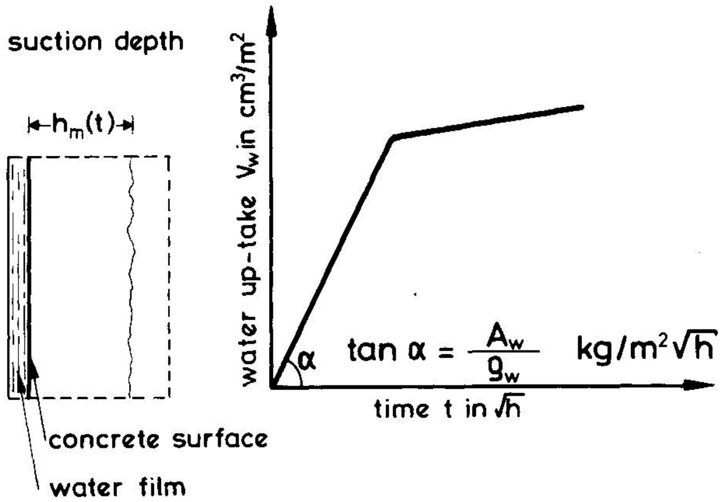


Fig. 2: Permeability test of Schönlin-Hilsdorf (a) and Figg (b)

$$I_{perm} = \frac{V}{\Delta t} \frac{\Delta p}{p_a - \Delta p/2} \tag{1}$$

with p_a, atmospheric pressure. The permeability index also depends on experimental parameters such as: total pressurized volume V, pressure range, magnitude and distribution of moisture and temperature within cover, geometry of permeated concrete surface. If the experimental parameters are strictly defined, the test methods are suitable to differentiate clearly with respect to the quality of curing. The coefficient of variation of Figg's method was determined in laboratory tests in the following range for different batches of identical concrete: $V \approx 11$ % for oven-dried concrete, $V \approx 30$ % for concrete dried at 50 °C and lower.

Absorption tests: These tests serve the same purpose as permeability tests. Their results may also be used for durability prediction [6]. The ingress of water occurs by capillary suction. Transport of water by capillary tension is described by Bernoulli's law. Assuming a one-dimensional flow at the on-set of suction, the volume of water v_w per unit contact area can be expressed by (s. Fig. 3):



$$A_w = K_2 \sqrt{2 r_h} \frac{\varepsilon_{abs}}{a_{Tabs}} \; ; \quad K(T) = \sqrt{\sigma \cdot \cos \vartheta / 2\eta}$$

Fig. 3: Capillary water absorption

$$v_w = \frac{A_w}{\rho_w} \sqrt{t_s} \qquad (2)$$

with A_w, coefficient of water absorption; ρ_w, density of water and t_s, suction time. The coefficient A_w can be expressed by pore structural parameters [6]:

$$A_w = K_2 \sqrt{2r_h} \frac{\epsilon_{abs}}{a_{Tabs}} \qquad (3)$$

ϵ_{abs} ....effective capillary porosity, part of the total porosity within the range 100 nm $\leq r \leq$ 10 μm

$r_h = \epsilon_{abs}/S_{abs}$ ..... hydraulic radius; S_{abs} = specific surface

$a_T = h_{id}/h_m > 1$ .... tortuosity factor which relates the suction depth of the ideal porous body h_{id} to that of the real porous body h_m

K_2 .... physical coefficient related to surface tension, contact angle, viscosity and temperature of water

Although the effective capillary porosity and the hydraulic radius may be determined in the laboratory, for example by mercury intrusion, the tortuosity remains unknown. Thus, the coefficient A_w must be determined experimentally.

2.4 ISA-test

One method to measure the absorption of water on-site is the ISA-test (initial surface absorption), which is standardized in BS 1881, pt 5. Fig. 4 shows the test set-up. A cap is sealed onto the concrete surface and then filled with water with a small pressure head. The rate of water intake can be derived from the rate of the retracting meniscus of the scaled glass capillary, after closing the tap. The ISA-value is taken at certain time values t_S.

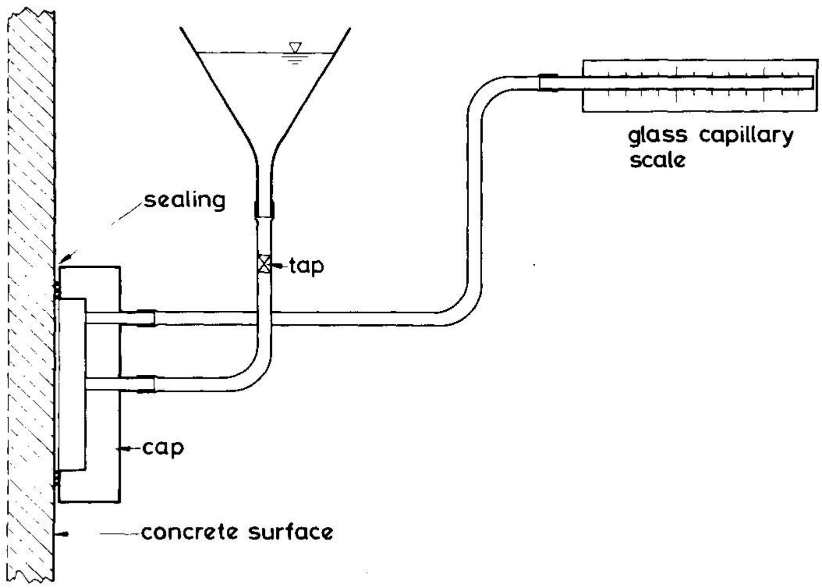


Fig. 4: Set-up of ISAT

The ISA-value is the non-steady, threedimensional flux of water. It can be approximatly expressed with Equ.(2) (see Fig. 5):

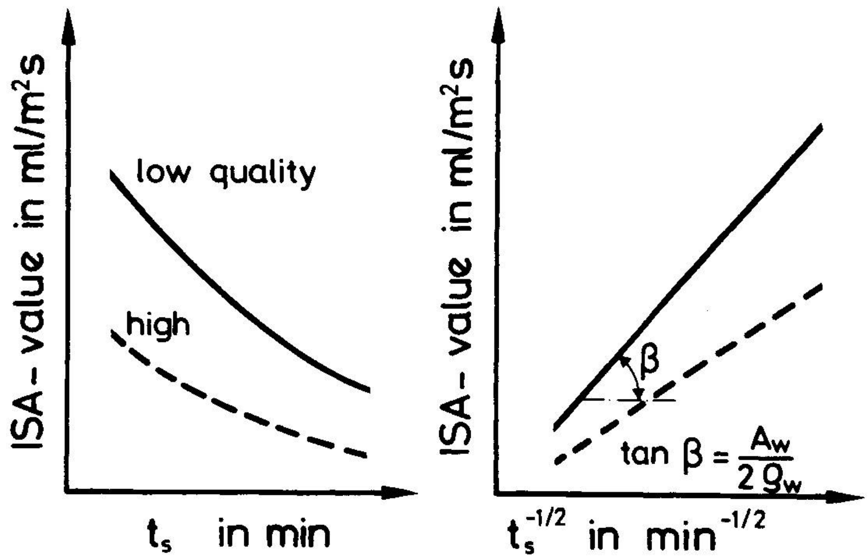


Fig. 5: Development of ISA-value vs. suction time

$$\dot{v}_W(t_S) = \frac{dv_W}{dt_S} = \frac{A_W}{2\rho_W}\, t_S^{-\frac{1}{2}} = ISA(t_S) \qquad (4)$$

The volume v_w corresponds to the effective capillary porosity ($r > 100$ nm) which can be filled by water, dependent on the momentaneous moisture content and

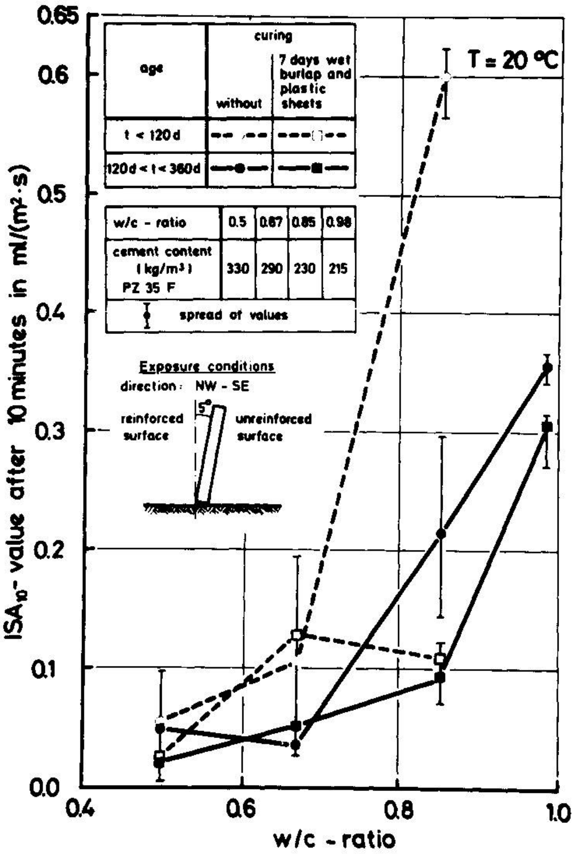


Fig. 6: Results of ISA_{10}-tests on the unreinforced surface of walls vs. w/c-ratio

temperature. Tests [6] proved that the ISA-reading is well correlated with pore structural properties as expressed by Equ.(3) and (4). The parameters water/cement ratio, curing and age can be satisfactorily identified and roughly quantified (see Fig. 6). Fig. 6 shows results for walls exposed unsheltered to weather (up to five readings per point in different locations). The coefficient of variation is about 33 %.

The moisture content of cover is of great influence [4]. Thus, ISA-tests should not be performed immediately after rainfall. A drying period of at least 2 days is necessary.

3. APPLICATION

The application of ISA-tests for the assessment of the quality of concrete and curing is shown in [4]. The application for prediction of durability is attempted in [7]. This requires the estimation of that portion of total porosity which is accessible for CO_2-diffusion. By insertion of the diffundable porosity into a carbonation law a model for the prediction of durability can be developed. The procedure is shown in [7].

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Untersuchungen an freibewitterten 20 Jahre alten Spannbetonträgern

Tests on Prestressed Girders after 20 Years of Weather Exposure

Essais de poutres précontraintes après 20 ans d'exposition aux intempéries

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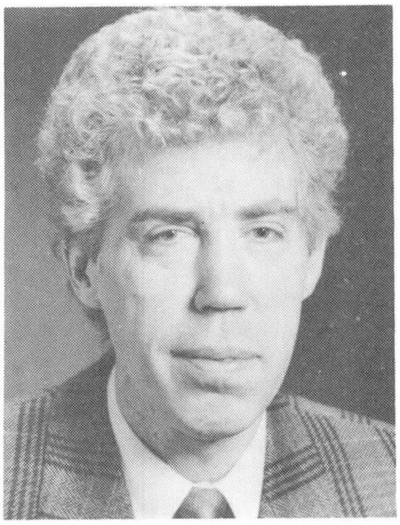

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ZUSAMMENFASSUNG

An vorgespannten Riegeln von Schilderbrücken der Berliner Stadtautobahn wurden umfangreiche Materialuntersuchungen durchgeführt und die noch vorhandene Tragfähigkeit ermittelt. Während der Belastungsversuche konnte mit Hilfe eines neu entwickelten zerstörungsfreien, dynamischen Prüfverfahrens, das ebenfalls zur Untersuchung von Bauwerken geeignet ist, die zunehmende Schädigung der Träger durch eine fortschreitende Rissbildung sicher nachgewiesen werden.

SUMMARY

The prestressed girders of three traffic sign bridges of the Berlin freeway were tested after 20 years of service. An extensive testing programme was initiated to determine material properties, the extent of corrosion of the reinforcement and the tendons, the penetration of chloride as well as the ultimate load of the girders. In order to detect possible alteration of structures and for future application a non-destructive test method was developed and used during loading tests to show the loss of stiffness due to cracking.

RÉSUMÉ

De nombreux essais sur des poutres en béton précontraint, de portiques de signalisation de l'autoroute urbaine de Berlin, ont été effectués en vue de déterminer leur stabilité résiduelle 20 ans après leur mise en service. Pendant les essais de chargement on a pu démontrer à l'aide d'une nouvelle méthode d'essais dynamique non destructive, pouvant être utilisée également pour les structures, une détérioration croissante des poutres due à la fissuration.

1. EINLEITUNG

Der Abbau einzelner Schilderbrücken des Berliner Stadtautobahnringes (Fig. 1) eröffnete die Möglichkeit einer eingehenden Untersuchung der in den Jahren 1962/63 in Spannbetonbauweise mit Stützweiten bis zu rd. 18,0 m hergestellten Brückenriegel [1]. In Anbetracht der in letzter Zeit vermehrt an Spannbetonkonstruktionen beobachteten Schäden bestand ein erhebliches Interesse, den baulichen Gesamtzustand dieser Tragglieder sowie die noch vorhandene Tragfähigkeit nach einer mehr als 20jährigen Nutzung und intensiven Beanspruchung durch eine freie Bewitterung und schädigende Umwelteinflüsse, wie Chloride, festzustellen.

Für die vorgesehenen Prüfungen standen zwei ausgebaute Brückenriegel sowie ein dritter zum gleichen Zeitpunkt hergestellter, jedoch seither in unmittelbarer Nähe der Stadtautobahn eingelagerter Riegel zur Verfügung. Die konstruktive Ausbildung der Bauteile ist aus Fig. 2 zu ersehen.

Fig. 1 Schilderbrücken der Berliner Stadtautobahn; vorn Neukonstruktion aus Stahl

2. ÄUSSERE BESCHAFFENHEIT UND AUSFÜHRUNG

Die zu untersuchenden Spannbetonträger befanden sich in einem guten äußeren Zustand. Auf Grund der jahrzehntelangen freien Bewitterung war es jedoch an den außenliegenden Oberflächen und den Stegunterseiten zu Auswaschungen gekommen, die zu einer rauhen Oberflächenstruktur geführt hatten. Im Gegensatz zu den Außenflächen wiesen die geschützter liegenden Innenseiten nahezu Sichtbetonqualität auf. Durch Korrosion der schlaffen Bewehrung verursachte Betonschäden hatten sich nur in geringem Umfang eingestellt. Lediglich an Stellen, an denen die Bügel beim Betonieren die Schalung punktuell berührt hatten, waren kleinere Absprengungen, in einem Fall ein größerer Riß, entstanden.

Die Ausführung der schlaffen Bewehrung entsprach weitgehend dem Entwurf. Als wesentlicher baulicher Mangel wurde bei allen drei Trägern eine stellenweise sehr geringe Überdeckung der Bügel bedingt durch fehlende Abstandshalter, festgestellt. Trotz der häufig vorgefundenen geringen Betondeckung und der an diesen Stellen ggf. vorhandenen hohen Chloridkonzentration ist bisher eine Korrosion der schlaffen Bewehrung weitgehend ausgeblieben. Grund hierfür dürfte die ausgezeichnete Qualität des eingebauten Betons sein, der hier als Korrosionsbremse fungiert.

Die Anordnung der Spannglieder in den Stegen entsprach ebenfalls den Planungsunterlagen. Bis auf ein Spannglied beim Träger A3 sind alle anderen voll verpreßt vorgefunden worden. Die Hüllrohre wiesen keine nennenswerten Korrosionseffekte auf. Gleiches gilt für die Spannstähle der Träger A1 und A2. Beim Träger A3 wurde am Spannstahl des verpreßten Spanngliedes eine leichte Oberflächenkorrosion festgestellt. Der Spannstahl im unverpreßten Hüllrohr war über die gesamte Länge gleichmäßig angerostet. Eine Querschnittsminderung konnte jedoch nicht festgestellt werden.

Trägerlänge: A1 = 1750; A2 = 1367; A3 = 1884 cm

Mittenquerschnitt

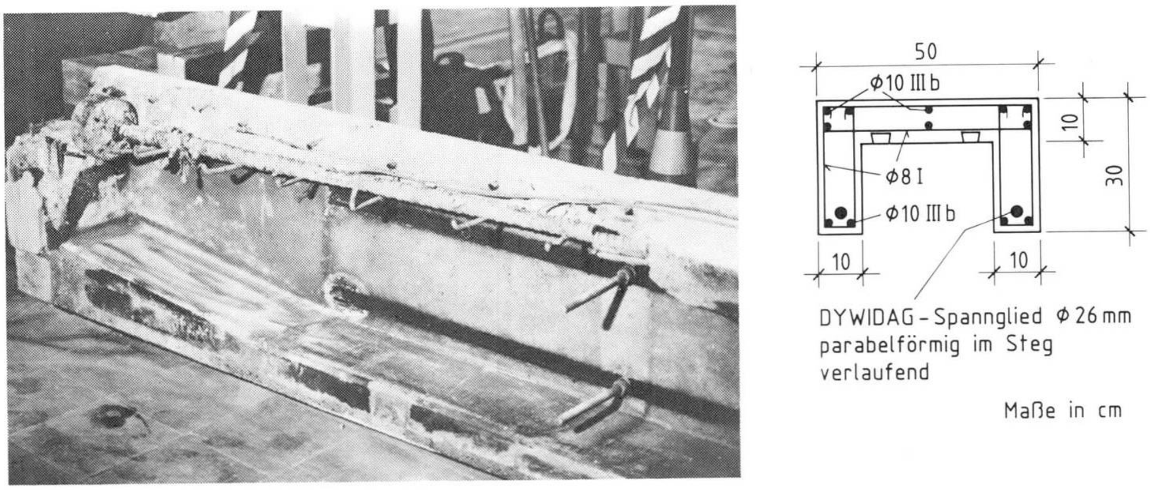


Fig. 2 Abmessungen und Bewehrung sowie Verankerung der Spannglieder

3. MATERIALUNTERSUCHUNGEN

Die an entnommenen Bohrkernen festgestellte Betondruckfestigkeit lag mit Mittelwerten zwischen $\overline{\beta}_W$ = 67,5 bis 75,5 N/mm² sehr hoch und übertraf damit den Sollwert von β_W = 45 N/mm². Der Beton war zudem gut verdichtet und wies eine gleichmäßige Kornstruktur auf. Die Karbonatisierung hielt sich in engen Grenzen und war mit 1-3 mm an den Außenseiten und bis zu 10 mm an den Innenseiten gering. Die chemische Analyse von Betonproben aus verschiedenen Trägerbereichen ergab für alle drei Probekörper eine vergleichbare Beanspruchung durch Chloride. In den bis zu einer Tiefe von 30 mm hin untersuchten Querschnitten erreichten die Chloridkonzentrationen mit 0,05 bis 0,95 % (in Einzelfällen bis 2,44 %), bezogen auf das Zementgewicht, relativ hohe Werte. Die Chloridkonzentration nahm in der Regel von außen nach innen hin ab. In einzelnen Bereichen war jedoch in tieferliegenden Schichten eine größere Anreicherung von Chloriden als an der jeweiligen Oberfläche festzustellen.

4. BELASTUNGSVERSUCHE UND ERMITTLUNG DER RESTVORSPANNUNG

Die Ermittlung der Tragfähigkeit der Spannbetonträger erfolgte durch Belastungsversuche (Fig. 3). Es zeigte sich, daß alle drei untersuchten Träger bei einer durch Einzellasten simulierten Gleichlast ein weitgehend ähnliches Trag- und Verformungsverhalten aufweisen. Der Beginn der Rißbildung erfolgte bei den Trägern A1 und A3 bei verhältnismäßig niedrigen Lasten, etwa bei $p = 0{,}5\ p_o$ bzw. $1{,}5\ p_o$ der mit $p_o = 0{,}66$ kN/m (A1 und A2) bzw. $p_o = 0{,}77$ kN/m (A3) angesetzten Gebrauchslast (Gewicht der Schilder plus Schneelast). Bei dem wesentlich kürzeren Träger A2 wurden erste Risse bei einer Last von ca. $p/p_o = 3{,}5$ beobachtet. Mit zunehmender Belastung stellten sich ganz erhebliche Durchbiegungen ein, die bei Versuchsende ca. 450 bis 650 mm betrugen. Erreicht wurde jedoch immer das in der statischen Berechnung ausgewiesene Bruchmoment. Das Versagen der Träger war gekennzeichnet durch eine zunehmende Durchbiegung (Fig. 4), ohne weitere Lastaufnahme infolge einer

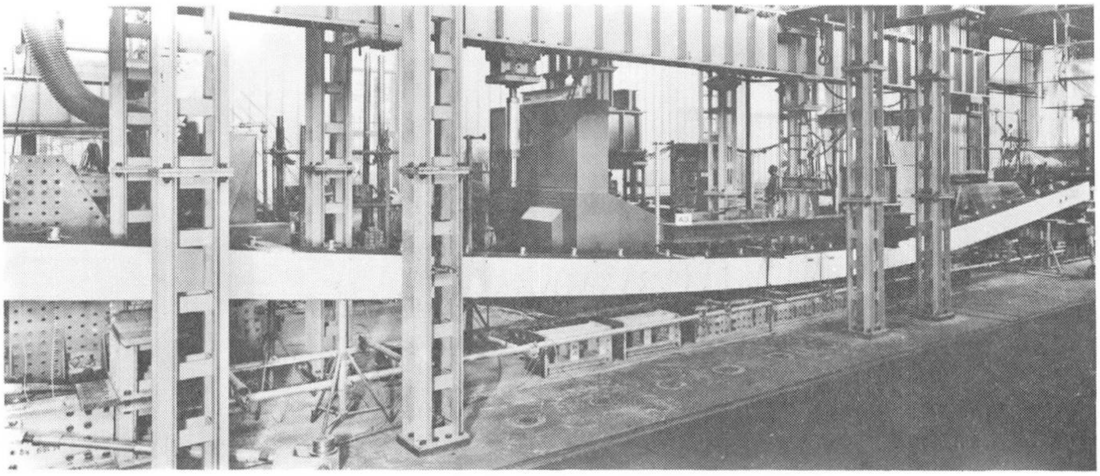


Fig. 3 Spannbetonträger A3 im Prüfstand mit einer Durchbiegung von w = 65 cm in der Mitte

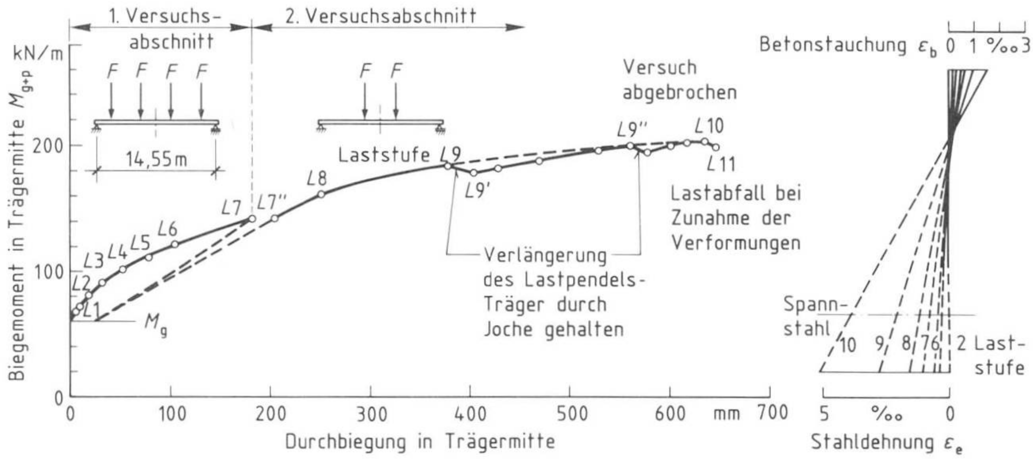


Fig. 4 Versuchsergebnisse für den Spannbetonträger A3 - Biegemoment - Durchbiegung (links) und Dehnungsverteilung über die Querschnittshöhe (rechts)

plastischen Dehnung des Spannstahls und zusätzlich beim Träger A2 durch eine Zerstörung der Druckzone. Ein Reißen der Spannstähle ist nicht eingetreten. Als Ursache für die niedrige Rißlast bei den Trägern A1 und A3 wurde die zum Zeitpunkt der Prüfung vorhandene geringe Vorspannung ermittelt. Durch Trennen der Spannstähle nach den Belastungsversuchen konnte aus der Rückdehnung die noch vorhandene Vorspannung rechnerisch ermittelt werden. Es ergab sich ein Spannkraftabfall gegenüber dem Sollwert der Vorspannung zwischen 23 % bei A2 - eingelagerter Träger - und 45 % bzw. 42 % bei A1 bzw. A3. Gründe für den eingetretenen großen Spannkraftverlust können nicht angegeben werden Ähnlich große Spannkraftverluste an Spannbetonträgern sind in der Literatur [2-4] aufgeführt.

5. ZERSTÖRUNGSFREIE ERFASSUNG DES SCHÄDIGUNGSZUSTANDES

Um zukünftig den baulichen Zustand einfacher vorgespannter Tragelemente leichter erfassen zu können, wurde der Träger A3 zusätzlich mit Hilfe eines zerstörungsfreien, auf Schwingungsmessungen basierenden Prüfungsverfahrens [5] untersucht (Fig. 5). Im vorliegenden Fall konnte die zunehmende Schädigung des Trägers und damit die Abnahme der Tragfähigkeit gut aus den gemessenen Antwortspektren abgelesen werden (Fig. 6). Berechnet man aus den Eigenfrequenzveränderungen die Steifigkeitsreduzierung des Gesamtsystems, so ergibt sich in Übereinstimmung mit den Ergebnissen des statischen Versuchs nach dem 1. Versuchsabschnitt ein Wert von ca. 11 % und nach dem 2. Versuchsabschnitt ein Wert von ca. 35-40 %.

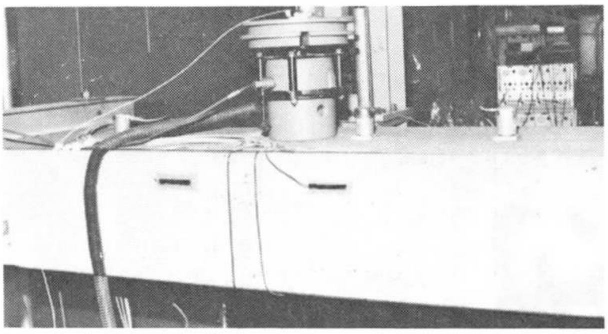
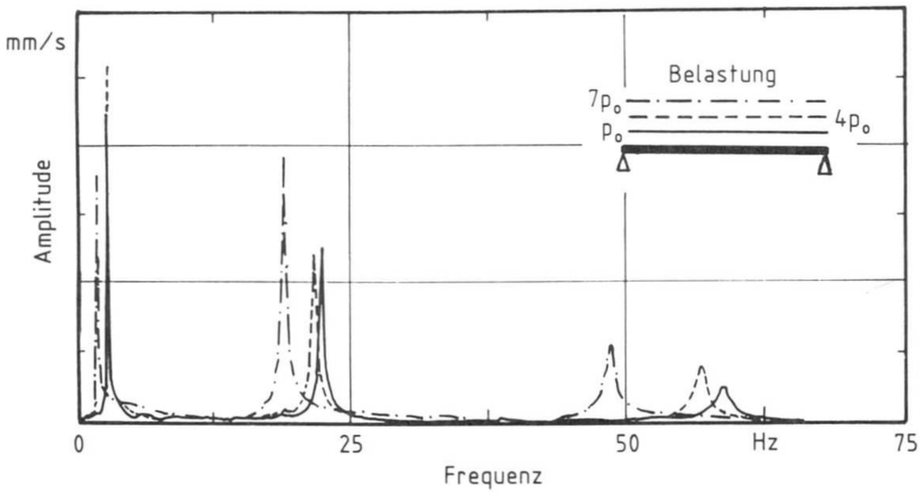


Fig. 5 Elektrodynamischer Schwinger zur Bauteilerregung und Geophone zur Aufnahme der Schwinggeschwindigkeit bzw. -beschleunigung

Fig. 6 Veränderung des Schwinggeschwindigkeits-Antwortspektrums in Abhängigkeit vom Grad der Schädigung



6. SCHLUSSBETRACHTUNG

Zusammenfassend läßt sich feststellen, daß trotz einiger Mängel bei der Herstellung, wie zu geringe Betondeckung und nicht verpreßte Spannglieder, die Tragfähigkeit der Spannbetonträger auf Grund der guten Betonqualität und der Unempfindlichkeit des Spannstahls SIGMA St 80/105 gegenüber einem Korrosionsangriff auch nach einer mehr als 20jährigen Nutzung noch uneingeschränkt gegeben ist. Die Beaufschlagung durch Chloride hatte bisher nur zu geringen Schäden geführt. Der in den Trägern aufgetretene Verlust an Vorspannung ist ebenfalls ohne Bedeutung für die Tragfähigkeit, da die Bemessungsmomente im Versuch immer erreicht wurden Die vorhandene geringe Vorspannung führt gegebenenfalls frühzeitig zu Rissen, die jedoch auch bei Überschreiten der Gebrauchslast um ein Mehrfaches lediglich Rißweiten von ca. 0,1 - 0,2 mm aufweisen und sich damit auf die Dauerhaftigkeit der Träger nur wenig auswirken dürften. Langfristig gesehen ist die Dauerhaftigkeit der Spannbetonträger jedoch durch die z. T. geringe Überdeckung der Bügel gefährdet. Bei einer Konservierung des jetzigen Zustandes und Ausbesserung stärker betroffener Stellen kann den untersuchten Bauteilen jedoch eine hohe Lebensdauer zugeschrieben werden. Die Belastungsversuche haben außerdem gezeigt, daß ein Versagen offenbar nie schlagartig eintritt, sondern sich bei einer Schädigung der Betondruckzone oder bei einem Querschnittsverlust infolge zunehmender Korrosion der Spannstähle durch zunehmend größere Verformungen ankündigt und daher frühzeitig erkannt werden kann.

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Abschließend möchte der Autor dem Senator für Bau- und Wohnungswesen der Stadt Berlin für seine Unterstützung und Förderung des Projekts sowie den Herren G. Kretschmann, W. Theisel und B. Stoeck für die sorgfältige Vorbereitung und Durchführung der Versuche herzlichen Dank sagen.

Radioscopie des ouvrages en béton précontraint

Radioskopie an Spannbetonbrücken

Radioscopy of Prestressed Concrete Bridges

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RÉSUMÉ

La qualité d'injection des gaines de précontrainte, dont dépend la durabilité des structures vis-à-vis de la corrosion des câbles, contrôlée en France depuis 1968 à l'aide de sources radioactives (gammagraphie) est contrôlée depuis 1985 par le LRPC de Blois par radioscopie grâce à un accélérateur linéaire miniaturisé.

ZUSAMMENFASSUNG

Die Qualität der Injektion von Spannkabel, von denen der Widerstand der Kabel gegen Korrosion abhängig ist, wird in Frankreich seit 1968 durch Gammaröntgen mit radioaktiven Quellen und seit 1985 durch Radioskopie bei den LRPC von Blois mit einem kleinem Linearbeschleuniger überprüft.

SUMMARY

The quality of the grouting in prestressed cable ducts, on which the durability of the structures against the corrosion of the cables depends, has been verified in France since 1968 by gammagraphy with radioactive sources and since 1985 by radioscopy by the LRPC of Blois with a miniaturized linear accelerator.

1. INTRODUCTION

La corrosion des armatures est un phénomène redouté pour les structures en béton armé et en béton précontraint. En béton armé, la protection des armatures est assurée naturellement par le béton pourvu que l'on respecte certaines règles d'usage. En béton précontraint, le plus souvent, la protection des câbles est assurée par injection d'un coulis de ciment dans les conduits ; ainsi la durabilité de la structure est-elle fort dépendante de la qualité de l'injection.

La gammagraphie classique du béton a trouvé son principal développement, ces vingt dernières années, dans le contrôle des injections, que ce soit sur ouvrage en cours de construction, ou sur ouvrage en service.

Les limites de cette technique résident dans le fait que l'information est ponctuelle (clichés de 30 x 40 cm), les investigations sont assez lentes, les épaisseurs de béton auscultables limitées à 60 cm.

Le système SCORPION est l'aboutissement d'une recherche destinée à remédier à ces inconvénients. Conçu, construit et mis au point par le Centre d'Etudes Techniques de l'Equipement Normandie-Centre en collaboration avec le service physique du Laboratoire Central des Ponts et Chaussées et la Compagnie Générale de Radiologie, il est utilisé depuis 1985 sur tout le territoire français par le L.R.P.C. de Blois.

Ce système utilise un mini accélérateur linéaire de 4 MeV : il permet la radiographie d'ouvrages en béton jusqu'à un mètre d'épaisseur et surtout leur radioscopie pour des parois d'épaisseur inférieure ou égale à 60 centimètres. C'est cette technique que nous décrivons principalement ici car elle présente de nombreux avantages en particulier pour l'auscultation de ponts à poutres.

2. LA RADIOSCOPIE

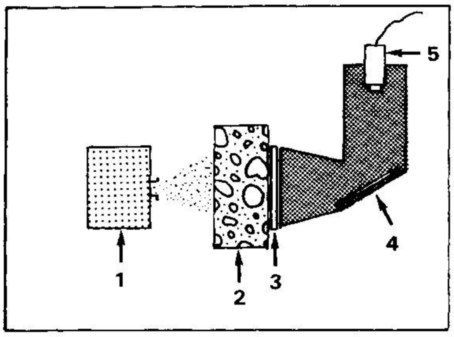
2.1 Principe

Le principe de la Radioscopie sur béton est le même que celui servant aux techniques médicales. Le rayonnement de photons, émis par le générateur X est atténué de façon sélective par les matériaux traversés. Le rayonnement émergeant est transformé en lumière visible par un convertisseur flurométallique de composition optimisée pour la gamme d'énergie utilisée.

L'image ainsi formée est reprise en temps réel à l'aide d'une caméra à très bas niveau de lumière et transmise à un système vidéo permettant aussi bien l'observation, que l'enregistrement et éventuellement le traitement de l'image. Les images intéressantes peuvent également être reproduites sur papier photographique à partir du signal vidéo.

2.2 L'apport de la radioscopie

Sur un ouvrage en béton précontraint en cours de construction, un contrôle partiel bien mené de la bonne qualité des injections des conduits de précontrainte suffit : la gammagraphie est bien adaptée à cette fin. Utilisée très largement en France ces vingt dernières années elle a non seulement contribué à assurer un bon contrôle de qualité des injections mais elle a en même temps permis d'améliorer la méthodologie et la technologie de celle-ci. Dans ce type d'application, la limite de la méthode réside dans les épaisseurs de béton à traverser ; au-delà de 60 cm les temps de pause et la définition de l'image notamment font que les applications ne peuvent plus être considérées comme "opérationnelles".



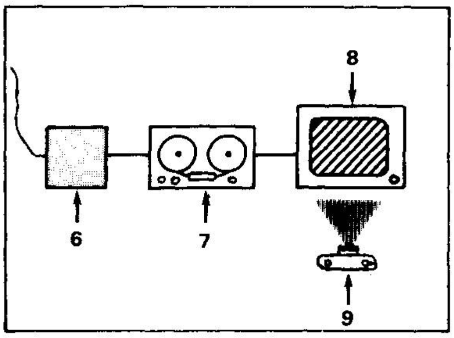


Fig. 1 - Schéma de principe de la chaîne de radioscopie

1. Accélérateur	2. Paroi de béton	3. Convertisseur
4. Miroir	5. Caméra	6. Coffret mémoire
7. Magnétoscope	8. Moniteur vidéo	9. Reproducteur d'image

L'utilisation d'un mini accélérateur linéaire de 4 MeV en remplacement des sources de Co 60, apporte alors plusieurs avantages :

- une meilleure qualité d'image que celle obtenue avec une source de Co 60 du fait de la taille du foyer plus petit : Ø 1,7 mm au lieu de 6 x 7 mm.
- une pénétration beaucoup plus grande du rayonnement, permettant la radioscopie jusqu'à 60 cm d'épaisseur de béton.
- une constance dans le temps des caractéristiques radiologiques, la source ayant toujours la même énergie et le même débit de dose, contrairement aux sources radioactives dont l'activité diminue très rapidement.
- une plus grande sécurité du point de vue radioprotection, le rayonnement étant immédiatement arrêté quand on coupe l'alimentation électrique.
- une sécurité totale pendant les transports par route : absence totale d'activité.
- une autonomie d'intervention apportée par la passerelle de manipulation.

Une autre utilisation de la gammagraphie est la recherche de défauts sur ouvrages en service (voire sur ouvrages terminés, avant leur mise en service). Alors le sondage statistique peut s'avérer insuffisant. C'est le cas lorsque l'on détecte effectivement des défauts et que l'on veut alors en connaître l'étendue (ou la répétition) : c'est ce que permet la radioscopie. Suivront, le plus souvent des investigations complémentaires : examens endoscopiques, ouvertures de fenêtres, analyses sur prélèvements ... qui devront permettre de formuler un diagnostic global sur l'état de santé de l'ouvrage. C'est sur cette base que pourra être établi, avec le minimum d'incertitude, un projet de réparation.

Le défaut le plus important recherché est l'absence totale ou partielle de coulis d'injection, mais l'on pourra également voir la présence d'armatures rompues ou détendues, des hétérogénéités de béton, des fissures, des ferraillages incorrects, des défauts de joints etc.

La visualisation immédiate permet à l'opérateur de fixer l'exploration sur un point particulier, de revenir en arrière, d'examiner l'environnement immédiat d'une zone douteuse et ainsi de "fouiller" sur un certain espace pour cerner l'importance d'une anomalie, alors que la gammagraphie classique procède par dépouillement à posteriori d'un cliché dont l'emplacement a été déterminé à l'avance.

Bien que la radioscopie ne permette pas une visualisation complète de l'ouvrage (il reste des zones d'ombre comme on le verra par la suite) la zone explorée qui est considérablement augmentée par rapport à la gammagraphie, constitue un apport évident ; la possibilité laissée à l'opérateur de guider son exploration peut permettre une détection plus sûre d'un défaut incertain ou mal défini.

3. DESCRIPTION DU SYSTEME SCORPION

"SCORPION" est donc constitué de trois éléments principaux :

3.1 Un accélérateur linéaire miniaturisé de 4 Mega électronvolts conçu par la Société CGR-MEV pour être mis en oeuvre dans des conditions de chantier. L'intensité du rayonnement X émis est règlable de 0,7 à 4 Grays par minute : elle peut ainsi être adaptée d'une part aux épaisseurs de paroi en béton à ausculter et d'autre part aux problèmes de radioprotection pouvant être rencontrés sur ouvrage. On choisit évidemment l'intensité minimale nécessaire pour l'auscultation : ainsi 0,7 Grays par minute permet la radioscopie de 30 centimètres de béton.

3.2 Le détecteur, convertisseur, caméra et chaîne vidéo a été conçu et breveté par les Laboratoires des Ponts et Chaussées. La taille du convertisseur permet d'obtenir des images de 30 x 40 cm, équivalentes en dimension, à celles obtenues avec les films classiques utilisés en gammagraphie.

L'épaisseur maximale de béton auscultable est de 60 cm.

La sensibilité de détection exprimée en rapport de taille du défaut minimum à l'épaisseur de béton traversée est de 0,7 % pour l'acier et de 1,5 % pour les vides.

Les images obtenues sont transmises à un camion-laboratoire équipé d'un moniteur permettant l'examen en temps réel, d'un enregistreur magnétique et d'un système de reproduction sur papier des clichés intéressants.

3.3 Ces deux éléments, accélérateur et détecteur sont mis en oeuvre sur ouvrages à l'aide d'un manipulateur conçu spécialement sous forme d'une passerelle qui peut être aussi utilisée pour la visite de l'ouvrage. En raison des problèmes de radioprotection, ce manipulateur doit être entièrement télécommandé à partir du camion-laboratoire placé à environ 80 mètres de la zone auscultée.

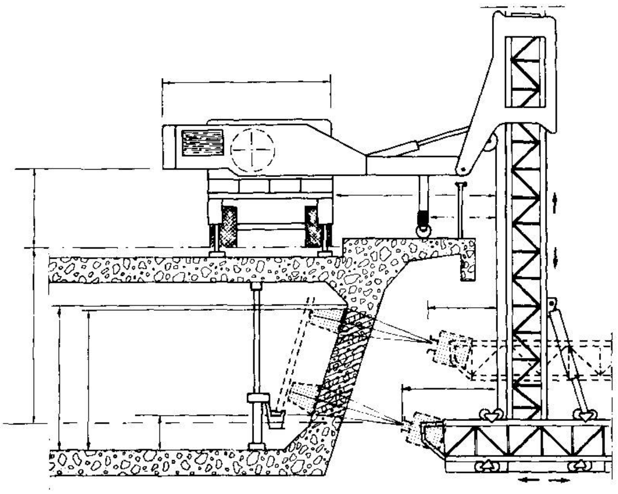
Etant donnéela diversité des géométries des ouvrages, la passerelle doit être adaptée au type de pont.

4. POSSIBILITES D'INTERVENTIONS DANS LA CONFIGURATION ACTUELLE DU SYSTEME

Dans sa première version, Scorpion a été conçu pour intervenir sur des tabliers à "poutre caisson" (ponts construits par encorbellement, ponts poussés). Une modification de cette version permet d'opérer sur des tabliers à poutres sous chaussées.

Sont montrées, ci-après les positions de travail et les surfaces explorables offertes dans l'un et l'autre cas.

4.1 Poutre caisson



La figure 2 montre la disposition de l'émetteur et du récepteur dont les déplacements sont complètement indépendants. La synchronisation des mouvements est obtenue au niveau des commandes.

La figure 3 montre la zone explorable d'une âme réalisable en une journée (pose du dispositif récepteur comprise).

Fig. 2 : radioscopie d'une âme de voussoir.

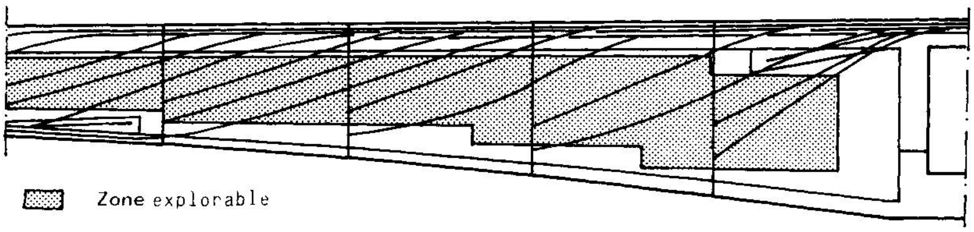
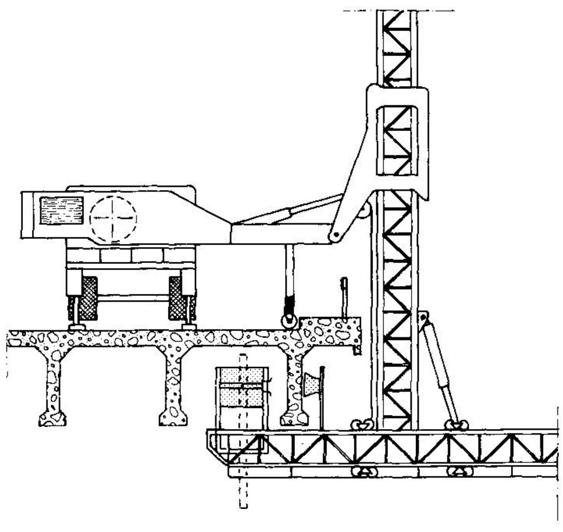


Fig. 3 : Zone explorable sur 5 voussoirs de 3 mètres, soit 30 mètres de câbles

4.2 Poutres sous chaussée



Sur la figure 4 on voit qu'émetteur et récepteur sont portés ensemble par la passerelle. La synchronisation des mouvements en est ainsi simplifiée par rapport à la version précédente. La mise en place est sensiblement plus rapide.

La figure 5 montre les zones explorables, les talons de poutres sont également radiographiables mais l'interprétation des images est difficile compte tenu du nombre de câbles interceptés qui se trouvent alors "en paquet".

Fig. 4 : radioscopie d'une âme de poutre

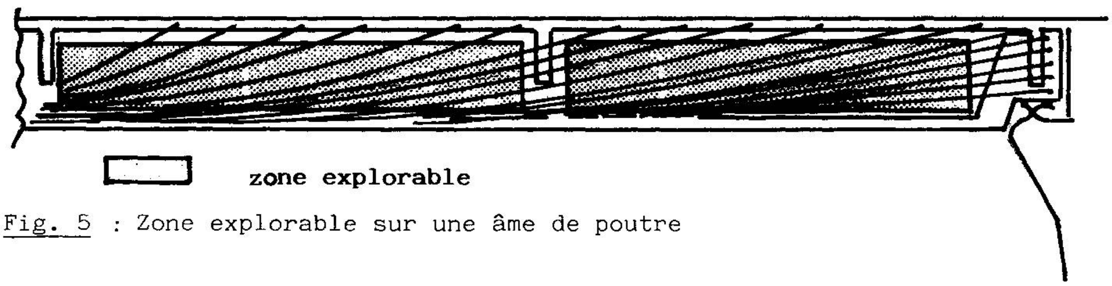


Fig. 5 : Zone explorable sur une âme de poutre

5. CONCLUSION

L'utilisation opérationnelle de Scorpion n'en est qu'à ses débuts, il est alors difficile de situer sa place parmi les moyens d'auscultation d'ouvrages.

En concurrence avec la gammagraphie, son intérêt économique est certain pour les interventions sur ouvrages importants (nécessitant la semaine de travail).

Son avenir réside plutôt dans la défectoscopie, c'est-à-dire la recherche de défauts sur ouvrages en service, où la richesse de l'information recueillie en fait un outil d'auscultation jusqu'à maintenant inégalé. Un ouvrage condamné qu'il faut reconstruire, une grosse réparation, coûtent cher. La prévention ou l'intervention à temps peut permettre d'éviter ces traitements coûteux. Actuellement, un contrat de collaboration établi entre les principales sociétés d'autoroutes françaises et italiennes prévoit 25 semaines d'utilisation par an pendant 4 ans.

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