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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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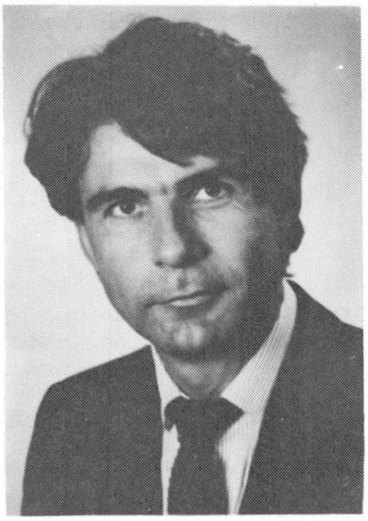

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SUMMARY

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.

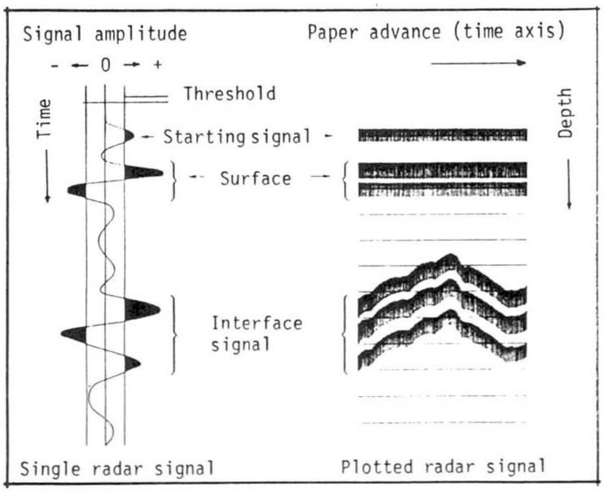


Fig.1 Graphic recording of a radar signal.

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.

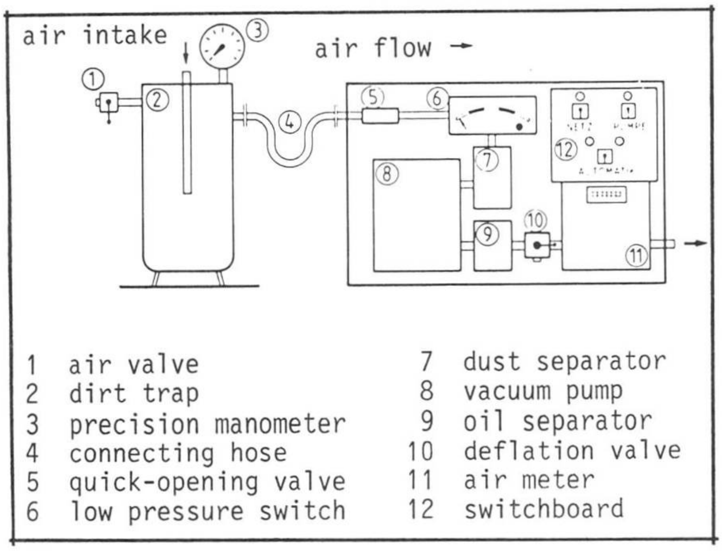
After opening of the sheathing, first the found situation as to voids must be described. Subsequently the bores are closed with injection hoses and sealing caps as protection against humidity.

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.

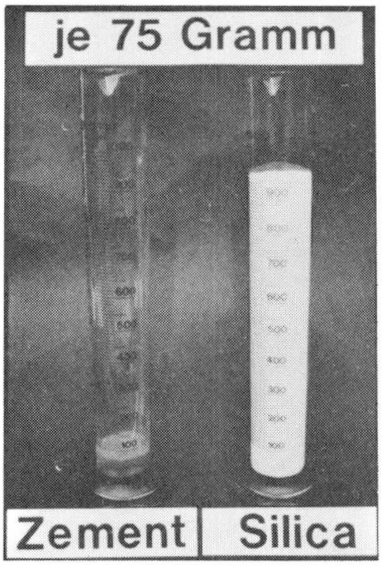


Fig.7 Comparison of volume

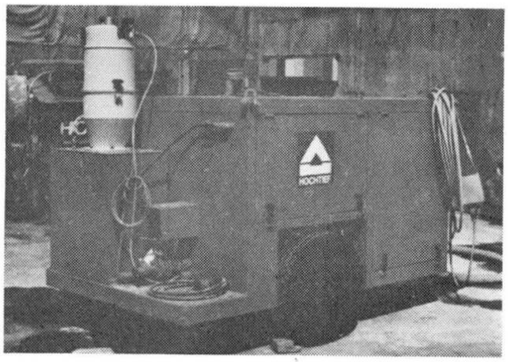


Fig.8 Batching device for loading the compressed air flow with synthetic silicic acid of 200 m²/g fineness.

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.

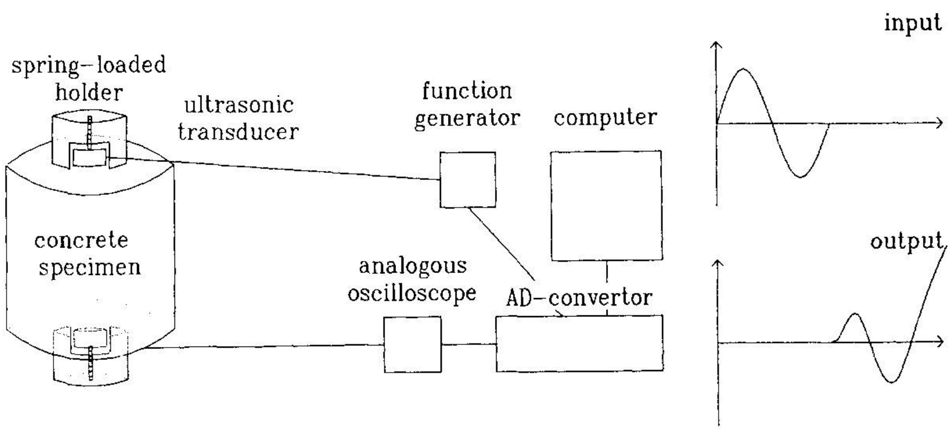

Fig. 1: Schematic diagram of test setup

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.

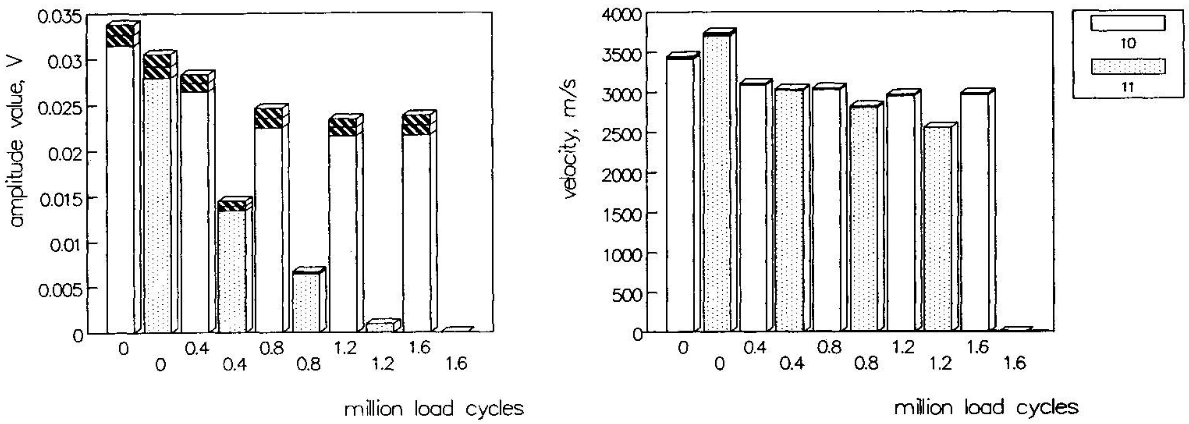

Fig. 2: Mean values with standard deviations of eight repeated measurements from specimens 10 and 11, at different stages of loading. Amplitude of first received sound wave, left and Velocity, right.

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.

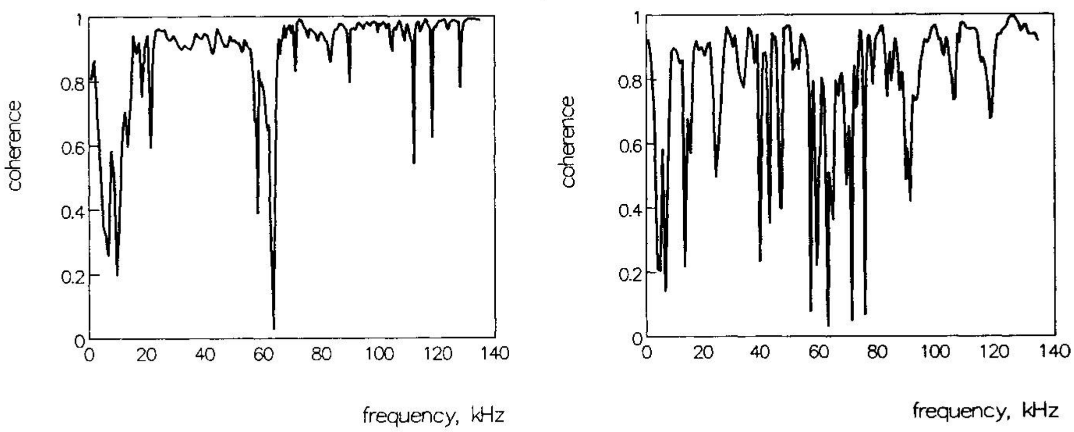

Fig. 3: Coherence functions for measurements on specimen number 11 at the initial stage, left, and after 0.8 million load cycles, right.

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.

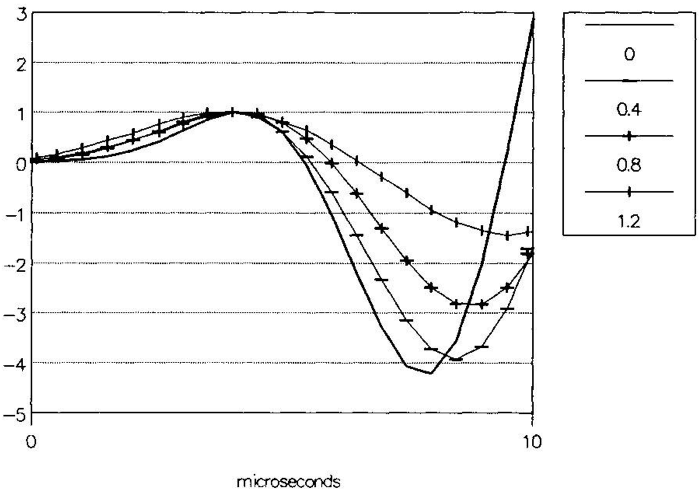

Fig. 4: First parts of received waveforms from specimen 11 at 0, 0.4, 0.8 and 1.2 million cycles. Amplitude values are normalized with respect to the first received peak.

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

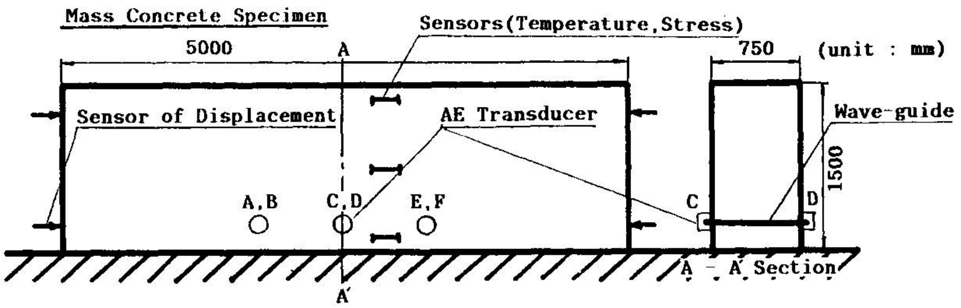

Fig. 1 Mass Concrete Specimen and Locations of Sensors

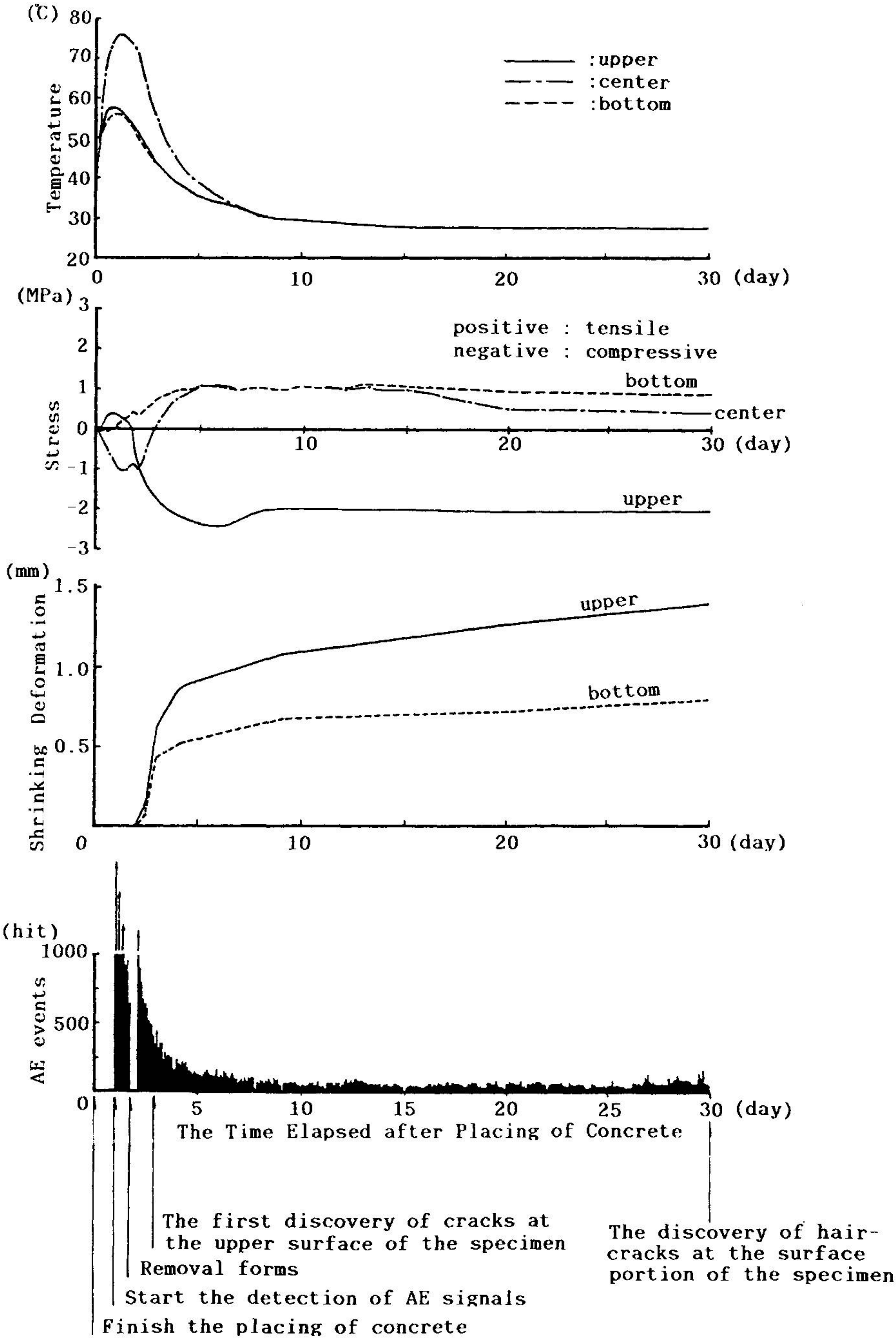

Fig. 3 The Time Elapsed versus Temperature, Longitudinal Stress, Longitudinal Deformation and AE Events

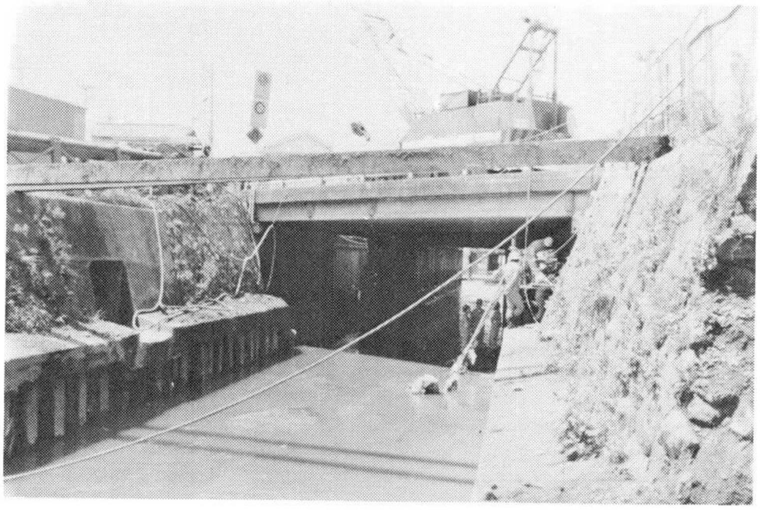

Photo. 1 Scene of the Shinomiyajyuku Bridge

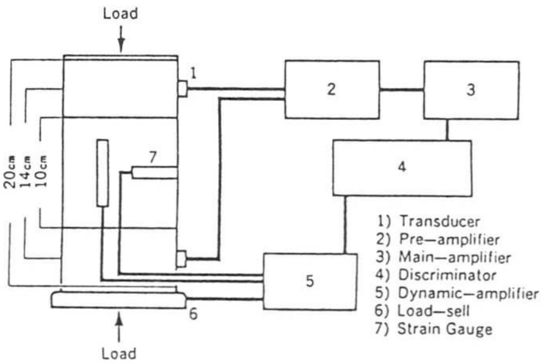

Fig. 5 Uniaxial Compressive Loading System and AE Measurement System

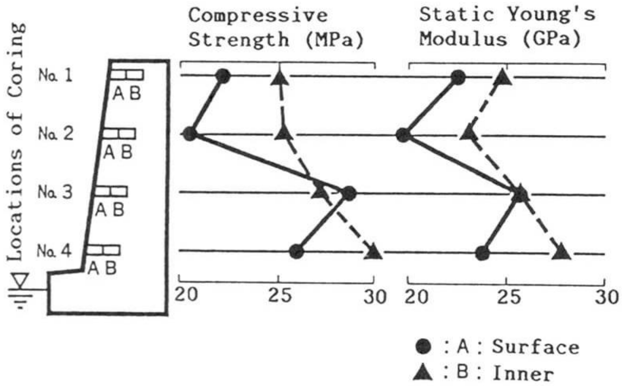

Fig. 4 Compressive Strength and Static Young's Modulus of The Core Specimens

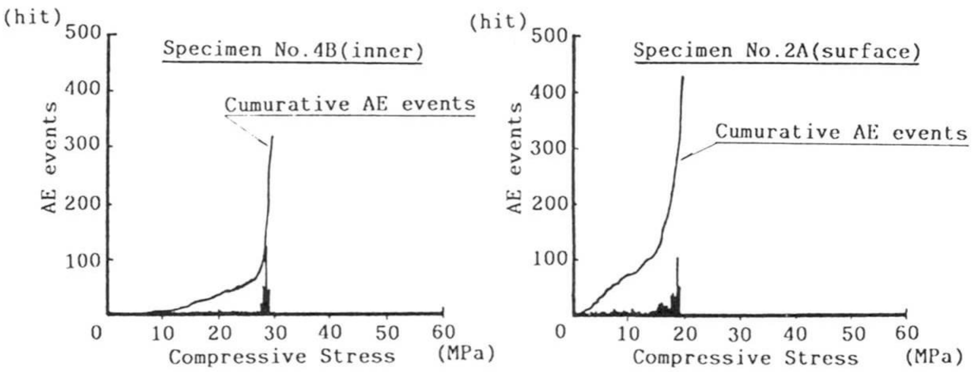

Fig. 6 Relationship between Compressive Stress and AE Events during Uniaxial Compressive Testing

Magnetoelastische Kraftmessung im Spannbeton

Magnetoelastic Force Measurement in Prestressed Concrete

Mesure magnéto-élastique de la force dans le béton précontraint

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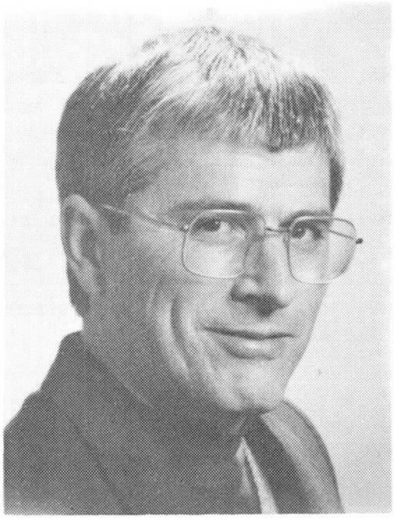

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ZUSAMMENFASSUNG

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).

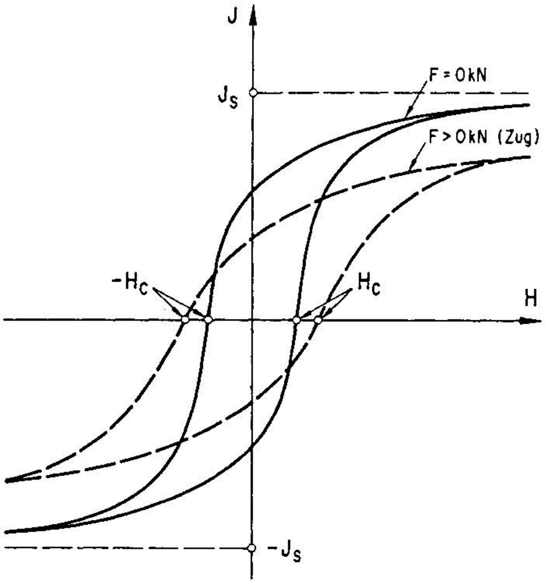

Fig.1 Magnetoelastic effect on prestressing steel

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.

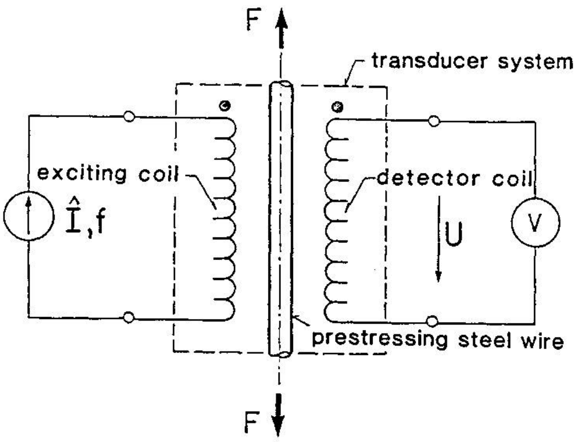

Fig.2 Magnetoelastic F-measurement

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:

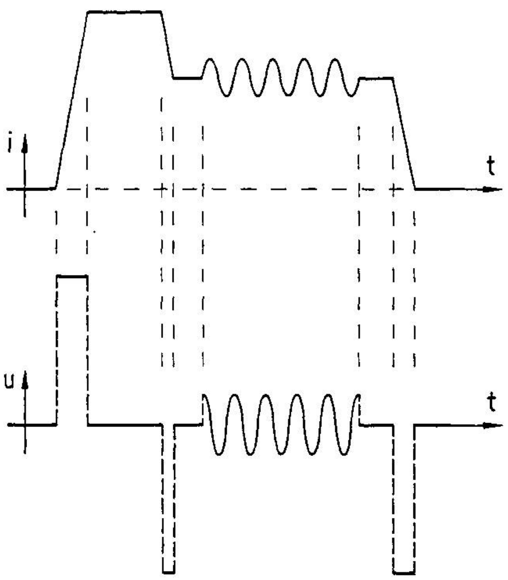

Fig.3 Excitation- and detectionsignal

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 ± .02)mm.

Supplier:	Color code	Min. value N/mm^2	Mean value N/mm^2
VOGT	RED	1670	1777±15
BEKAERT	YELL	1670	1757±17
F + G	BLUE	1670	1732±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 ± 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} 1	H_c A/cm	B_r T	J_s 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].

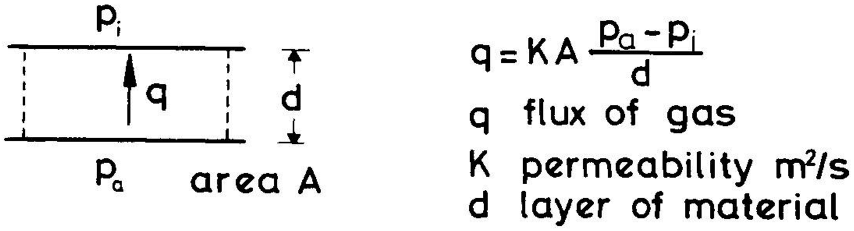


Fig. 1: Hagen-Poiseuille's Law

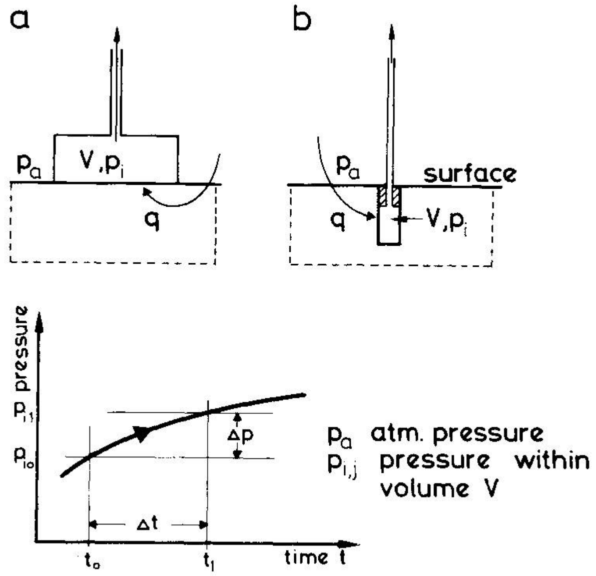
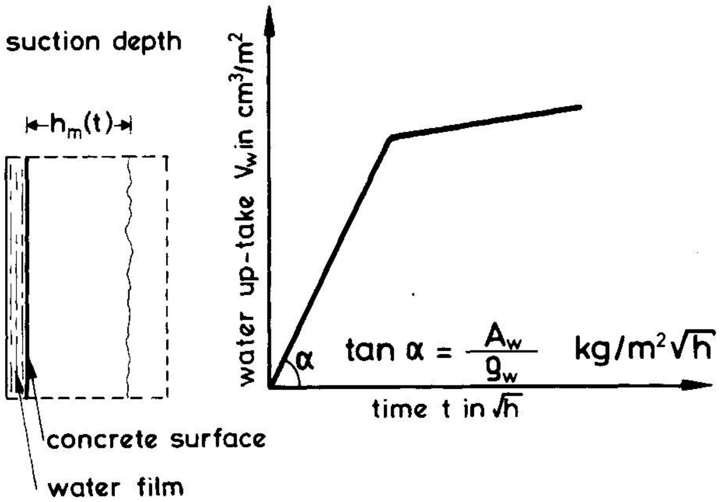


Fig. 2: Permeability test of Schönlin-Hilsdorf (a) and Figg (b)

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

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

$$v_w = \frac{A_w}{\rho_w}\sqrt{t_s} \tag{2}$$

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]:

$$A_w = K_2 \sqrt{2r_h}\,\frac{\epsilon_{abs}}{a_{Tabs}} \tag{3}$$

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.

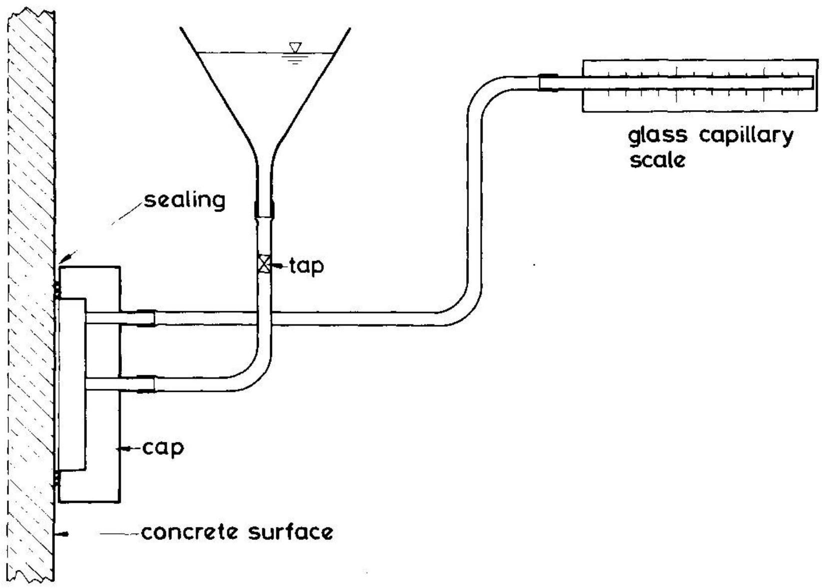


Fig. 4: Set-up of ISAT

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

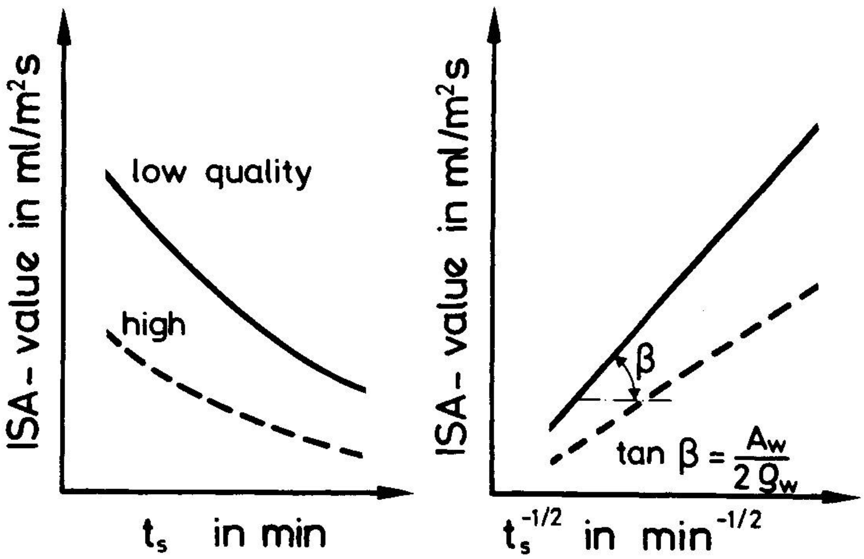


Fig. 5: Development of ISA-value vs. suction time

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

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

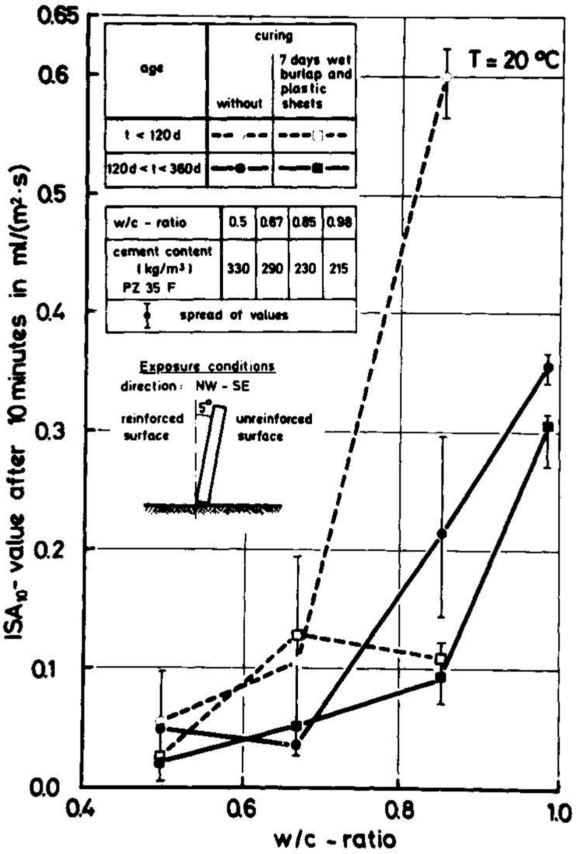


Fig. 6: Results of ISA_{10}-tests on the unreinforced surface of walls vs. w/c-ratio

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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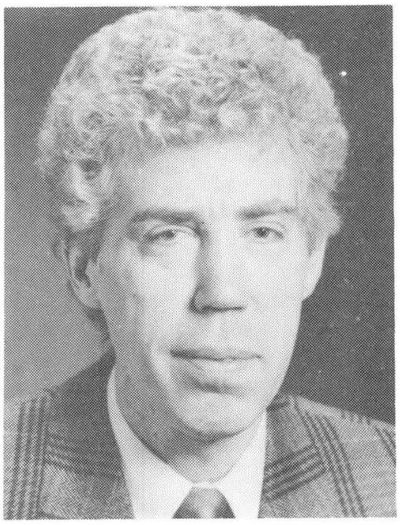

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ZUSAMMENFASSUNG

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

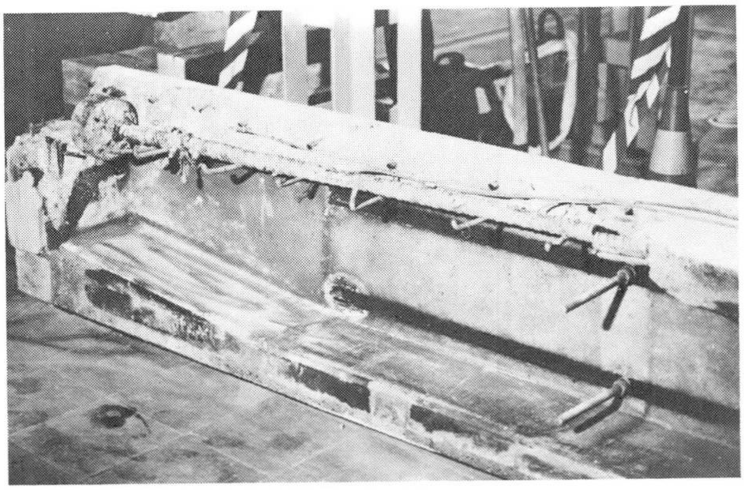
2. ÄUSSERE BESCHAFFENHEIT UND AUSFÜHRUNG

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

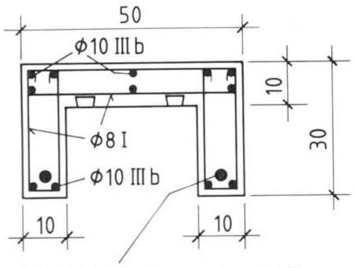


Fig. 2 Abmessungen und Bewehrung sowie Verankerung der Spannglieder

3. MATERIALUNTERSUCHUNGEN

Die an entnommenen Bohrkernen festgestellte Betondruckfestigkeit lag mit Mittelwerten zwischen $\overline{\beta}_W = 67{,}5$ bis $75{,}5\ N/mm^2$ sehr hoch und übertraf damit den Sollwert von $\beta_W = 45\ N/mm^2$. 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

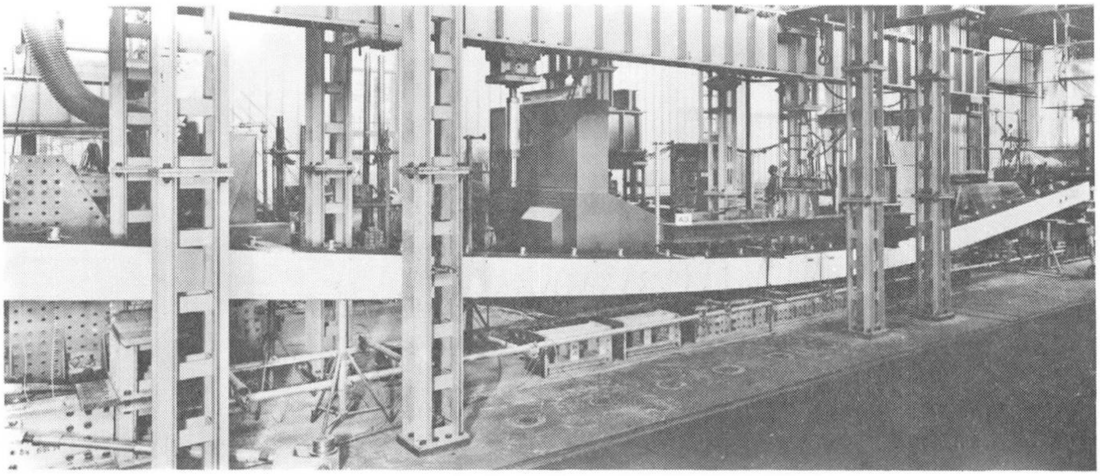


Fig. 3 Spannbetonträger A3 im Prüfstand mit einer Durchbiegung von w = 65 cm in der Mitte

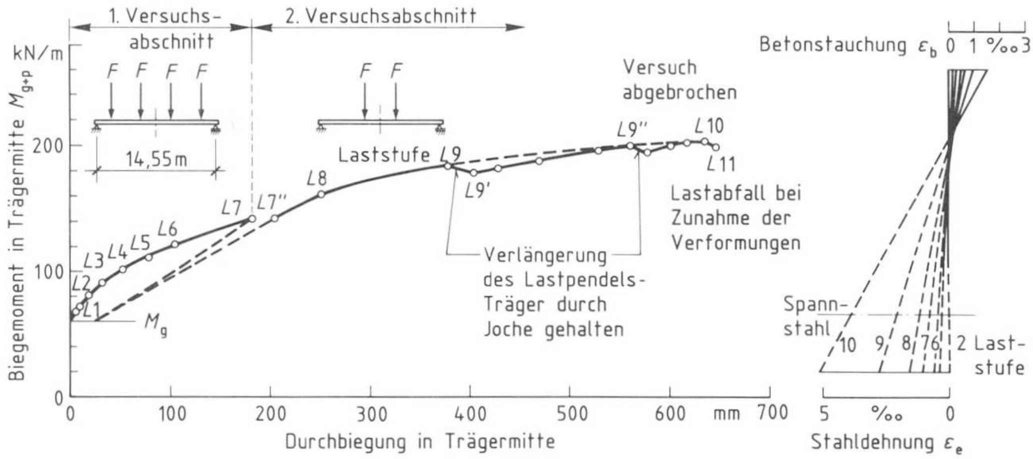


Fig. 4 Versuchsergebnisse für den Spannbetonträger A3 - Biegemoment - Durchbiegung (links) und Dehnungsverteilung über die Querschnittshöhe (rechts)

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 %.

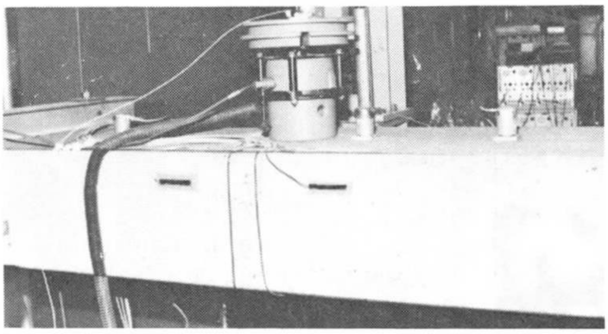
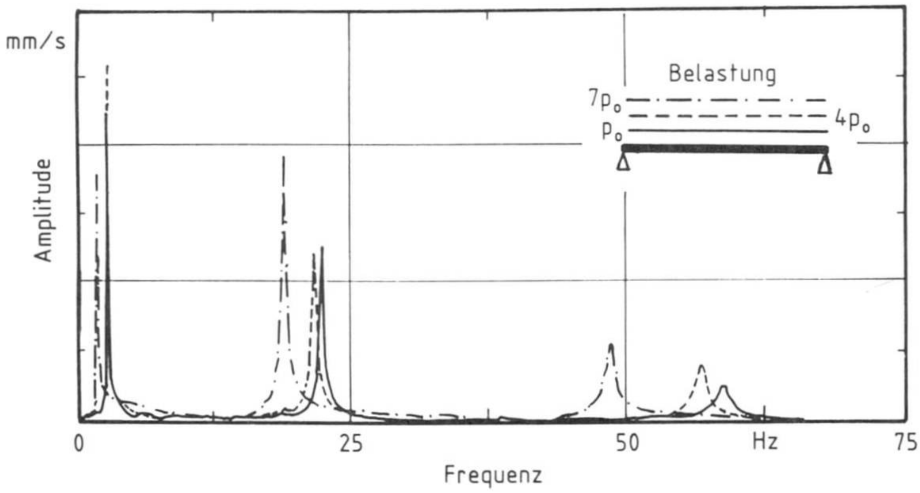


Fig. 5 Elektrodynamischer Schwinger zur Bauteilerregung und Geophone zur Aufnahme der Schwinggeschwindigkeit bzw. -beschleunigung

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".

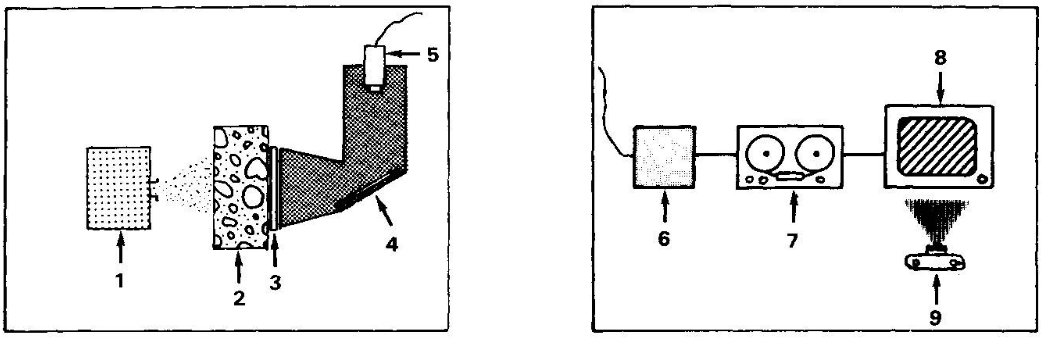


Fig. 1 - Schéma de principe de la chaîne de radioscopie

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.

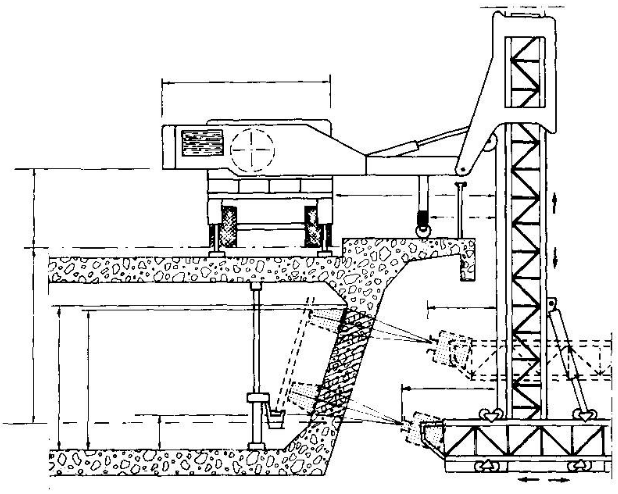
Etant donnéela diversité des géométries des ouvrages, la passerelle doit être adaptée au type de pont.

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.

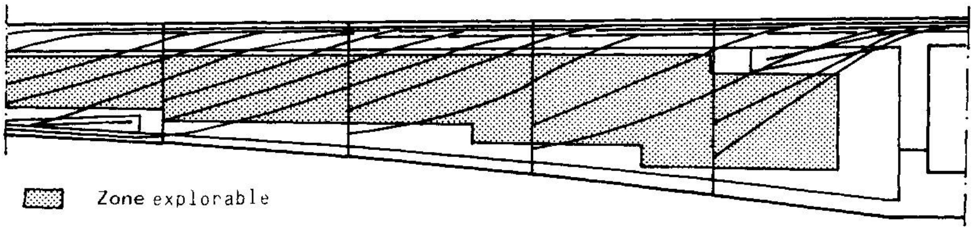
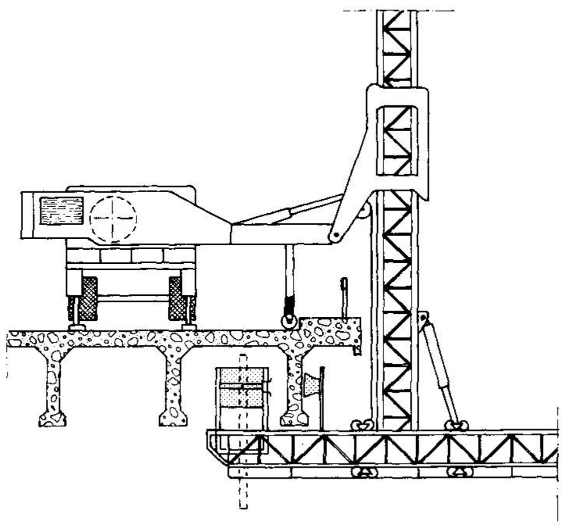


Fig. 3 : Zone explorable sur 5 voussoirs de 3 mètres, soit 30 mètres de câbles

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

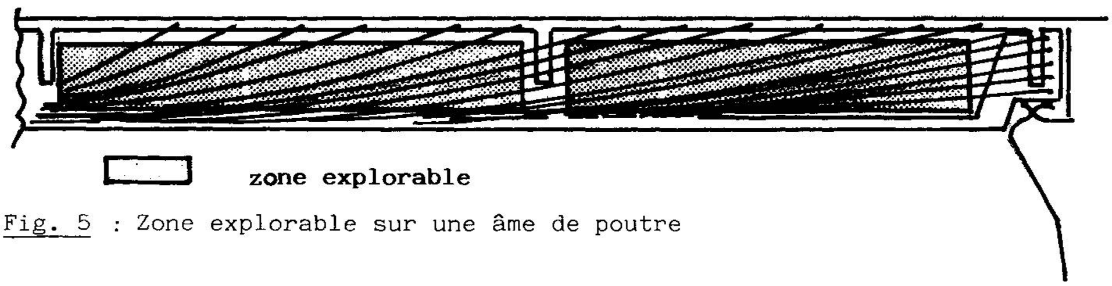


Fig. 5 : Zone explorable sur une âme de poutre

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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Life Expectancy Studies of Reinforced Concrete Using Microcomputer

Étude de la durée de vie du béton armé par simulation

Studie zur Lebenserwartung von Stahlbeton mit Hilfe von Simulationen

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SUMMARY

The paper describes how a systematic stochastic life-expectancy and financial analysis can be performed by exploiting the combination of microcomputers and Monte Carlo simulation. The use of the program "Venturer" developed by the author, is illustrated with an example of a footbridge.

RÉSUMÉ

L'article décrit comment les analyses systématiques et aléatoires, de durée de vie et de risque financier, peuvent s'effectuer à l'aide de micro-ordinateurs en utilisant la simulation de Monte Carlo. L'utilisation du programme "Venturer", qui a été développé par l'auteur, est illustrée par un exemple de passerelle.

ZUSAMMENFASSUNG

Der Beitrag beschreibt wie eine systematische stochastische Lebenserwartungsberechnung unter Einbezug der Kosten mit Hilfe einer Monte Carlo Simulation auf Mikrocomputern durchgeführt werden kann. Die Anwendung des vom Autor entwickelten Programmes "Venturer" wird am Beispiel einer Fussgängerbrücke erläutert.

1. INTRODUCTION

A designer usually has a target life, referred to as a design life, for which s/he will attempt to design the structure. The actual service life, however, will be shorter or longer than the design life. It is, therefore, necessary to examine the uncertainties that influence the service life if rational decisions for economical design are to be made. The economic analysis should not only acknowledge the engineering uncertainties (i.e. physical, statistical and model) but must also take into account the financial risks caused by the uncertainties in the economic environment. The life-cycle costing exercise must include estimates of construction cost, salvage value (if any), design life as well as maintenance/repair costs over the life of the structure. Deterministic life-cycle (i.e. single point) estimates produced to compare and evaluate alternative designs can lead to bad decisions. These estimates, at best, aspire to produce likely values of life and cost. Figure 1 shows that if only likely values of the objective function (e.g. life) of the alternative designs A and B (on scale OS) are available to a designer then B will be chosen. On the other hand availability of the full probability distribution not only provides insight into the nature of the designs but also throws a very different light on their relative merits, and may lead to the alternative A being chosen as a safer or a low risk design. There is a rapidly growing awareness in the professions of the need for taking a view that reflects the stochastic nature of the problems that the designers are called upon to model and analyse.

2. DESIRABLE FEATURES OF LIFE EXPECTANCY STUDIES

All quantitative studies have two important features. Firstly, there must be a mathematical model which expresses the relationship between the objective (e.g. life, cost) and the various variables that are recognised to be the controlling factors. For a model to be successful it should be reasonable, complete and adaptable. A reasonable model does not violate the basic logic of the process being modelled and provides plausible results. A 'complete' model contains all the important influencing factors (termed uncertainties) and their interrelationship. Generally a real-life system will have innumerable factors influencing the objective. Inclusion of all these will cause loss of manipulative flexibility. Unimportant factors have to be identified and neglected. An adaptable model can be easily enriched in light of new knowledge and insight that ensues from its use as well as in light of continuing research in materials and structures. The studies should reflect this and allow the designer to assess the effect of choice of a model on the objective. Secondly, a reliable probability distribution of the objective can result only from a set of carefully estimated probability distributions of the variables (termed uncertainty profiles). When these are based on empirical studies or historical data it is important to be aware of the changing circumstances and environment. Due weight should be given to expert opinion and motivational and cognitive biases should be avoided by using Delphi technique. An important point that needs to be emphasised is that the uncertainty profiles of the various variables do not all conform to some convenient shape but can range, for example, from a near normal for one variable to near exponential shape for another within a model.

3. ANALYTICAL TYPE OF STUDY

This type gives stylised probability distribution of the objective function. It is obtained analytically from the uncertainty profiles of the controlling factors which have to be expressed in stylised forms. The mean and standard deviation of the objective are obtained from those of the variables and from the coefficients of correlation between them. This involves the use of simplifying assumptions about the variables and their correlation coefficients [1,2]. With many design problems of even modest complexity this approach may not even be possible [3] unless the model is simplified so much that it is no longer reliable; in that case the analyst gets a (so called) precise answer to a wrong question. Surely even an approximate answer to the right question is vastly superior. The author's "experiments" with senior engineers, postgraduates and undergraduates, have shown that they are ill-at-ease with this method and experience difficulties even with very simple problems.

4. MONTE CARLO SIMULATION TYPE OF STUDY

Unlike a field or a laboratory experiment, simulation can be conducted entirely on a computer by expressing the interactions and the dependencies among the various controlling factors in the form of a mathematical relationship. It has been used in a host of situations [4,5,6]. In Monte Carlo Simulation, we "construct" a large number of structures with our model to reflect the characteristics of our design. From these large number of results, the probability distribution (termed venture profile) of the objective is obtained. The input information does not have to be forced into some idealised mathematical shape, and full probability distribution of the objective is obtained. There is virtually no constraint as far as the complexity of the model is concerned. Thought processes associated with analysis and synthesis in this approach are positively more attractive to engineers. Sensitivities of the objective to the various factors as well as to the various mathematical models can be easily studied. This helps to identify the factors which contribute most to the phenomenon under study and to assess the effects of errors in the estimation of the data and in development of the models. Efforts to improve the estimates and the design procedures can then be made in proportion to their relative importance. Additionally the results of these analyses are very valuable for drawing attention and allocating effort towards improving the construction, maintenance and use of structures.

5. MICROCOMPUTER SIMULATION

It is most relevant to note that in the coming decade, microcomputer power is expected to quadruple, at least. Memory size alone is likely to quadruple every three years for a constant price. Bell [7] predicts that "the power of todays Cray X-MP (four processors delivering a peak power of one billion floating point operations per second... and a main memory of one million 64-bit words) will be available in a workstation". This is roughly 60,000 times the power of a personal computer. Even today, one of the criticisms levelled against simulation is that it is computationally expensive. This view is out of date by a number of years. These costs will be very trivial in the near future. Other reservations that are often expressed are the time and the cost of programming. This is valid if every problem needs to be programmed separately. To overcome this and to encourage greater use of probabilistic modelling techniques a package has been developed [1] which is designed to (a) provide an aid to learning modelling and simulation techniques and (b) provide an applications program for general use in a host of situations. It has been used for synthesising and analysing a number of problems, such as cost risk analysis of a hydroelectric project, stability of dams, project appraisal and atomic bomb detonation effects [4]. The application program allows direct entries of the mathematical model and data interactively without any need to access the codes. These entries can be "enriched" and revised as and when necessary. Other features included are resimulation, filing and hard copy facilities. Facility for sensitivity analysis is of particular value. Perhaps the most important argument for using Monte Carlo Simulation is the fact that the exponents of the analytical approach resort to using this method to check the validity and the reliability of their analytical methods.

6. ILLUSTRATION

Figure 2 shows a section of the foot-bridge used as the example. For this illustration we shall consider the service life in terms of the deck (slab) only. In order to determine the service life of a structure it is necessary to identify the influencing factors, their actual effects and the respective failure mechanisms. This is called "Failure Mode and Effect Analysis" (FMEA). A check list of the factors, mechanisms and their effects for structures can be obtained from various studies [8]. Corrosion of reinforcement as a result of carbonation of concrete is considered to be the most significant failure mode. Here we shall consider failure mechanisms mainly due to this factor. The effects of the other factors are implicitly included in variables that define the climatic conditions.

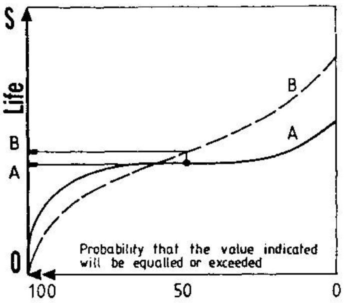

Fig.1 Comparison of Venture Profiles

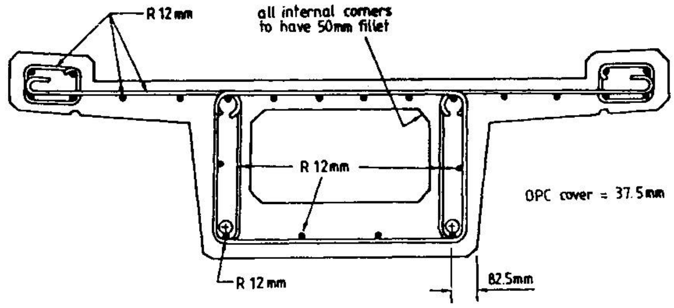

Fig.2 Foot-bridge: X-section

6.1 Model for Life

The question as to which model should be used is an area of wide controversy. The author does not intend to enter into this argument, and for his illustration he will employ a model provided by Siemes et al [8]. In their model the service life (L) is the sum of three elements; the time for carbonation, the time gap until the visibility of corrosion and the prolongation of the service life due to coating, i.e.

$$L = t_{cb} + t_{cr} + t_{ct}$$

Enrichment of this equation yields the following model:

$$L = \left[\frac{(c-d)}{R\,k} \times \frac{2.7}{46w-17.6} \right]^2 + \frac{0.08c}{d_i\,V_c} + \left[\frac{(c-d)S}{180 f_o} \times \frac{(T/T_c)\ln f_o}{1-e\hat{}-[(T/T_c)\ln f_o} \right]$$

where t_{cb} = carbonation time; t_{cr} = time to visible corrosion
t_{ct} = time extension due to coating; w = water/cement ratio
R = cement type factor; k = climate type factor
c = concrete cover; d = carbonated depth (mm)
d_i = bar diameter (mm); V_c = rusting rate (mm/year)
T = maintenance period; T_c = coating durability parameter
f_o = damage of coating (mm/year) S = coating thickness (mm)

For economy of space the last term is discarded with the assumption that coating will not be employed. Our model now has seven variables. The diameter of bars d_i is considered to be deterministic (12 mm). The uncertainty profiles of the other six are shown in Figure 3. Venture profile of the service life is shown in Figure 4. The sensitivity profile shows the relative importance of the factors. The lengths of the darker bars are scaled to give the variations in the average value of the objective as influenced by changes in the respective uncertainty profiles. The lighter bars give the ratio of the total change in the average value of the objectives to the total change made in the average value of the variable.

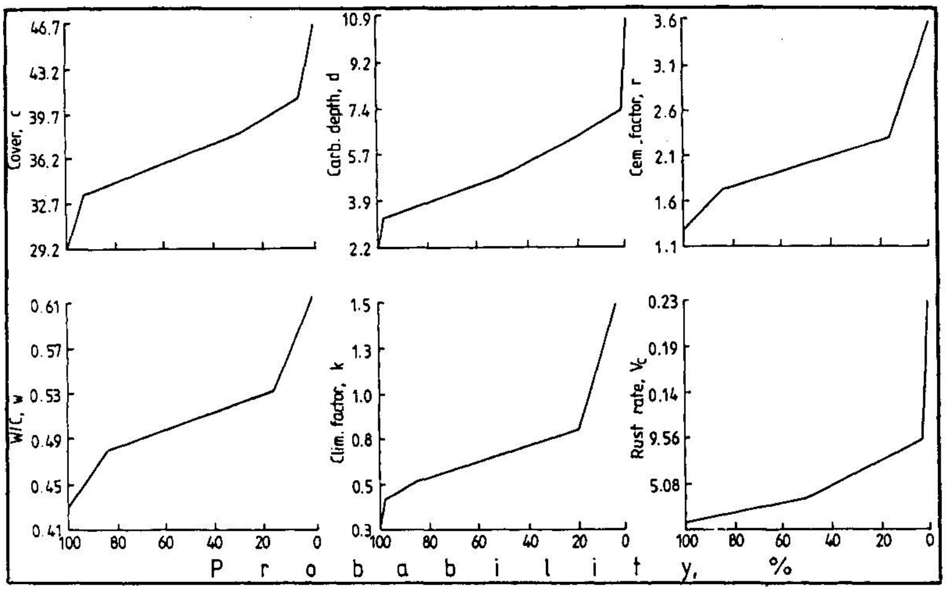

Fig.3 Uncertainty Profiles of variables in Life model

6.2 Model for Cost

There are, again, various ways in which the total cost of a structure can be modelled. For illustration let us use the following for estimating the Net Present Value (NPV) of the bridge.

NPV = Present worth factor x salvage value + present worth factor x (annual income - annual maintenance cost) - initial construction cost.

Enrichment of the model leads to the following; written in the form which can be entered directly through the keyboard (with the help of inbuilt logical checks and editing facilities provided in the Venturer) with meaningful variable names.

```
NPV = (1/(1 + (discourate/100))^life) * salvage)
    + ([(1 + (discourate/100)) ^life-1]/[(discourate/100)
    * ((1 + (discourate/100)) ^life)] * (anincome - anmaint))
    - constrcost
```

Venture profile for life (L) obtained earlier can now be entered as an uncertainty profile. Figure 5 shows the results with the assumption that the ranges of the other five variables are, in order in which they appear above, 7 to 12%; 14 to 147 years; £70,000 to 130,000; £100,000 to 220,000; £50,000 to 90,000 and £1,000,000 to 1,400,000. The results are obviously very valuable in design and economic studies such as evaluation of alternatives or establishing optimum maintenance policies. The sensitivity profile puts the relative importance of the engineering and economic factors in proper perspective.

7. CONCLUSIONS

Monte Carlo technique as implemented in the form of VENTURER, an interactive and tolerant educational and applications package, helps to overcome effectively the serious limitations of the analytical approaches to life-cycle studies. It encourages healthy scepticism towards assumptions in modelling and towards quantification of the influencing factors. It encourages and facilitates sensitivity analyses at the modelling and parameteric stages.

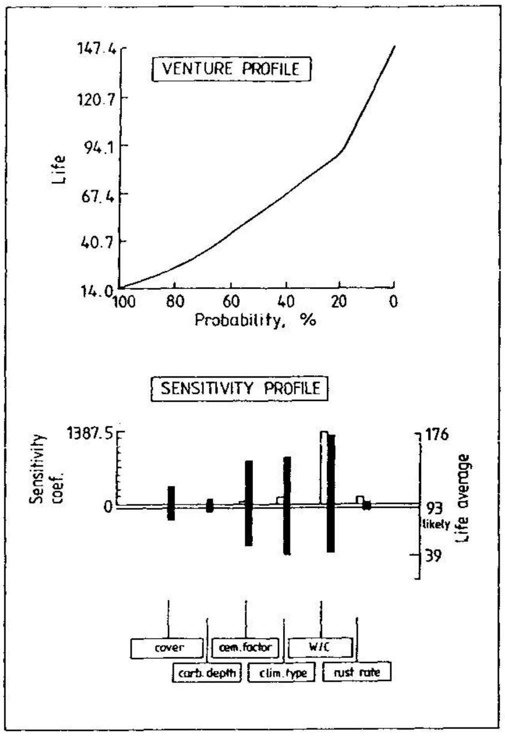

Fig.4 Life results

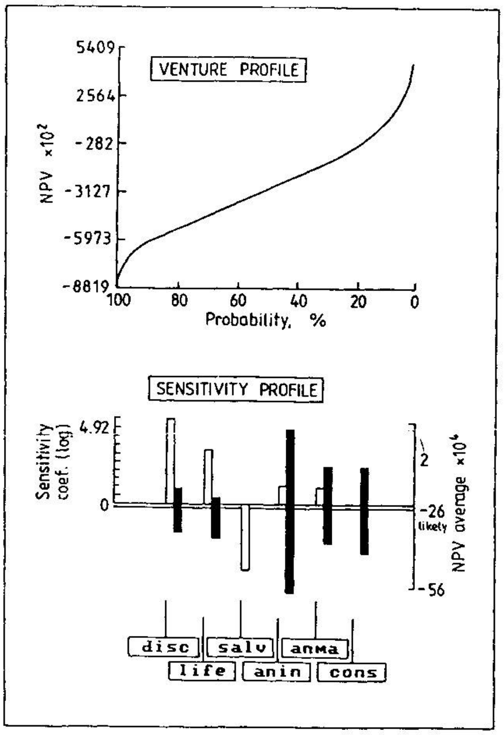

Fig.5 Cost results

ACKNOWLEDGEMENTS: Invaluable help from Ashok and Ranjit in the development of VENTURER and assistance from Javeed Shakeel in putting it through its paces is acknowledged.

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Remaining Service Life of Corroding Structures

Durée de vie restante de structures corrodées

Restlebensdauer von korrodierten Stahlbetonbauten

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SUMMARY

The prediction of the remaining service life of a corroding structure has been calculated up to now using empirical approaches and without taking into consideration the possible progressive structural damage that corrosion provokes. In a first attempt, values of several corrosion rates were implemented in Tuuti's model on service life and some kind of quantitative calculation could be presented. In the present paper, engineering considerations are introduced in the approach and the loss in load-carrying capacity is calculated for different corrosion rates assuming several simplifications.

RÉSUMÉ

La prédiction de la durée de vie restante des structures en train de se corroder, est faite habituellement par calculs approximatifs et sans considérer les différents niveaux de détérioration que la corrosion provoque. Dans la première tentative, des valeurs de la vitesse de corrosion ont été introduites dans le modèle de Tuuti sur la durée en service et un certain niveau de quantification a pu être atteint. Dans la présente communication, quelques considérations structurelles sont introduites et des exemples de pertes de résistance mécanique sont calculés en fonction de la vitesse de corrosion.

ZUSAMMENFASSUNG

Die bisherigen Verfahren zur Bestimmung der Restlebensdauer von korrodierten Stahlbetonbauten waren empirische Näherungen ohne Berücksichtigung der zunehmenden Schädigungen. In einem ersten Versuch wurden im Modell aus Tuuti die Daten verschiedener Korrosionsgeschwindigkeiten verwendet, um quantitative Abschätzungen zu erhalten. In der vorliegenden Arbeit werden zusätzliche ingenieurwissenschaftliche Betrachtungen zur Einschätzung der Tragwiderstandsverminderung vorgenommen. Die Restlebensdauer wird, unter der Annahme gewisser Vereinfachungen, für verschiedene charakteristische Korrosionsgeschwindigkeiten abgeschätzt.

INTRODUCTION

Service life prediction is complex matter in which both technical and economical consequences are involved. The need to study parameter has arisen from the unexpected premature deteriorations shown by reinforced concrete structures exposed to aggressive environments. The corrosion of reinforcements has resulted to be one of the most frequent causes of these premature failures.

Different proposals intended to calculate, either the life time of a new structure or the remaining life of a deteriorating one, may be found in the literature (1). Three of the authors of this contribution have also suggested in previous papers (2)(3) a methodology to calculate the remaining service of structures damaged by rebar corrosion. This methodology could be proposed due the large amount of corrosion intensity values, i_{corr}, which were collected by the authors along 20 years of experiments. These i_{corr} values were determined from Polarization Resistance results, Rp, measured in specimens prepared in the laboratory and on-site in real structures. In this paper a new advance in this line of research is offered, which considers the conversion of corrosion rate values in loss of load-carrying capacity terms. Some simple examples for columns and beams are calculated.

CALCULATION OF THE REMAINING SERVICE LIFE OF A CORRODING STRUCTURE

Three main points need to be considered to attempt to calculate the remaining service life: 1) The type of deterioration process involved; 2) The main parameter which controls the deterioration rate; 3) The unacceptable level of damage which makes the structure unsafe.

In the evoked previous paper (3)(4), for the particular case of corroded structures the following answers were given to these points:

1. Tuuti's model (4) was adopted as a deterioration model for a corroding structure. This simple model considers an initiation and a propagation period.
2. The loss of bar cross-section of the rebar was taken as rate-determining parameter. This loss in cross-section was determined from real i_{corr} values, whether they remain constant along the propagation period of (2) whether they change with the moisture content of the concrete (3).
3. The levels of deterioration suggested by the CEB in its Bulletin no. 162 were those taken into account.

Figure 1 is the result of jointly considering all these aspects. This figure allows the approximate calculation of the residual service life in terms of corrosion rates and of the loss in bar cross-section, assuming that this loss in diameter decreases linearly with the corrosion rate. The following relationship may be stablished from Figure 1.

$$\varnothing (t) = \varnothing_i - 0.023 .\ i_{corr} . t$$

$\varnothing (t)$ = the rebar diameter at time t (mm)
$\varnothing_i$ = the initial diameter of the rebar (mm)
i_{corr} = the corrosion rate ($\mu A/cm^2$)
t = the time after the beginning of the propagation period (years)
0.023 = the conversion factor of $\mu A/cm^2$ into mm/year

However, the translation of these concepts into engineering terms is necessary, if the remaining load-carrying capacity and the safety of the structure are to be de - termined.

ENGINEERING CONSEQUENCES OF THE REBAR CORROSION

The main undesirable effects of the corrosion in the structure may be summarized as: a) a loss in the steel integrity: loss of cross-section and likely in the mecha nical properties; strenght and ductility; b) the splitting and spalling ot the cover with a loss in the concrete cross-section in the case of spalling; c) a loss in bond between concrete and steel in the case of cracks running paralled to the reinforcements and provided that the loss in steel section is high.

Very little attention has been paid in the literature to these effects (5)(6) and thus, the experimental data are scarce and usually obtained through and artificial acceleration of the corrosion process.

HYPOTHESES CONSIDERED IN THE PRESENT STUDY

The study of the load-carrying capacity loss will be approached at three different levels. A first one which consider the deterioration of a section in such a way that the strength loss against different action effects (bending moment, shear force, axial force, etc.) could be established. A second level which will introduce the deterioration model of an element, so that the loss in load-carrying capacity if isolated elements (for instance; simply supported beams) could be proposed. Finally a third level which will consider the whole structure and take into account the possible redistribution of the action effects, provided that it is allowed by the remaining materials ductility. For the purpose of the present study only the first level is going to be considered.

The assumption considered here may represent the case of corrosion in carbonated concrete where cracks are not produced in the cover because the concrete remains wet and therefore the oxydes may diffuse through the pores. They are:

- an homogeneous loss around the whole steel surface is supposed wheter i_{corr} remains constant or varies with ambient humidity. No pitting or localized corrosion is studied at this moment,
- no loss in bond is produced, which means that no parallel cracks were generated during the corrosion process and therefore the cover remains free from damages,
- no loss in steel mechanical properties is taken into account.

EXAMPLES AT THE CROSS-SECTION LEVEL

Ultimate bending moment (M_u)

The decrease in the ultimate bending moment has been studied at a cross-section 0.40 m deep and 0.25 m wide, with a tension reinforcement (4 Ø 14 mm or 2 Ø 20 mm).

Four corrosion rates have been considered (0.1, 1.0 and 100 $\mu A/cm^2$), kept constant along time, and it has been assumed that corrosion only affects the decrease in the diameter of the rebars, according to the equation given in the preceeding section.

In this way and bearing in mind the conventional hypotheses for reinforced concrete, the curves in figure 2 have been plotted, which show the decrease of the ultimate bending moment in terms of the time elapsed since the beginning of the reinforcement corrosion (propagation time). Thus, while corrosion rates of 0.1 and 1 $\mu A/cm^2$ result in a slight decrease of the safety factor along the 50 years of the service life assumed, rates of 10 and 100 $\mu A/cm^2$ result in the desappearance of the safety factor in lifetimes of 16 and 2 years respectively, when the sections have been reinforced with 14 mm diameter rebars.

In figure 2, it can also be observed that cross-sections reinforced with higher diameter bars are less sensitive to the damages produced. Thus, with two reinforcements with the same steel ratio but with different bar diameters (4 Ø 14 mm and 2 Ø 20 mm) and for the same corrosion rate (10 $\mu A/cm^2$), the loss of the safety factor is reached at different time lapses of 16 and 22 years, respectively, the corrosion damage being reached at an earlier time in the cross-section reinforced with the 14 mm diameter bar. The interest to use big-size rebars is thus shown in order to delay the damages if an adequate concrete cover is provided.

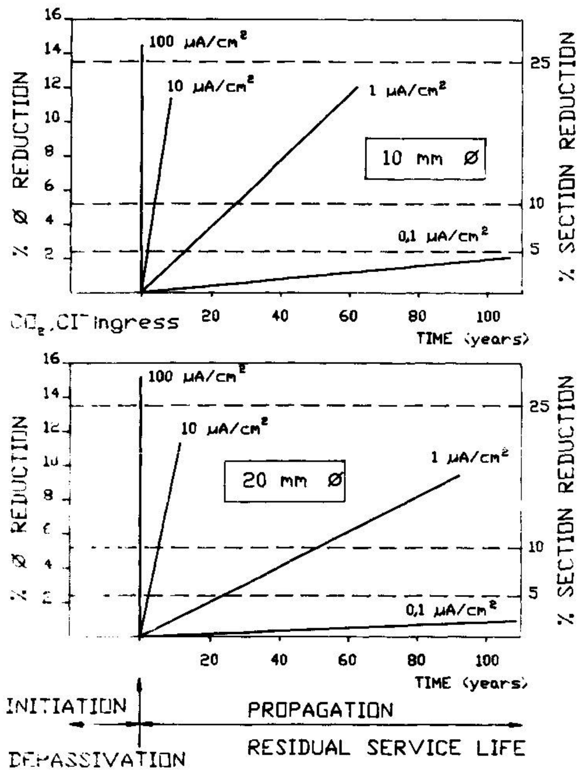

Fig. 1 - Rebar life time in function of its diameter and corrosion rate

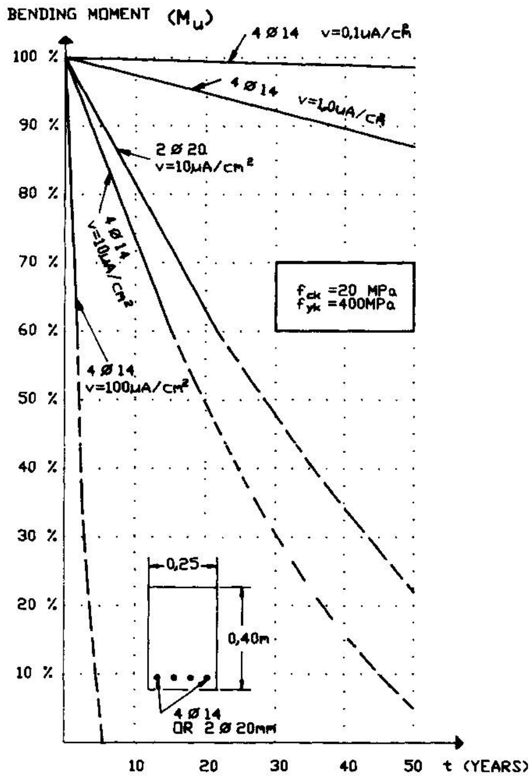

Fig. 2 - Loss in bending moment in function of the corrosion rate

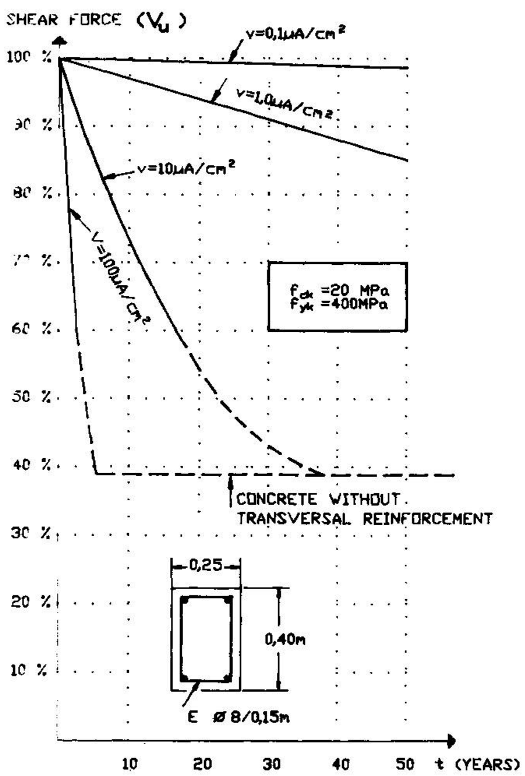

Fig. 3 - Loss in shear force in function of the corrosion rate

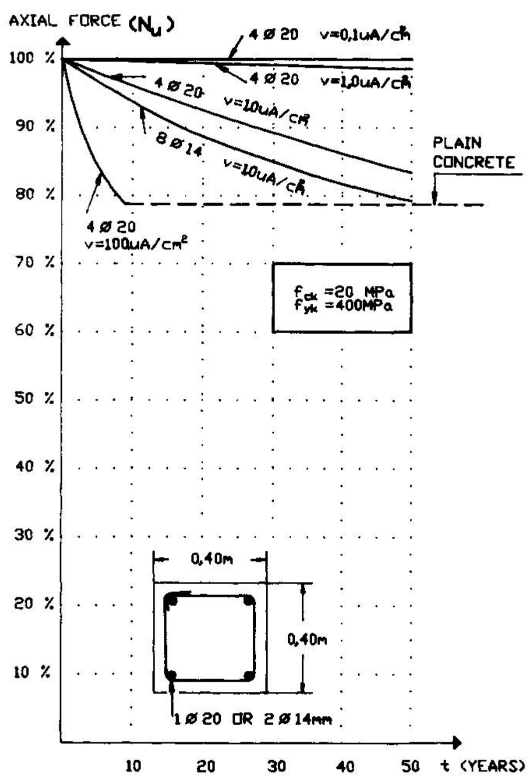

Fig. 4 - Loss in axial force in function of the corrosion rate

Ultimate shear force (V_u)

The decrease in the ultimate shear force of the same concrete cross-section reinforced with 8.0 mm diameter stirrups, located at 0.15 m distances, has also been investigated for different corrosion rates.

It is assumed that the deterioration of the longitudinal reinforcement is negligeable, what may be true, upon the reinforcement having a thicker cover, and that the shear force can be estimated by adding the shear carried by the concrete to the shear analysed by the truss analogy, which consists of the concrete and the longitudinal and transversal reinforcements.

The curves in figure 3 have thus been plotted which show again that corrosion rates of 0.1 and 1 $\mu A/cm^2$ scarcely impair that value of the shear force along the 50 years lifetime contemplated for the structure, while with corrosion rates of 10 and 100 $\mu A/cm^2$ the safety factor becomes zero along periods of 15 and 2 years respectively. At the latter two corrosion rates, the web reinforcement collapses along 4 and 35 years periods, and, from that time, the shear force is exclusively carried by the concrete itself and by the longitudinal reinforcement. This situation is somewhat theoretical, since, after these time periods and at these corrosion rates, the derioration of the longitudinal reinforcement must also be quite considerable, and therefore the shear force carried by this reinforcement must also be largely smaller than the shear shown in the horizontal lengths of the curves shown in figure 3.

Ultimate axial force (N_u)

Finally, the reduction in the bearing capacity of a 0.40 x 0.40 m cross-section axially loaded, reinforced with rebars at its four corners has also been studied for the same corrosion rates as indicated in the foregoing cases.

The curves plotted in figure 4 shown the evolution of the axial force along the time elapsed from the beginning of the reinforcement corrosion and have been estimated by adding up the axial force carried by the concrete to that carried by the reinforcement. There has not been taken into consideration the possible buckling of the longitudinal reinforcement, when neither the concrete cover nor the transversal reinforcement, also deteriorated can duly brace it.

At the cross-section reinforced with a 20 mm diameter rod at each corner, the damage caused is significative for corrosion rates of 10 $\mu A/cm^2$, so that, at 50 years life, most ot the axial force is virtually carried by concrete alone. For corrosion rates of about 100 $\mu A/cm^2$, the reinforcement disappears in its entire by before 10 years, so that the axial force is then carried by concrete only.

It is well known that a centrally loaded section is a theoretical situation, for these always appears certain eccentricities induced by the loads or by imperfections in the concrete cross-sections. In this connection, the possible irregular damaging of the reinforcement, may also result in a certain additional eccentricity which, combined with the previous eccentricities, results in a cross-section with a highly damaged reinforcement bearing with greater difficulting the theorical simple compressive stresses. Therefore, the horizontal lengths of curves in figure 4 give "optimistic" values, which will be reduced due to the higher sensitivity of plain concrete to the action of such "accidental eccentricities".

In figure 4 there can also be compared the different evolution of the axial force with the time elapsed, in cross-sections reinforced with the same steel ratio, but different rebar diameters (1Ø 20 mm or 2Ø 14 mm at each corner). It is shown again the interest of using greater diameters, taking in advance the precautionary measures mentioned above.

DISCUSSION

The estimation of the remaining service life of corroing structures has been, up to now, mainly based on empirical or qualitative models and on the subjetive experiencie of expertises. A quantitative method could not be found in the literature.

The methodology presented here relates the loss in load-carrying capacity with the loss in steel cross-section as a function of its corrosion rate. It is still a simplified method of approaching the prediction of the remaining service life and some simplifications had to be assumed due the lack of experimental information, but it offers a semiquantitative methodology to make predictions concerning the damage in the critical sections of the structure. Provided that, the corrosion intensity of a corroding structure and its age are known, an estimation of the time required to reach a critical loss, can be made.

For a more accurate estimation, several questions should be experiments. Aspects such as bond, steel mechanical properties, the presence of cracks, will have to be elucidated considering different corrosion or damage levels. Also, not only sections, but simple elements (beams and columns) and, finally, the whole structure will have to be investigated.

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Remaining Fatigue Life of Old Steel Bridges

Capacité de résistance à la fatigue des vieux ponts en acier

Beurteilung der Restlebensdauer alter Stahlbrücken

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SUMMARY

Because of the large number of existing old steel bridges, the evaluation of the remaining fatigue life of these bridges is a very important economic problem and also an interesting one from the engineering point of view. For investigating old steel bridges, crack growth tests have been added to the well established types of investigation. To verify the established procedure of investigation and rating traffic load will be simulated on dismantled bridges in the laboratory.

RÉSUMÉ

Etant donné le grand nombre de vieux ponts en acier existants, l'évaluation de leur capacité de résistance à la fatigue est un problème économique de grande importance et un problème de grand intérêt pour l'ingénieur. Aux méthodes classiques d'inspection des vieux ponts en acier, il est possible d'ajouter l'étude du développement de la fissuration. Afin de pouvoir vérifier et étalonner la procédure d'inspection établie, les cas de charge de trafic ont été simulés en laboratoire sur des ponts démantelés.

ZUSAMMENFASSUNG

Die Ermittlung der verbleibenden Nutzungsdauer alter Stahlbrücken stellt wegen der grossen Anzahl dieser Brücken ein beachtliches wirtschaftliches Problem dar und ist zugleich eine interessante ingenieurwissenschaftliche Aufgabe. Bei Untersuchungen alter Stahlbrücken wurden die üblichen Untersuchungen um Rissfortschrittsversuche an standardisierten Proben erweitert. Zur Bestätigung der Beurteilungsverfahren werden alte ausgebaute Brücken simulierter Verkehrsbelastung bis zum Auftreten von Rissen unterworfen.

1. INTRODUCTION

In Germany as well as in other countries, there are many old riveted steel bridges which are approaching their design life after about hundred years of service. The replacement of all these bridges far exceed the available financial resources. However, even if the funds existed, replacement would be the least acceptable option in several cases because many of the bridges are historic structures [1]-[4].
Being concerned with the rating of old steel bridges, we added a further component to the procedure of investigation. We performed crack growth tests on samples of the material of some high stressed structural elements of the bridge shown in fig. 1, thus determining data to define the fatigue behaviour of the material [5].

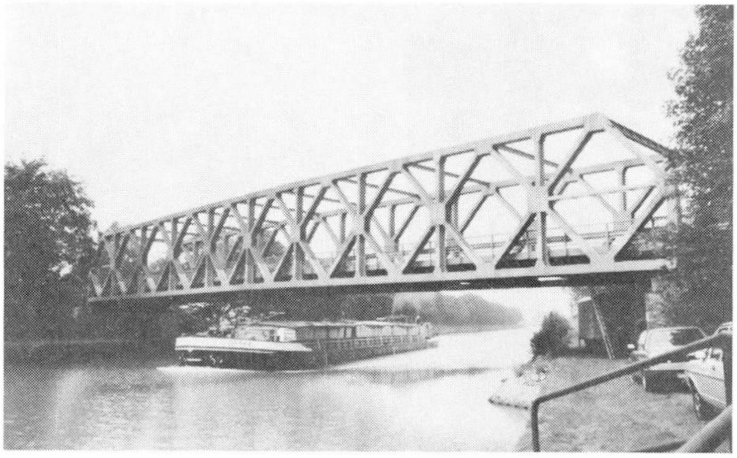


Figure 1.
Riveted highway bridge of 59 m span in north-west Germany

However, faced with the problem of rating old bridges, the need of further research is recognized in order to calibrate the results of the rating procedure with experience. This calibration can only be done by simulating traffic load on old removed bridges the stresses in which are well-known from measurements before removing. Research is under way in BAM on bridges one of them is shown in Fig. 2.

2. BRIDGES UNDER INVESTIGATION

Two bridges are under investigation (fig. 1, fig. 2). In the following, a brief description about the investigation and their results are presented.

2.1 Truss Bridge Crossing a Canal in North-West Germany

The load bearing elements of the bridge (fig. 1) that crosses the Dortmund-Ems-Kanal near Osnabrück, are outlined in fig. 3. It was built in 1953, the rolled girders were produced in the thirties. During an inspection, cracks have been detected at some of the cross girders near the connection to the principal load bearing elements, trusses of 59 m span (fig. 4). The investigation of the bridge included strain measurements at longitudinal and cross girders under static load and under traffic flow and tests on the

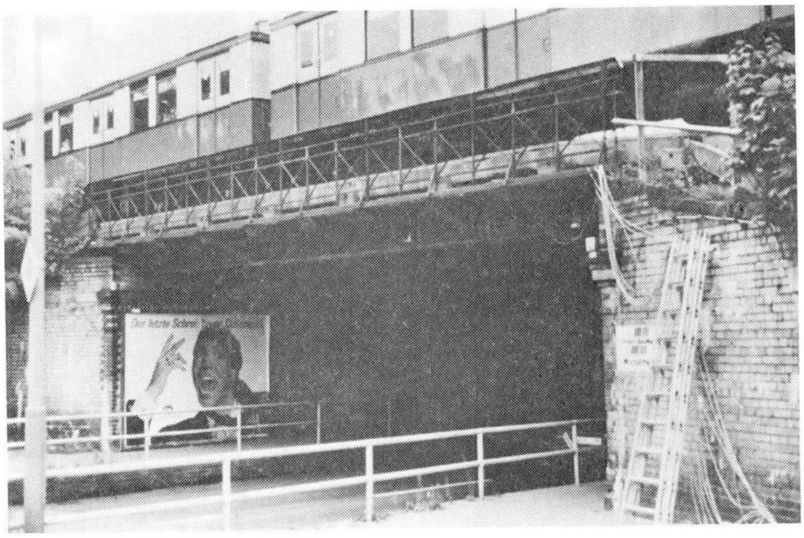


Figure 2.
Girder Bridge of Berlin Metropolitan, replaced in 1988

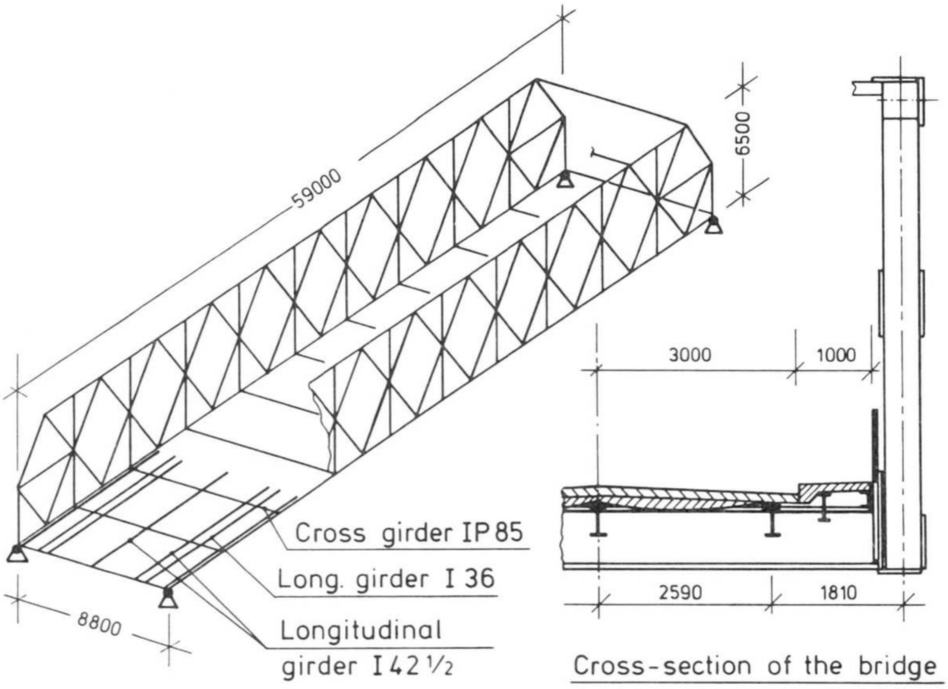


Figure 3: Load bearing elements of the bridge of fig. 1

material of the structural elements in question. The measurements under defined static load by a two-axle truck of about 20 t weight showed that the calculated stresses at the points of maximum stressing are considerably higher than the measured ones. From the measurements under traffic we obtained that only in very few cases of passing of heavy trucks, stresses exceeded the cut-off limits of the standardized fatigue curves [6] (fig. 5). In addition to the standardized material investigation, we performed crack growth tests on CT-specimens as recommended in ASTM E-647-83. The results in terms of the representing Paris equation (fig. 6)

$$\frac{da}{dN} = C \, \Delta K^n \qquad (1)$$

a crack length
N number of loading cycles
ΔK cyclic stress intensity factor
C;n material parameters of crack growth

confirmed that the material could be regarded as mild steel concerning the fatigue behaviour.

Figure 4.
Structural detail at the connection of the cross girders to the main truss girder and the location of cracks

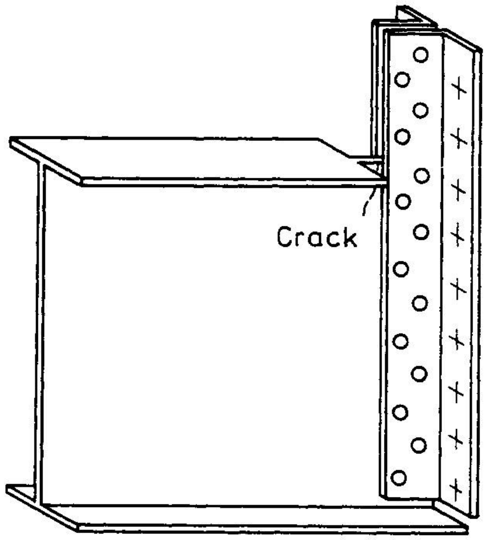
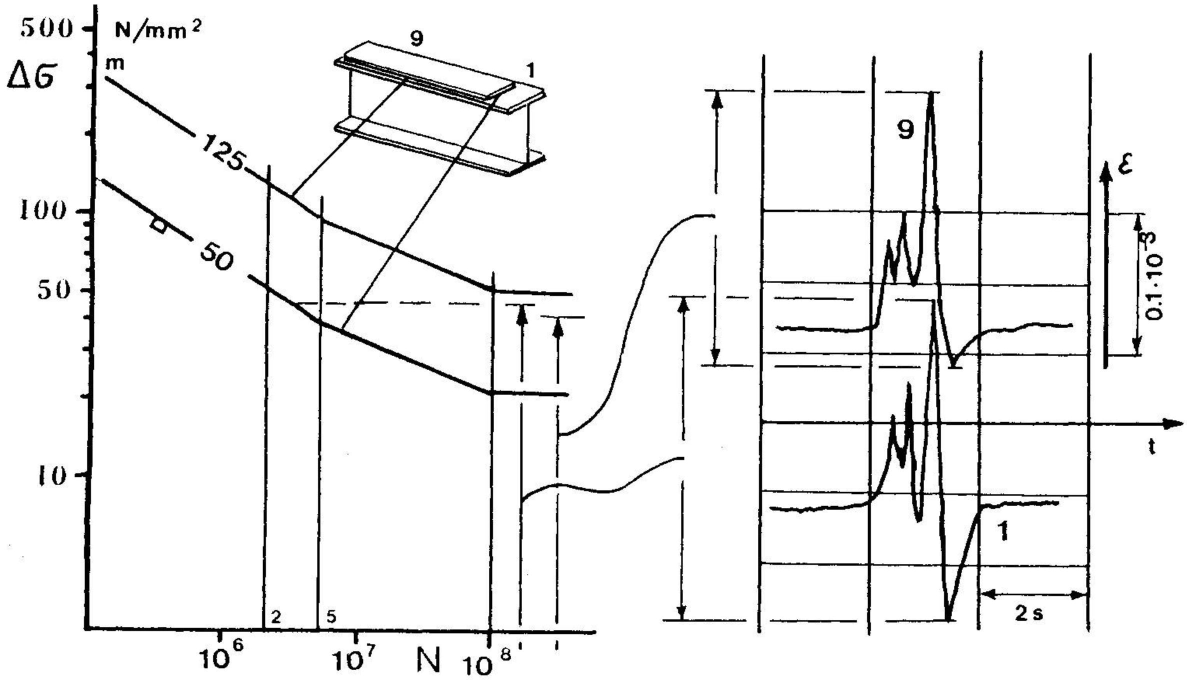


Figure 5 (below).
Fatigue strength curves (categories 125 and 50) and maximum measured stress cycle at two measuring points at the central longitudinal girder during passing of a heavy vehicle.



The crack growth tests offered the possibility to assess the amount of growing of a crack of a certain length which was not detected during an inspection within the time interval up to the next inspection (3 years). From the measurements, it turned out that the cracks (fig. 4) were caused by residual forces.

2.2 Riveted Girder Bridge of Berlin Metropolitan

The single-track bridge (fig. 2) with a span of about 12 m was built in 1896. It had to be replaced because the road it crosses had to be widened.
Before the bridge was removed, we performed strain measurements at several points of the structure, at elements of the main girders and of cross girders. The stresses during the hours of measurement did not reach the cut-off limits of the standardized European Fatigue Strength Curves[6]. However, we do not know accurately which type of traffic passed the bridge during the nearly 100 years of the bridge's life in an eventful history.

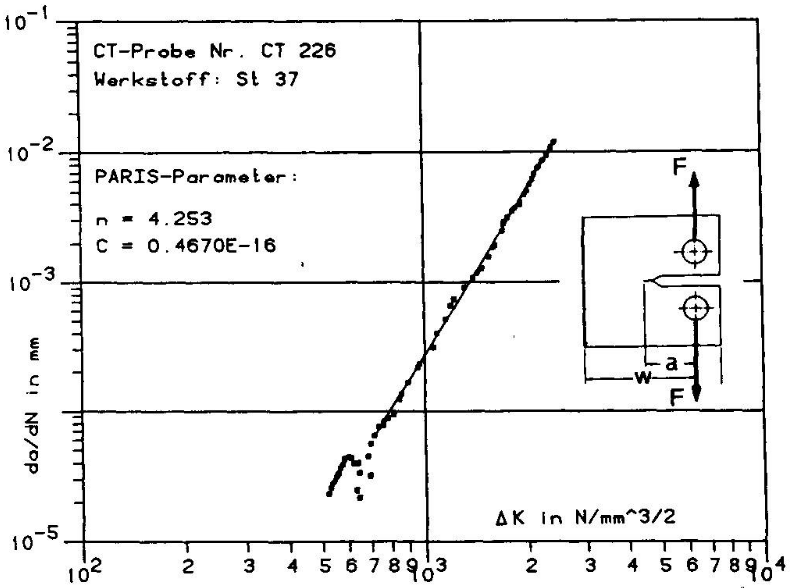


Figure 6: Diagram of crack growth rate da/dN versus cyclic stress intensity ΔK as elaborated for a compact tension (CCT) specimen (ASTM E 647-83)

It is intended to simulate traffic load on this bridge and another one similar to it in the laboratory to confirm the assessment of remaining fatigue life by the evaluation procedure that is common practice.

3. PROBLEMS OF APPLICATION OF CRACK GROWTH INVESTIGATION

The application of fracture mechanics in the evaluation of remaining fatigue life implies the modelling of cracks in structural elements which can probably occur, fig. 7. The crack growth is only governed by the stress intensity at the crack's tip as expressed by the Paris equation (1). It is an advantage of the concept of fracture mechanics that it leads to a deeper insight into the problem of survival of structural members under cyclic loading than only the application of fatigue strength curves after calculation of cummulative damage by collectives of stresses. Fracture mechanics procedures are recommended when cracks have been observed, because the shape and the dimensions of the cracks are known as well as the surrounding stress fields.

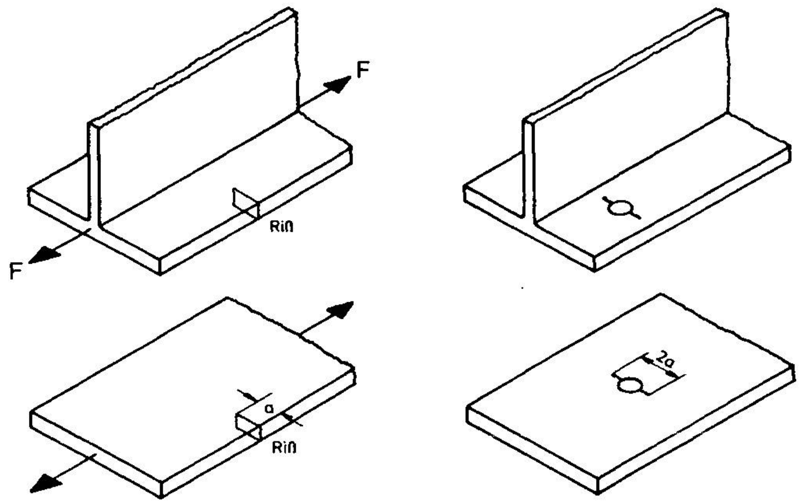


Figure 7:
Crack pattern for evaluating stress intensity factors and crack propagation

4. CONCLUSION

The evaluation of the remaining fatigue life of old riveted steel bridges is a very important task of structural engineering and comprises several different investigations. Faced with the problem to give an estimation of the remaining fatigue life of a certain bridge, which had cracks in a few structural elements, we extended the commonly used evaluation procedure by crack growth tests. The results of these tests made it possible to identify the material in terms of fatigue strength and gave the fundament to determine the propagation of postulated or observed cracks within the inspection period of time.

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Stress Corrosion of Cement Mortars in Ammoniumsulfate Solution

Corrosion sous contrainte des mortiers de ciment dans des solutions de sulfate d'ammonium

Spannungskorrosion von Zementmörteln in Ammoniumsulfatlösungen

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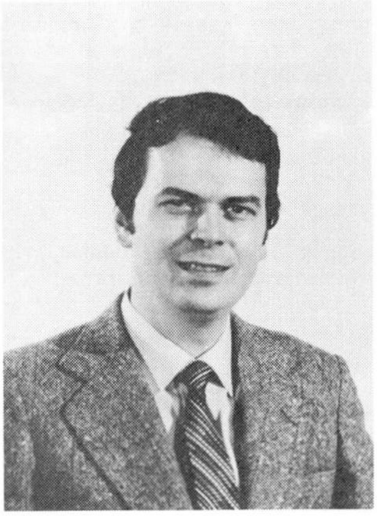
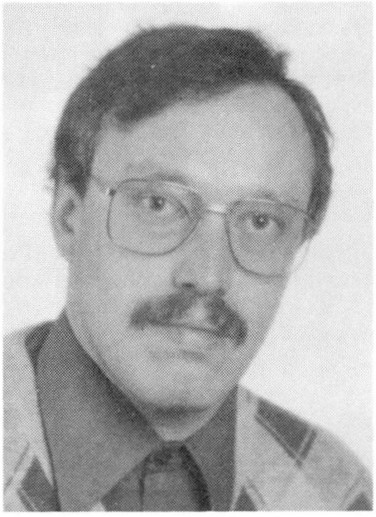
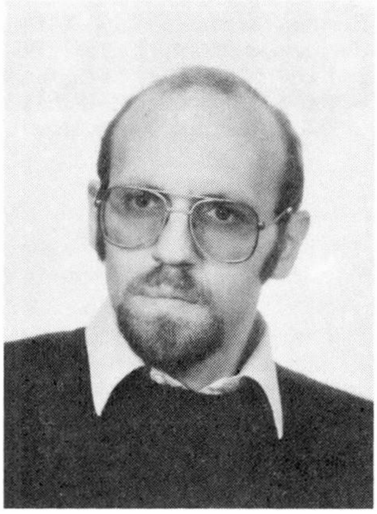


Prof. Dr.-Ing. habil. Ulrich Schneider, born 1942, received his master's degree in mechanical engineering at the TU Braunschweig in 1969. He received his doctorate in civil engineering 1973 and works as a university lecturer since 1979. In 1981 he was awarded the chair in building material science in the dep. of civil eng. at the Univ. of Kassel. His research work comprises mainly the durability and corrosion of non-metallic inorganic material and thermal properties of building structures and materials.



Dr.-Ing. Erich Nägele, born 1953, received his master degree in chemistry at the University of Karlsruhe, FRG in 1978. He received his doctorate in civil engineering 1981. Dr. Nägele works now in the civil engineering department at the University of Kassel in the field of stress corrosion of non-metallic inorganic materials and on the electric properties of materials.



Norbert Dujardin, born 1950, received his civil engineering master degree at the University of Aachen. For 4 years he was involved in the design and calculation of pipe supports for nuclear reactors. For the last 3 years he has been working in the civil engineering department at the University of Kassel. His research is devoted to the stress corrosion of non-metallic inorganic materials and fracture mechanic of concrete.

SUMMARY

The effects of type of cement, of water/cement-ratio and of surface coatings on the stress corrosion of cement mortars in 5%- ammonium sulfate solution are reported. The effects observed for different cements and water/cement-ratios in stress corrosion correspond to the effects observed in classical corrosion, i.e. mechanical stresses are not present. Coated specimens, however, show a distinct decline in strength when a mechanical stress acts simultaneously to a chemical stress and when the coatings are destroyed by cracks. Thus, the use of coatings has to be reconsidered, taking into account stress corrosion.

RÉSUMÉ

Ce travail examine les effets du type de ciment utilisé, du rapport eau/ciment et de la protection des surfaces vis-à-vis de la corrosion sous contrainte des mortiers de ciment dans les solutions de sulfate d'ammonium. Les effets du type de ciment et du rapport eau/ciment sont les mêmes que pour une corrosion classique, s'il n'y a pas d'actions mécaniques. Des échantillons montrent une diminution de résistance marquée lorsque des actions mécaniques agissent simultanément à une action chimique et lorsque les couches superficielles sont détruites par fissuration.

ZUSAMMENFASSUNG

In dieser Arbeit wird über den Einfluss der Zementart, des W/Z-Wertes und von Oberflächenbeschichtungen auf die Spannungskorrosion von Zementmörteln in 5%iger Ammoniumsulfatlösung berichtet. Die für die verschiedenen Zemente und W/Z-Werte beobachteten Effekte entsprechen denen bei normaler Korrosion, d.h. wenn zusätzliche mechanische Spannungen nicht vorhanden sind. Beschichtete Proben zeigen jedoch unter Spannungskorrosion einen starken Festigkeitsabfall, wenn die Beschichtung Fehler aufweist. Daher muss die Verwendung von Beschichtungen neu überdacht werden, wobei die Spannungskorrosion zu berücksichtigen ist.

1 INTRODUCTION

Chemical attack and mechanical properties of cementitious materials are usually being studied separately. However, in practice, chemical attacks are common to load bearing structures, therefore the simultanous action of chemical and mechanical stresses, known as "stress corrosion", has to be considered in any evaluation of the durability of cementitious materials. It has been shown that cementitious materials like many other building materials are also subjected to stress corrosion [1-5].

2 EXPERIMENTAL

2.1 Materials

The experiments were performed with mortar prisms 4x4x16 cm made according to the German standard DIN 1164. Three cements were used throughout the tests, a German Portland cement,PZ 35F (PC), a German blast furnace slag cement, HOZ 35L with a slag content of 50% by weight (BFC) and a German fly ash cement, FAZ 35L with a fly ash content of approximately 30% by weight (FAC) . The analysis of the cements is given elsewhere [2- 5]. The aggregate was quartzitic German standard sand.

2.2 Specimens

The specimens were made with the general composition aggregate : cement : water = 3:1:w/c. Hence the contents of cement and aggregate were kept constant in all experiments.

The specimens were made with notches 10mm depth, 45° opening angle by pressing a suitable wedge into the fresh mix. A detailed overview of the tests is given in table 1. The specimens were cured for 28 days under water before immersed into the aggressive media.

2.3 Test Procedure and Test Program

The mortar prisms were immersed into 5%-ammonium sulfate solutions and loaded with load levels up to 50% of their initial flexural strength, which was determined prior to immersion. The loading device is shown in fig. 1. It is decribed in more detail in [2-5].

Unloaded specimens were stored in the same containers, so loaded and unloaded specimens were stored in the same solutions and under the same conditions. The load levels applied to the respective series may be taken from table 1, where the test program is summarized.

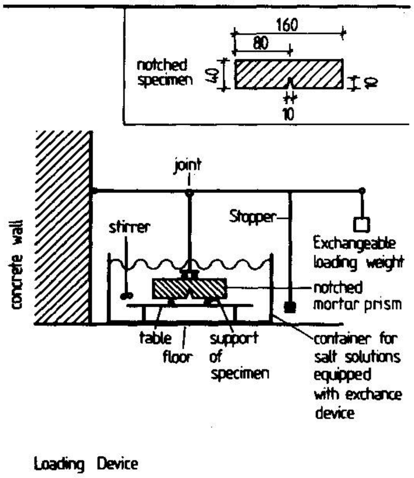


Fig.1: Loading Device for Stress Corrosion Tests

Table 1: Test Program

Series No.	Effect investigated	Type of Specimen	Load levels (%)
1	Type of Cement	W/C= 0,7; PZ, HOZ, FAZ	0 30
2	W/C-Ratio	PZ , HOZ W/C=0,55; 0,65;0,75	0 30
3	Surface Treatment and Coating	PZ , W/C=0,7 Sand blasted PUR-coated EP-coated	0

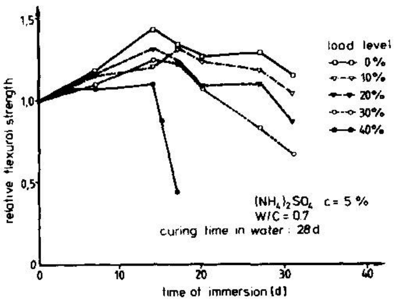
3. RESULTS AND DISCUSSION

Due to the limited space available not all results can be presented and discussed here. Thus only a brief survey on the most importnant features of the stress corrosion of cementitious materials is given and those topics most important for building practice are discussed briefly. Further details may be found in [2-5].

3.1 Stress Corrosion Phenomena in Aqueous Solutions

Fig.2, where the relative flexural strength is plotted versus the immersion time, shows some results of stress corrosion experiments from series 1 in 5%- ammonium sulfate solutions. Strength first increases due to chemical reactions like the formation of calciumsulfoaluminmates etc. and then decreases due to the formation of gypsum which causes sulfate expansion. If a mechanical stress acts simultaneously to the chemical one the initial strength development is

Fig.2: Stress Corrosion of Portland Cement Mortars



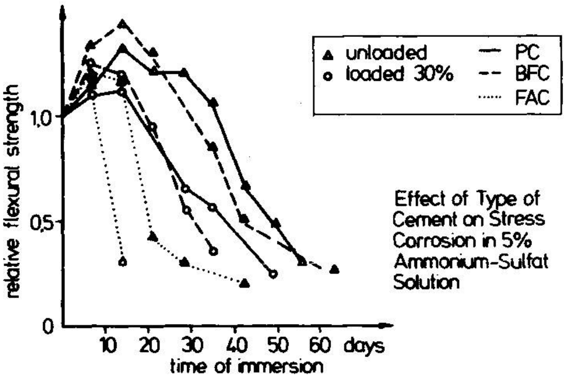
Effect of Load on Strength Development
of Mortar Prisms

diminished and the deterioration of the specimens occurs at earlier times. At very high load levels no initial strength increase is observed. Thus, stress corrosion leads to a significant decline in flexural strength for loaded specimens compared to unloaded ones. The higher the load the earlier failure occurs. Similar results are obtained for other media [2,3,5]. Thus it may be concluded, that cementitious materials are subjected to stress corrosion in many common aggressive aqueous solutions.

3.2 Effect of Type of Cement

Fig.3 shows the effect of type of cement. Stress corrosion occurs for all types of cement investigated.

Fig. 3: Effect of Type of Cement on Stress Corrosion of Mortars



The type of cement affects the life time .i.e. in ammonium sulfate solution the portland cement and the blast furnace slag cement specimens show a higher life time compared to the fly ash cement specimens respectively, because in the corrosion times involved the higher $Ca(OH)_2$ content of the portland cement and the blast furnace slag cement pastes acts as a buffer. The strength decline is most pronounced for the fly ash cement. For the loaded specimens the same behaviour is observed. Portland cement and blast furnace slag cement show nearly the same strength decline with immersion time, whereas the fly ash cement showns very rapid failure. Thus, in stress corrosion the effect of the type of cement is the same as in ordinary corrosion of cementitious materials.

3.3 Effect of Water/Cement-Ratio

Due to the fact that cementitious materials in ammonium sulfate solution are subjected to two entirely different forms of corrosion failure, namely the dissolving attack and the sulfate expansion attack the effect of the water cement ratio depends also on the type of cement. Fig.4 shows the effect of the w/c-ratio for portland cement and fig.5. for blast furnace slag cement prisms both unloaded and loaded with a load level of 30% of initial strength. The decline of strength becomes

Fig.4: Effect of W/C-Ratio on Stress Corrosion of Portland Cement Mortars

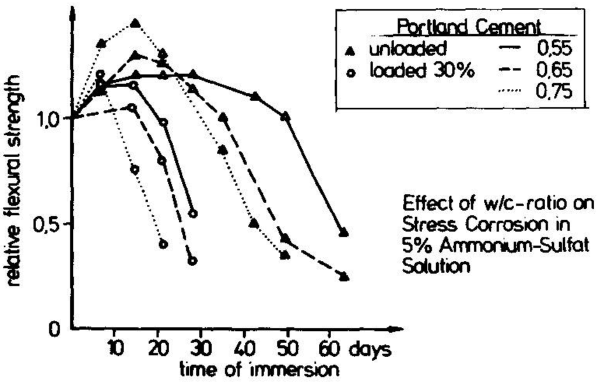
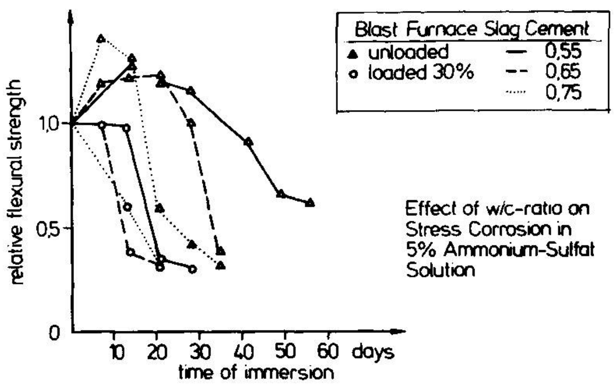


Fig.5: Effect of W/C-Ratio on Stress Corrosion of Blast Furnace Slag Cement Mortars

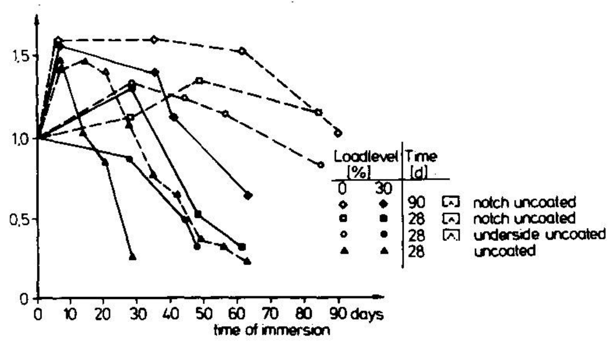


more distinct with increasing w/c-ratio for loaded and unloaded specimens. Thus, in general the effect of w/c-ratio on stress corrosion is the same as during ordinary corrosion without mechanical stresses. However, when a mechanical stress is superimposed onto the chemical stress the differences in strength development seem to become smaller than those observed with unloaded specimens. Furthermore, it can be seen from fig's. 4 and 5 that the initial increase in strength increases with increasing w/c-ratio, i.e. with increasing capillary porosity. This is an indication that the initial strength increase of mortar specimens stored in ammonium sulfate solution is due to the formation of new phases inside the pore volume.

3.4 Effect of Surface Coating

Fig.6 shows the decline of flexural strength of differently coated notched cement mortar prisms /6/. The coating prevents an attack on the coated areas of the specimen. Thus, the corrosion resistance of coated specimens is signifikantly higher than that of uncoated ones. The more of the specimens surface is covered by the coating the better is the performance under corrosion. However, the picture is quite different under stress corrosion conditions. The coated specimens, both those with the tensile side uncoated and those with only the notch region uncoated show extremely distinct load effects. This means, that if a coating has some failure points i.e. cracks, through which the aggressive medium can penetrate, these areas are the starting points of a severe stress corrosion, which under load, can reduce the life time of the structure to values of the unprotected material. The effect is observed for both types of coatings studied thus far, namely notch-free and one side free-coatings. It seems therefore to be necessary to reconsider the use of coatings at least for highly load bearing structural members subjected to a chemical attack.

Fig. 6: Effect of Coated Mortar Prisms under Stress Corrosion 5% Ammonium-Sulfat Solution



4.SUMMARY AND CONCLUSIONS

Stress corrosion leads to significant reductions in strength of cementitious materials subjected to simultaneously acting mechanical and chemical stresses. Several media, amongst which are ammonium and sodium sulfate have been shown to cause stress corrosion. Severe reductions in strength and life time are the consequence of this process. In general, most of the important parameters of concrete technology affecting corrosion resistance also affect stress corrosion behaviour in about the same manner. An additional deleterious effect of stress corrosion has been discovered with coated specimens. Cracks and other failures in the coating result in an enhanced stress corrosion reducing the life-time of a coated structure to that of an uncoated one. Stress corrosion is further assumed to be responsible for the large differences known to exist between conventional laboratory studies, neglecting the effects of mechanical stresses on chemically attacked cementitious materials. It is therfore urgently necesseary to study the observed effects in more detail and to incorporate stress corrosion in the standards and evaluation procedures of durability of cementitious materials.

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Prestressed Bridge Girders after 20 Years of Service

Poutres en béton précontraint après 20 ans de service

Vorgespannte Brückenträger nach 20 Jahren Beanspruchung

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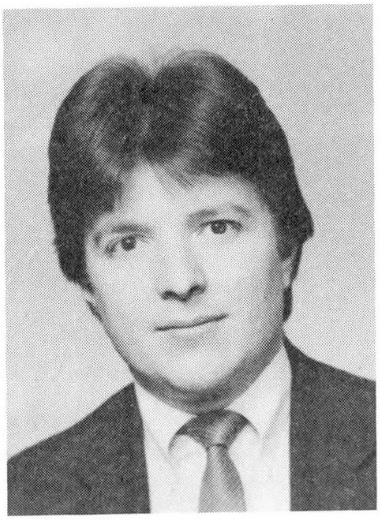

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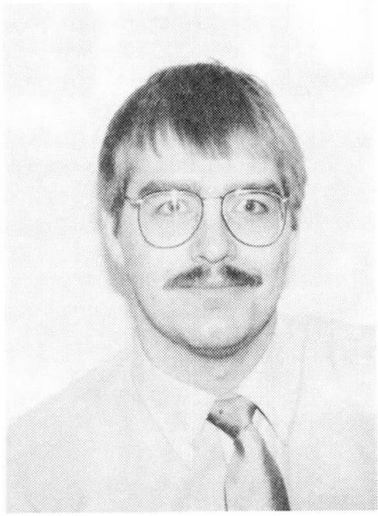

S. A. Olson received his M. S. degree from the University of Minnesota in 1989, and is working on a doctoral degree. His research is on the reusability and fatigue life of prestressed bridge girders.

SUMMARY

The prestress losses, material properties, fatigue, and structural performance characteristics of four prestressed bridge girders are described. The girders had been in service under actual traffic conditions for twenty years, and showed little corrosion or durability problems. Prestress losses with time were approximately equal to those predicted by current American codes.

RÉSUMÉ

Les pertes de précontrainte, les propriétés des matériaux, la fatigue, et les caractéristiques structurales de quatre poutres de pont en béton précontraint sont décrites. Les poutres ont été en service sous des charges de trafic durant vingt ans, et n'ont pas eu de problèmes de corrosion ou de durabilité. Les pertes de précontrainte avec le temps ont été pratiquement égales à celles prescrites par les codes américains actuels.

ZUSAMMENFASSUNG

Die Vorspannungsverluste, Materialeigenschaften, Ermüdung und das konstruktive Verhalten vier vorgespannter Brückenträger sind beschrieben. Die Träger waren für ca. 20 Jahre unter Verkehrslasten in Gebrauch und zeigen nur geringe Korrosions- oder Gebrauchstauglichkeitsprobleme. Die Vorspannungsverluste im Verlauf der Zeit entsprechen etwa den Werten der zur Zeit gültigen amerikanischen Normen.

1. INTRODUCTION

The determination of the actual material and structural properties is a key step in the rating of structures. In the case of most steel bridges the assessment of the material properties is quite straight forward. This is not the case, however, for concrete and prestressed concrete bridges where significant time effects are involved. In prestressed concrete bridges, in particular, loss of prestress due to relaxation, creep, shrinkage, and cyclic loads is very difficult to predict. In this paper the results of a study on four prestressed girders subjected to real traffic for twenty years is reported. The main objectives of the study were:

(1) To determine the actual prestress losses of the girders and to compare them to those given by current code-prescribed equations.
(2) To assess the remaining fatigue life of such girders.
(3) To determine the material properties of the girders and components and to compare them with non-destructive test results.
(4) To determine the amount of impact damage that such a girder can sustain before replacement or repair is required.
(5) To assess the performance of two types of strand repair techniques subjected to fatigue loading.

Due to space limitations only the results of the first two girder tests will be addressed with respect to items (1)-(4).

2. DESCRIPTION OF THE SPECIMENS

In 1986 a four-span county road bridge over an interstate highway in Minneapolis, Minnesota, was removed due to road realignment work. Four of the girders removed from the center spans of this bridge were brought to the University of Minnesota Civil and Mineral Engineering Structures Laboratory for testing. The girders were originally fabricated in July of 1967. At the time of removal they had been in service for approximately twenty years.

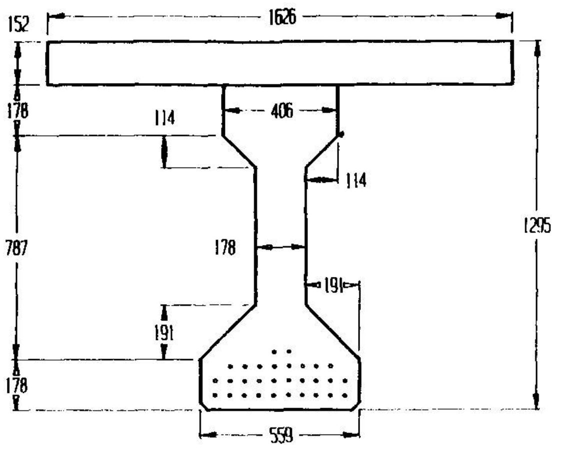

Figure 1 - Girder dimensions (in mm)

The girders are standard AASHTO-PCI Type III girders, and details of the girders are shown in Figure 1. The girders were 1143 mm. deep, 19.71 m. long, prestressed with thirty 13 mm. diameter 1724 MPa stress-relieved strands. Twenty-two of the strands had a straight profile in the bottom flange of the girders. The remaining eight strands were draped; the two hold-down points were located 1525 mm. either side of centerline. The strands were initially stressed to a design prestressing level of 1206 MPa. The construction records indicate that the girders were pulled to approximately 2 percent over this design prestress.

Each girder was tested individually, with a new slab cast to simulate the actual bridge. The new slab was 152 mm. thick, 1626 mm. wide, and reinforced in the same manner as the original bridge slab. The loads were applied by two actuators located at the hold-down points, resulting in a constant moment region over the middle 3025 mm of the girder.

3. PRESTRESS LOSSES

The girders arrived at the laboratory in an apparently uncracked condition. The only visible damage was located at the ends of the girders: a small amount of epoxy paint covering the ends of the strands had spalled and rust was evident on the ends of the strands. The first loading applied to the girders was a cracking cycle. This test was repeated several times to accurately determine the crack opening load. The measured load at first cracking was 378 kN, and the loading was continued to 623 kN, about 45% of its calculated ultimate capacity. Small cracks propagating up to 620 mm from the bottom fiber in the constant moment region were the only visible damage due to these loadings. Four techniques were used to estimate the prestress losses:

(a) Formation of the first crack: From the load at first cracking and tensile capacity of the concrete from cores, the prestressing can be estimated.
(b) Reopening of cracks: By carefully monitoring the opening of cracks with high-resolution LVDT's during reloading, the decompression load can be calculated.
(c) Discontinuities of the load-deflection curve: The change of stiffness when the cracks reopen can be obtained directly from load-deflection curves.
(d) Exposing, instrumenting, and cutting strands: By using strain gages, a direct measurement of the prestress level can be obtained.

While it is difficult to state the initial prestress because the construction documents are the only source available, the in-situ prestress levels as given by methods (a) to (c) were very close to one another. They indicated a remaining prestress of 895 MPa in the strands, or about 74% of the original prestressing. Method (d) gave somewhat lower values (64%), but this can be explained by slight misalignment of the gages, transfer length and differences in prestress from strand to strand. The total prestress losses calculated were about 310 MPa, very close to the lump sum prestress losses predicted by the current AASHTO specification [1].

4. FATIGUE LOADING

The complete load histories imposed on the two girders are shown in Table 1. The load history was selected to model past and current American bridge specifications [1] for prestressed girders. They are based on the bottom fiber stress. The oldest specifications allowed no tension there, while more recent editions allow between 0.8 f'_c and 1.6 f'_c, where f'_c is the compressive strength in kgf/cm^2.

Girder 1 - Following the static tests (G1PL) to investigate prestress losses, the girder was subjected to almost three million cycles of load at increasing levels of nominal tensile stress in the bottom fiber. The corresponding stress ranges in the strands for fatigue tests G1F1 through G1F4 were 55, 69, 90, and 207 Mpa, respectively. Only a small crack growth was noticed during the initial cycling for G1F1 and G1F2; in general cracks stabilized with the first 10×10^4 cycles at each level. Most new cracks developed during the static tests conducted intermittently to monitor the amount of damage accrued during from fatigue. Figure 2 shows the load-deflection curves after each of the loading runs. Very little, if any, fatigue damage was evident after this very severe load history. There was noticeable permanent set in the specimen only after loading G1F4.

Girder 2 - After an initial series of static tests (G2PL), the beam was cycled for two million cycles (G2F1 and G2F2) in its undamaged state. The concrete in the bottom flange at the centerline was then chipped away to expose four strands to simulate damage due to impact from a truck travelling underneath the bridge (G2D1). The beam was then subjected to fatigue loading (G2F3). Two bottom strands were then cut to simulate additional damage (G2D2), and the girder was fatigued again (G2F4). After this two more strands were cut (G2D3) and the fatigue load repeated (G2F5 and G2F6). One strand broke during G2F4, and at least another during G2F5. Figure 3 shows the load-deflection curves for static tests after each of these runs.

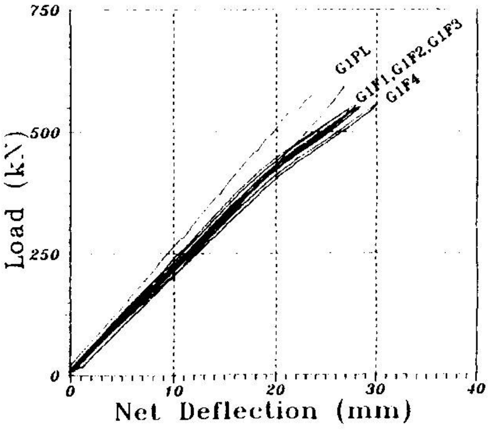

Figure 2 - Load-deflection for G1

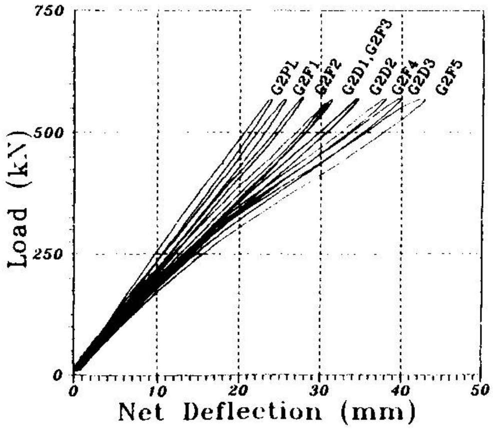

Figure 3 - Load-deflection for G2

The results of the tests on the first two specimens indicated that fatigue loading which produces strand stress ranges of less than 104 MPa has little or no effect on the ultimate strength and ductility of the member. The size of these girders and their close spacing in the field means that very little damage could have been done by fatigue since the sections were uncracked during their service life. The stress range in service was probably less than 20 Mpa.

TABLE 1 - Load Histories

Girder Number	Test Label	Load Type	Number of Cycles (N)	Bottom Fiber Stress	Purpose of Test
1	G1PL	Static	10	$3.2\sqrt{f'_c}$	Determine cracking load and losses
1	G1F1	Cyclic	5×10^5	0	Fatigue
1	G1F2	Cyclic	10×10^5	$0.8\sqrt{f'_c}$	Fatigue
1	G1F3	Cyclic	12×10^5	$1.6\sqrt{f'_c}$	Fatigue
1	G1F4	Cyclic	6×10^4	$3.2\sqrt{f'_c}$	Fatigue
1	G1U	Static	1	---	Test to ultimate
2	G2PL	Static	10	$3.2\sqrt{f'_c}$	Determine cracking load and losses
2	G2F1	Cyclic	5×10^5	$0.8\sqrt{f'_c}$	Fatigue
2	G2F2	Cyclic	15×10^5	$1.6\sqrt{f'_c}$	Fatigue
2	G2D1	Static	5	$1.6\sqrt{f'_c}$	Concrete around strands removed
2	G2F3	Cyclic	5×10^5	$1.6\sqrt{f'_c}$	Fatigue
2	G2D2	Static	5	$1.6\sqrt{f'_c}$	Cut two strands
2	G2F4	Cyclic	5×10^5	$1.6\sqrt{f'_c}$	Fatigue
2	G2D3	Static	5	$1.6\sqrt{f'_c}$	Cut two strands
2	G2F5	Cyclic	18×10^3	$1.6\sqrt{f'_c}$	Fatigue
2	G2U	Static	1	---	Test to ultimate

5. ULTIMATE STRENGTH TEST

After approximately three million cycles, both Girder 1 and 2 were tested monotonically to failure (Figure 4). In both cases the failure was initiated by the upward buckling of the longitudinal slab reinforcement at very large centerline deformations (530 mm or greater). The final failure occurred as crushing of the slab followed by an explosive outward failure of the poorly confined beam web. For Girder 1, the failure occurred at a load of 1303 kN and at a centerline deflection of 530 mm. The load at ultimate constituted approximately 95 percent of the ultimate capacity of the beam based on nominal material properties and the assumption that all steel yielded. By the time failure was reached the entire 10 ft at the center of the beam had formed a long plastic hinge, and inclined shear cracking had moved out from the constant moment region to the quarter points in the beam. The inclined shear cracks in this area had reached the top flange of the beam, while flexural cracking had progressed to the bottom of the slab. The fatigue loading did not appear to have affected the ultimate strength of the section.

The failure for Girder 2 was similar, except that it occurred at a much lower load (890 kN) and somewhat higher deflection (635 mm.). This was due to: (1) the cutting of the strands and loss of concrete section and (2) fracture of at least two additional strands during G2F4 and G2F5. The cracking in Girder 2 during the last three loading runs was very severe, with large portions of the bottom flange completely separated from the girder.

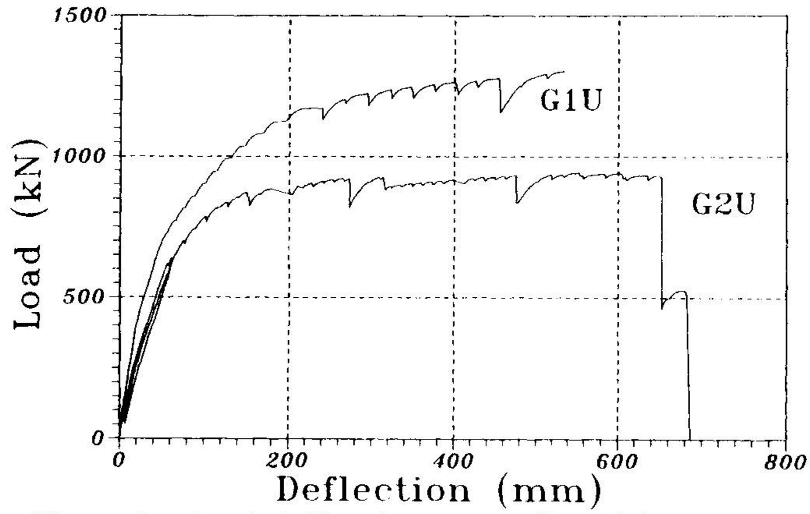

Figure 4 - Load-deflection curves for ultimate tests.

6. MATERIAL PROPERTIES

A series of nondestructive tests was conducted on Girder 1. The tests included a variety of uniformity/strength tests as well as chloride ion penetration investigations. Windsor probe (penetration test), Schmidt hammer (surface hardness), pulse velocity (compressive wave velocity) and breakoff tests (lateral pressure required to detach countersunk cylinder) were conducted and were correlated with 50 mm. and 100 mm. diameter cores drilled from the girder. In these tests, the concrete was found to be quite uniform throughout the girder with a compressive strength of about 58 Mpa. THe results showed a low coefficient of variation for all of the test methods with the exception of the break-off test and the 100 mm. cores. Problems with obtaining straight cores and good capping of the specimens help explain this variability.

Because the girders had been exposed to substantial amounts of deicing salts as spray from the interstate highway underneath, corrosion of the strands was considered a major potential hazard. The girders themselves received little or no deicing salts from above, as the original deck acted as a barrier. The top steel in the original deck was quite corroded, but the chlorides had not penetrated down to the girders. The strands which were exposed to check the effective prestress of Girder 1 were located in the flange near the end of the girder in an uncracked region. Evidence of some pitting corrosion appeared on one of the strand within 100 mm. of the end of the girder. Otherwise, the strands, which had a cover of 50 mm., appeared to be in excellent condition. Chloride ion penetration tests gave readings which were within the threshold limits of 250-350 ppm by weight of concrete usually assumed as the corrosion threshold. The highest reading obtained was 270 ppm, but the next highest readings were on the order of 120 ppm. It is interesting to note that the readings obtained from the bottom flange were consistently higher than those obtained from the web, and the readings obtained on one side of the girder were consistently higher than those obtained from the other side. It is expected that the girders which were exposed to the incoming traffic would have higher readings because the salt-concentrated mist tends to be carried under the bridge with the forward motion of the cars. Consequently, the side of the girder with the higher concentration of salts most likely faced the incoming traffic.

7. SUMMARY

The tests carried out so far indicate that the prestressed girders were in excellent condition after twenty years of service in an aggressive environment. There did not appear to be problems associated with corrosion of the girders, most likely because of the excellent concrete quality and the depth of cover. The prestress losses, estimated to be on the order of 310 Mpa, correlated well those predicted by the current AASHTO lump-sum method. The fatigue loading imposed indicated that for loadings up to HS20-44 or Type 3S2 vehicles the stress range was probably in the infinite life region of the fatigue curves. Thus the girders could be reused in other bridges if they are removed with care and NDT techniques indicate adequate material properties and no evidence of strand corrosion.

ACKNOWLEDGEMENTS

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Problems in Concrete Culvert Durability Studies

Problèmes liés à l'étude de la durabilité des ponceaux en béton

Probleme mit Untersuchungen zu Dauerhaftigkeit von Betondurchlässen

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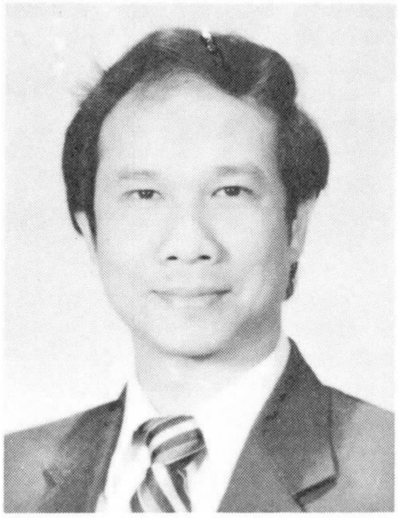

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SUMMARY

Several studies made concerning the durability of concrete culverts have led to inconclusive results. Inadequate consideration of enabling and triggering events contributed to the general problems in assessing the life expectancy of culverts. The more recent techniques using regression analyses for determining the culvert's service life encountered problems related to the inadequacy of sample data representative of the culverts and the inferior concepts employed to treat the data.

RÉSUMÉ

Quelques études, menées sur la durabilité des ponceaux en béton, ont débouché sur des résultats peu concluants. Des considérations insuffisantes, en ce qui concerne les multiples paramètres à considérer, contribuèrent à rendre difficile le problème général de la prévision de la durabilité des ponceaux. Des techniques d'analyse récentes, basées sur les régressions, furent appliquées à la détermination de l'aptitude au service des ponceaux. Toutefois, ces analyses se sont heurtées à l'inadéquation aussi bien des données simplifiées représentatives des ponceaux que de l'infériorité des concepts employés pour traiter les informations.

ZUSAMMENFASSUNG

Verschiedene Untersuchungen zur Dauerhaftigkeit von Betondurchlässen führten nicht zu schlüssigen Resultaten. Ungenügende Berücksichtigung der massgebenden Einflüsse führten zu Problemen bei Lebensdauerprognosen. Neuere Versuche unter Verwendung von Regressionstechniken scheitern an mangelhafter, nicht representativer Datenbasis und an ungenügenden Konzepten bei der Verarbeitung der Daten.

1. INTRODUCTION

Emphases of earlier studies of culvert durability are primarily placed on field surveys and experience of the analysts. Simple manipulations and statistics were performed manually to estimate culvert service life. Certain studies show the effects of enabling events resulting from pipe manufacturing and installation procedures. Examples of these events are the inadequacy of material, design, or construction processes. However, during the culvert durability studies, information concerning these events was not considered. The most frequently investigated events that caused the deterioration of culverts are the environmental conditions. These conditions are called the triggering events or external events that triggered the deterioration of the culverts, such as, the acid attack and flow velocity in the culverts. Current studies emphasize more on these events.

With the utilization of computers, more sophisticated statistical models can be performed using a large number of data. Unfortunately, this does not necessarily guarantee the reliability of the techniques developed. In some cases, casual observations were performed, and inadequate number of data was used to force the production of prediction models. In the following section, significant studies on durability of concrete culverts from several sources, including the author's own investigations, are presented. Emphasis of the studies are placed on the problems that may occur in relation to the reliability of the results.

2. STUDIES OF CONCRETE CULVERT DURABILITY

Annual deterioration, qualitative, and regression studies had been performed for the durability of concrete culverts. The following are examples of such studies performed in various states and summarized in Tables 1 and 2. Experimental studies are not discussed here.

2.1 Annual Deterioration and Qualitative Studies [1]

The earliest culvert durability study known to the author was performed in the State of Georgia in 1928, where 252 monolithic concrete culverts and 326 concrete concrete culverts were examined. The culverts were installed between 1915 and 1924. The average life expectancy is 31.2 and 27.6 years for monolithic culverts and concrete culverts, respectively. Another survey of 1,837 reinforced concrete pipe culverts was conducted in West Virginia between 1932 and 1933. The age of the culverts inspected ranges from 3 to 11 years. Variables contributing to concrete durability were discussed. The study results in 50 years average expected service life with an average annual deterioration of 2%. In 1947, Pennsylvania Department of Highways performed culvert study based on a survey of 10,439 concrete culverts installed from year 1918 to 1927. The study concludes a life expectancy of 40 years assuming that yearly deterioration remained constant.

A total of 442 concrete pipes were investigated in the State of Mississippi in 1964. The age of pipes ranges between 5 to 41 years, averaging 22.9 years. The study that was conducted qualitatively based only on field performance prescribed no life expectancy. However, it concluded an excellent condition of the pipes.

2.2 Regression Studies [1]

The research performed in 1974 for the State of Utah may be the first study of concrete culvert durability using regression analysis. Fifty eight pipe culverts were studied; however, only 14 of them were of concrete. A rating scale from 0 (failure) to 10 (excellent) was used in the study. Several dependent variables were used such as pH, soluble salt content, and electric resistivity. Pipe rating was used as the dependent variable. The study recommended service lives of 40 years for the design of interstate highways and 30 years for other installations.

Two surveys of concrete culverts were conducted by the Ohio Department of Transportation: the first was in 1972 for 545 culverts and the second was the reinspection of 64 concrete pipes in 1984. In the first study the rating scale used ranges from 1 (excellent) to 5 (poor). Pipe rating was used as the dependent variable, while age, slope, pH, sediment depth, and rise were used as independent variables. Two sample groups, pH below 7 and above 7, were analyzed for separate prediction models. Emphasis was placed on the latter group which resulted in prediction models. For pH value over 7, the models yield culvert service life of thousands of years. The second study is a refinement of the earlier one performed for sample group of pH below 7. Pipe ratings from 0 (as manufactured) to 100 (reinforcing gone) were used here. Rating was used as the dependent variable, while flow, pH, pipe size, slope, and sediment depth were used as the independent variables.

The author of this paper performed two regression studies. The first is based on information of 521 sections of concrete culverts made available by the Ohio Department of Transportation (ODOT) [1]. Rating scale with ranges similar to that of ODOT (1=excellent to 5=poor) was used in the study. Pipe rating was used as the dependent variable, while age, rise, pH, slope, and sediment depth become the independent variables. No grouping of pH values was performed here. Six multiplicative and six additive models were generated as the results of the study. However, these models can not be used for predicting the service life of a particular culvert. The expected "service life" of the culverts yield 86 years. The second study was performed by the author as a discussion of of ODOT's culvert study [3].

3. PROBLEMS ENCOUNTERED IN STUDIES

3.1 General Problems

Studies for determining the life expectancy of culverts have not been completely satisfactory. The variability in the design, material, manufacturing, installation, and maintenance of concrete culverts were seldom included in the analyses since usually such information is not available. Table 2 shows that only the earlier study in Georgia investigated the enabling events. Other studies had implicitly included the existence of these events through the rating scales. Furthermore, observations of culverts are usually performed at a certain "point in time" during the survey, where the variables related to the culvert geometry and triggering events are measured. During the observations, these triggering events are assumed constant. However, events such as, flow velocity, flow depth, sediment depth, and acidity may not be the same throughout the life span of culverts. These factors may contribute to the inaccuracy and unreliable results of analyses based on the average annual deterioration or regression analyses.

3.2 Inadequacy of Culvert Data

Many of the sample data, including those used by the author, are not representative of culverts in a particular state. For example, Figure 1 shows a bi-nodal sample distribution of pH values of observations in the State of Ohio. A representative sample data is expected to have a uninodal distribution. A plot of the variable Age and pH of these data shows a "boxing" of culvert ages above pH=7 as shown in Figure 2. These data indicate the lack of observations in other pH regions.

3.3 Conceptual Problems

In the regresssion analyses, the rating of culverts is usually treated as the response variable dependent upon other variables, such as, age, rise, flow depth, flow velocity, sediment depth, slope, and pH values. Only independent variables that contribute any information to the prediction of the culvert rating are included in the regresssion equation. An example of such an equation is as follows [2]:

$$RATE = -05469+0.0316AGE+0.0099RISE+11.2484/PH+0.2377SLOPE^{1/2}$$

In several studies, the variable AGE is algebraically exchanged with variable RATE, such that AGE becomes the response variable. Then, as RATE is set to a scale that represents the expiration of the culvert's service life, AGE becomes the "service life" of the culvert and the prediction equation is used for predicting a particular culvert for given variables. This technique is erroneous since throughout the regresssion, AGE is assumed, as it should be, an independent variable, but at the same time becomes a dependent variable when algebraically altered with RATE. Also, an attempt to treat AGE as a response variable for use in the regression analysis is inapropriate, since AGE is the time from installation to inspection of the culverts and is logically independent from other variables [3].

Incomplete or inadequate data adds to the problem of predicting the culvert's service life. For example, in a sample data set, the majority of the data were related to culverts still in service, and only a small fraction of the inspected samples represent expired culverts [3]. Despite the fact that none of the culverts in the data set are over 60 years old, yet through the algebraically altered prediction equation, one could predict that many of these culverts are already several hundred years old. Figure 3 shows an example result showing the relation between the

TABLE 1. Method of analysis and life expectancy of culverts

STATE	METHOD	SAMPLES	AGE (YRS)	LIFE EXP. (YRS)
Georgia	Avg. Annual Loss	252 monol. 326 pipes	2-13	31.2. 27.6.
W. Virginia	Avg. Annual Deterioration	1,837 RC pipe	3-11	50.0
Pennsylvania	Avg. Annual Deterioration	10,439 culv.	20-29	40
Mississippi	Qualitative Field Perf.	442 pipes	5-41	NA
Utah	Regression	14 pipes	NA	30-40
Ohio (ODOT)	Regression	545 pipes	1-45	Varies
Ohio (ODOT)	Regression	64 pipes	14-56	Varies
Ohio (ODOT)	Regression	198 pipes	5-57	Varies
Ohio (OSU)	Regression	521 pipes	1-45	86 (avg)
Ohio (OSU)	Regression	198 pipes	5-57	NA

TABLE 2. Enabling and triggering events considered for analysis of culvert

STATE	TIME OF SURVEY	VARIABLES RELATED TO ENABLING EVENT	VARIABLES RELATED TO TRIGGERING EVENT
Georgia	1928	*Fill height *Poor quality control of concrete mat'l *Joint failure of rigid sectional culverts *Poor alignment *Poor maintenance	*Wet/dry cond. *Scour cond.
W. Virginia	1932	*Type of fill *Trenching method	*Acidity *Flow of water
Pennsylvania	1947	NA	*Flow of water *Wooded area
Mississippi	1964	*Alignment offset *Faulty joints	*Clay flow upward and outward *Acidity
Utah	1974	NA	*Acidity *Soluble salt *Electric Resistivity
Ohio (ODOT)	1972, 1984, 1988	NA	*Acidity *Sediment depth *Presence of sediment *Flow depth *Flow velocity
Ohio (OSU)	1986, 1988	NA	*Flow depth *Flow velocity *Acidity *Sediment depth *Presence of sediment

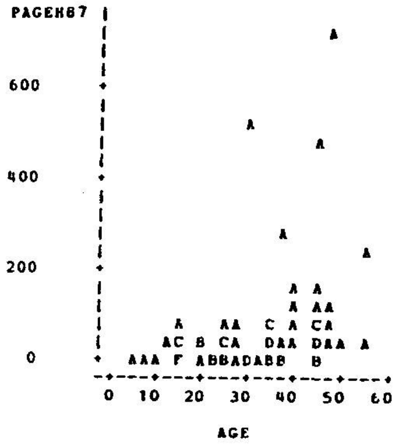

Fig. 3. Actual Age vs. Predicted age

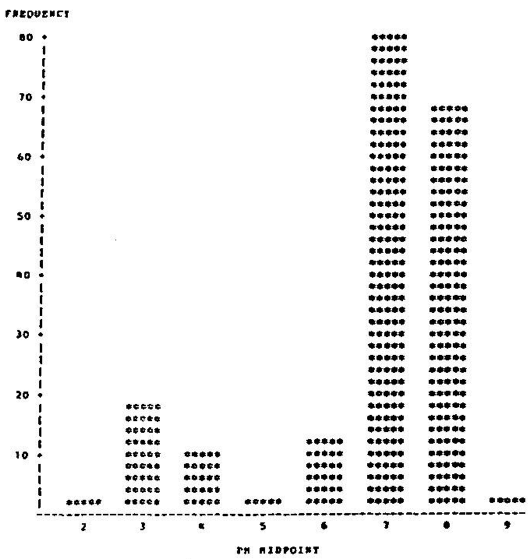

Figure 1. Binodal sample distr.

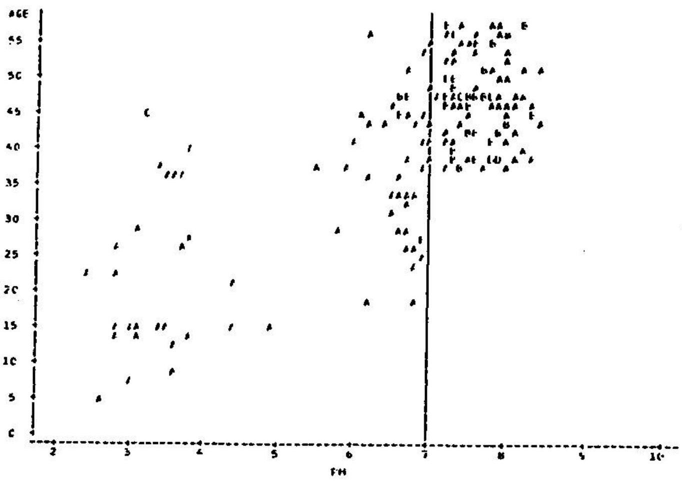

Fig. 2 . Boxing effect of sample data

predicted age (obtained using the prediction equation) and the actual age of the culverts [3]. Note that about 30% of the predicted ages are more than the oldest culverts in the sample. In the figure, the correct term of "predicted age" is used instead of the "predicted service life" of the culverts. Such an equation could erroneously predict up to nearly 3,500 years of concrete culvert service life.

4. CONCLUSIONS

The inadequacies of data and information concerning the culverts, in addition to the often conceptual problems involving the methods of treating the data, have resulted in the large variability of the predicted "service life" of the culverts. With the emergence of computers, culvert experts began to capitalize the use of regression techniques. However, the author feels that the results of recently applied regression techniques are not conclusive, if not unreliable. Problems had occurred in relation to the culvert data and the concept employed for analyzing the data. Several culvert data sample sets used for regresssion analyses are inadequate and not representative of culverts of a particular environment. Furthermore, these sample sets often represent culverts still in service while regression analyses were employed to generate prediction equations for determining the expiration of culverts' life.

The author of this paper feels that much has to be done if one expects reliable yet accurate results using regression techniques. Specifically, adequate amount of expired culvert samples from the same environmental conditions are required. Also, samples manufactured and installed in a relatively similar period of time (small range of age) are preferred. In addition, experimental accelerated testing of culverts installed in certain environmental conditions should also be considered.

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Design Approach with Respect to Durability

Approche de projet en vue d'une bonne durabilité

Bemessungsvorgehen zur Gewährleistung einer hohen Dauerhaftigkeit

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SUMMARY

The present report deals with a case study of realization of design specification with respect to durability on a medical and welfare complex for the aged. Various kinds of durability damage for the life span should be analyzed and synthesized to develop design specifications architecturally, structurally and environmentally which can be successfully accomplished at construction sites.

RÉSUMÉ

Ce rapport concerne le projet relatif à un complexe de soins hospitaliers, pour personnes agées, la conception étant soumise aux conditions imposés non seulement par la construction mais aussi par l'environnement, du point du vue de la durabilité et de la maintenance. Les dommages éventuels, tels que déformation des planchers, altération des revêtements de sol, neutralisation du béton, infiltration des parois, dilatation ou resserrement des joints, ont fait prévoir un ensemble de dispositions visant, au niveau même de la construction, à satisfaire aux tests de contrôle de la qualité.

ZUSAMMENFASSUNG

Dieser Beitrag behandelt eine Fallstudie für die Ausschreibungsbedingungen eines Seniorenheimes im Hinblick auf die Dauerhaftigkeit. Verschiedene Arten von Alterungsschädigungen sind abzuklären, um den Entwurf bezüglich architektonischen und ingenieurmässigen Anforderungen derart zu verbessern, dass die Dauerhaftigkeit durch die Ausführungskontrollen gewährleistet werden kann.

1. INTRODUCTION

Durability of buildings is related to various kinds of complicated characteristics including traditional construction technology and material, the climate of temperature, humidity, wind and sunshine quantity at the construction site, various type of loading, particularly due to natural causes such as earthquake or typhoon, and client attitude toward maintenance after completion. Thus, reinforced concrete buildings(RC) should be designed and constructed in terms of these factors architecturally, structurally and environmentally. Damages of concrete building due to durability over its life span in Japan should be divided into three categories of loading of earthquakes, of non-structural members such as finishment or facement appeared naturally or artificially and of serviceability including environmental equipments, change of occupancy and architectural deterioration. Particularly Japanese seems sensitive to this architectural deterioration to a large extent, which should be predicted at an early stage of project. Generally the climate of Japan has distinct four seasons with arid, cold winter and humid, hot summer. Traditionally people has enjoyed their life under beautiful cherry blossoms in spring, a hazy moon in autumn, even cicada sound of hot summer in rapport with the nature in which people would like to die after aging. Historically Japanese architecture as a shelter, built on soft soil layers or alluvium, has been reflected conformity to the nature, including, positively, these beautiful seasons and, negatively, strong earthquake or typhoon. Furthermore, because of dense urban area due to recent economic growth landownership demands more complicated restrictions on almost all projects. If traditional shelters, which is made of wood, could resist severe cold winter, life of remaining seasons should be comfortable with aid of breeze. After the war a large number of concrete shelters appear and not necessarily conform to the traditional Japanese mind architecturally and environmentally. Consequently, it is necessary to develop more sophisticated design and technology in conformity to the nature and human behavior with respect of durability of concrete shelter over its life span.

The present report deals with a case study of realization of design method of durability on a medical and welfare complex for the aged which locates outside Nagoya where it is hot and stick in summer, and cold and windy in winter and furthermore sometimes has heavy rain. Concrete buildings in the complex should resist disastrous loadings of strong earthquake and typhoon structurally and the natural climate environmentally and architecturally[1],[2].

2. PRESENT COMPLEX

This private medical and welfare complex for the aged holds 200 beds and consists of three RC wings, namely, a medical clinic, two nursing homes and a day-care center as shown in Fig. 1. Various kinds of restriction surrounding this project demand more sophisticated design review not only structurally but also environmentally or even architecturally particularly of durability closely correlated to maintenance for the life span. Prominent properties are as follows.

Location : a coastal city outside Nagoya with 300 thousand population,
surroundings : faced a national route of considerable traffic noise, vibration and air pollution,
on alluvium or soft soil layers,
climate : temperature of $-2°C \sim 32°C$,
humidity of 50% ~ 95%, occasional coastal wind and heavy rain,
facilities : a medical clinic (two story RC),
nursing homes (three story RC) and a day care center (one story RC),
including various equipments (solar panel, heating, cooling, sprinkler systems).

Hence, the design and construction should be accomplished against durability of these properties in harmony with the financial requirements.

3. DURABILITY DEFECT

This type of RC buildings, which is popular in Japan, are frequently built in dense urban area on alluvium certainly subjected to strong earthquake for their life span thus indispensably resulting the choice of appropriate piling foundation and anti- seismic walls in frame, which is controlled by regional structural regulation and code which can narrow diversity of durability with respect to safety against loading to satisfactory extent. Durability damages concerning to architectural details including cladding and finishment has diversity similarly to equipment system, which are closely related to maintenance by the clients for the life span of building. From a recent investigation[3] ill-conditions

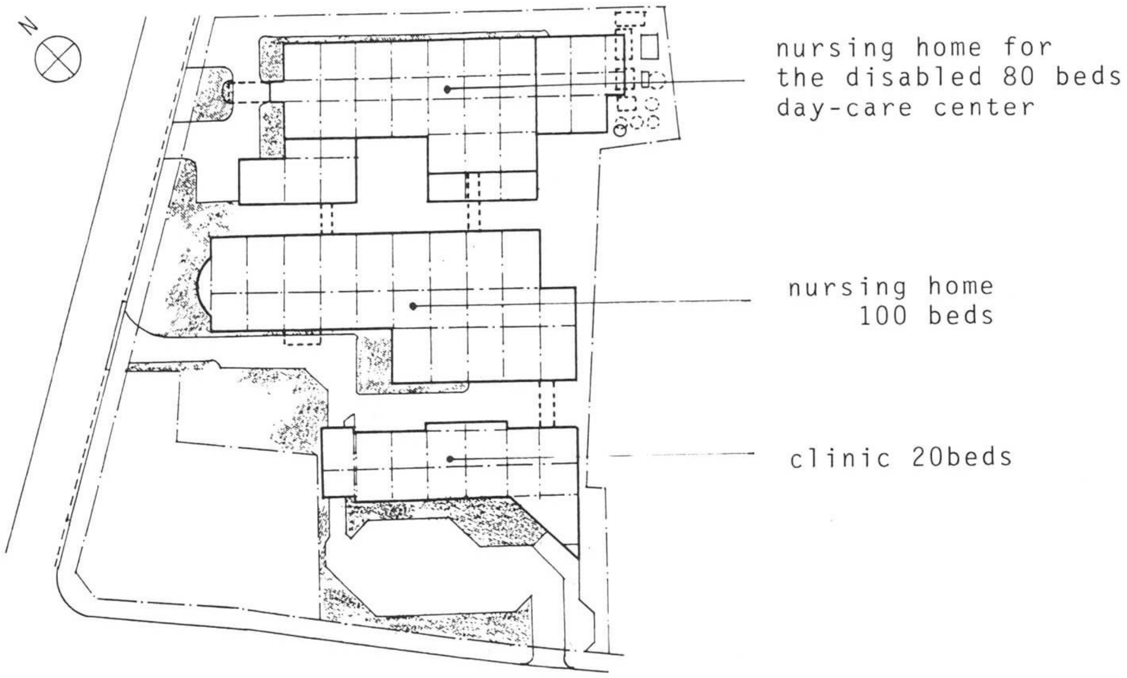

Fig. 1a Site plan of the complex

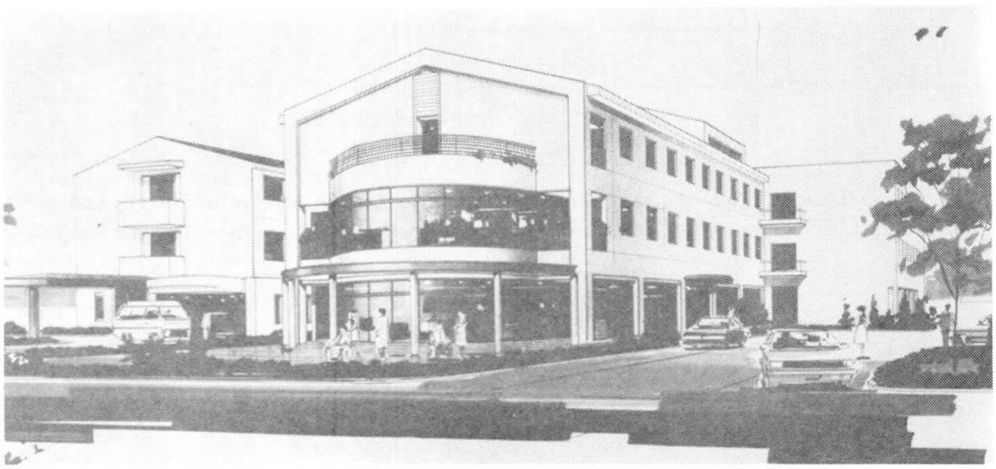

Fig. 1b Perspective of wings

due to a lack of durability can be classified in order of number of causes as follows.

ill-condition due to durability
- materials
- waters(rain, leak of equipment and pipe)
- strength of materials
- deterioration by aging
- form and size
- application
- noise and vibration
- temperature
- electricity and RI
- coloring
- miscellaneous

These ill-conditions can be extended into actual defects in detail which should be more practically evaluated at design specification and construction.

- defect
  - leaking
    - roof
      - water proofing materials
      - slope
      - flashing
      - number of roof drain
      - over-flow
      - pitched roof
    - exterior wall
      - intentional cracks
      - sealing materials
      - embeded pipe diameter $\leq \frac{1}{5} \times D$
      - coverage of pipe$\geq 30mm$
    - piping
      - leak of oil or gas
      - anti-errosion
  - dewing
    - roof
      - heat insulation, thickness
      - domain
    - exterior wall
      - heat insulation, thickness
      - domain
    - window
      - waste of dewing
      - heat insulation of frame
    - humid area
      - ventilation
      - air duct
      - heat insulation of pipe
  - wall-off
    - tiling
    - cornering
  - fire proof
    - spreading(grease trap)
    - ducting
  - settlement
    - porch slab
    - adjoining pipe
    - embeding

Furthermore, thus classified defects can be deployed more concretely from design and construction point of view. The following is an example of cracks which will be prevented as strictly as possible at each job stage.

- concrete cracking
  - design specification
    - exterior wall
      - intentional crack
      - wall thickness
      - reinforcement, a_t
      - diagonal reinforcement
      - opening crack
      - piping
    - roof, eaves
      - reinforcing
      - diagonal reinforcing
      - intentional crack
  - ready mixed concrete
    - mixture
      - slump
      - water cement ratio
    - material
      - normal
      - AE
  - concreting
    - amount per day
    - humid curing

As a result these categorical characteristics on durability are strongly correlated to each other which is, however, difficult to evaluate its priority precisely. When these characteristics are asserted as a database of AI technology, which consists of facts and rules, it becomes rather easy to search an optimal path to any durability goal by means of declarative languages such as Prolog which has versatility of backtracking manipulation. Herein, on an personal computer effective rules are asserted by Prolog.

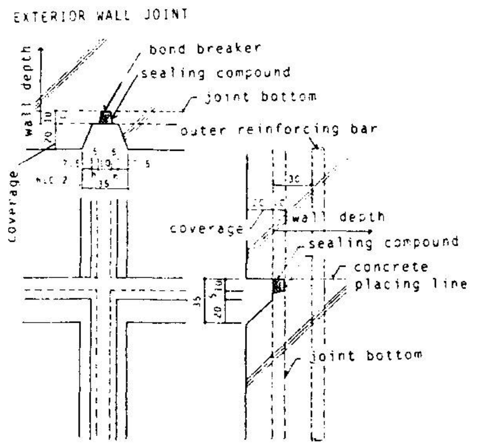

Fig. 2a Exterior wall joint

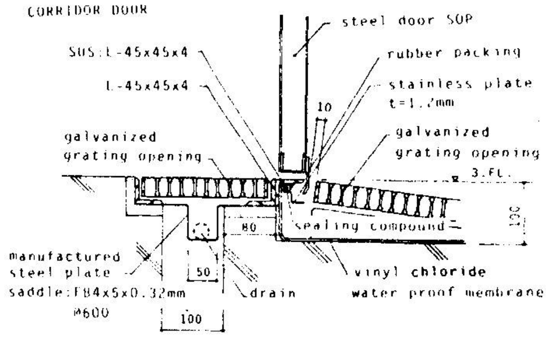

Fig. 2b Corridor door

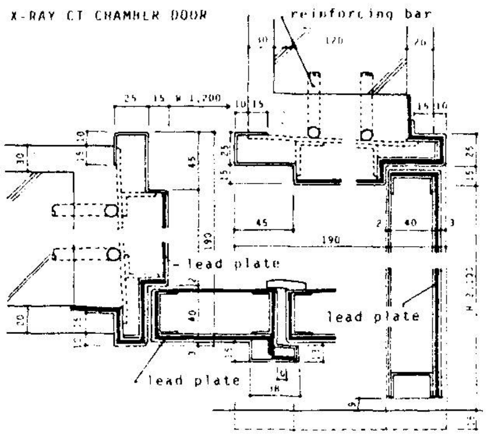

Fig. 2c X-ray CT chamber door

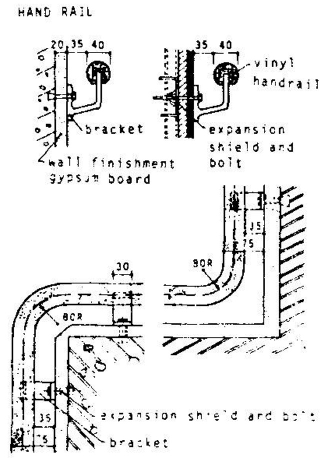

Fig. 2d Handrail

4. DESIGN SPECIFICATION

Although the occurrence of durability defects should be avoided as strongly as possible particularly after completion, an architecture should also original and beautiful itself. Thus creation of idea must be flexible with consideration of the restriction of durability. It is necessary to develop more rational details as the design specification to the construction site with reasonable investment. The design specification should be an understandable description of necessary information.

Fig. 2 shows several examples of details developed relating to durability defects. Furthermore, these details should be translated into field information for workers who will appreciate and realize by means of TQC.

5. REALIZATION BY TQC

From the investigation on durability of various RC buildings, damages concerned with construction process are due to the following reasons, namely, uncertain information of design specification to the construction field and a lack of appropriate construction technique. Particularly, on the latter there is a large technological diversity among workers and technicians at the site whose traditional skills become to decline as economical growth. It is necessary to develop a more progressed approach or an alternative to synthesize these unlevel skills to attain the prescribed durability tolerance.

To accomplish a certain level of quality control the traditional management or SQC (Statistical Quality Control) demands an inspection system, which tends to decrease productivity. Although a strict and large amount of inspection is accomplished there may appear oversights which causes durability defects particularly after completion. Hence, TQC (Total Quality Control) movement can be applied,

which means continuous quality control movement by QC circles at the site by all members particularly participated by workers who propose and devise various improvements on problems related to durability. TQC was originally developed in the industrial production process, and then prevailed extensively in the construction field successfully in Japan.
Concrete goals relevant to almost all kinds of field problems are discussed in QC groups including field workers and technicians at the construction site continuously. These group discussions are reflected to the construction process in progress. The construction industry of Japan has accepted this movement from the early seventies prevailing prominently nowadays. Presently the TQC is extended to even GWQC (Group Wide Quality Control). At the present construction site field workers including engineers accomplish the TQC movement to realize the durability tolerance specified by the prescribed design specifications. Many QC circles are established, each of which consists of several persons who belong to the same occupation. Each QC circle find an activity subject on durability from its surroundings, which is not necessarily sophisticated from the engineering point of view. Rather trivial problems to be improved at the site should be preferred, which QC circle participants analyze and discuss. The causes and results of subject are derived by means of interaction charts after discussion ordinarily for half an hour or less of several times on duty hours. This implies that a direct reflection of proposals by the field workers to concrete device of improvement can be accomplished. Naturally, these proposals are found through daily construction process and the results of their realization is compared to goal. This process is repeated until satisfactory level is attained. Thus, the realization of TQC is practically achieved for cost less than the required.

6. CONCLUDING REMARKS

Medium and low rise RC buildings, which are in the majority in Japan, reflect the social, economical situation with even sometimes chaotic contradictory results of durability demands including safety and deteriorations of architectural, environmental serviceability after completion. These durability demands should be realized and balanced not only engineeringly, architecturally but economically at each stage of design specification and then construction level. It is practical to make analysis and synthesis of durability by AI technology on a personal computer to arrange facts and rules with respect to damages. Thus obtained design specification is deployed at the construction site. However its reliable realization is not always easy because of unlevel skill of subcontract workers. TQC movement by these workers can ensure the realization at the site successfully and furthermore voluntarily. This process could be successfully applied to highly facilitied RC buildings of a medical and welfare complex for the aged to guarantee of balanced durability for life span.
Finally, the authors would like to thank Shimizu Corp. on this study.

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Inspection Evaluation, and Rehabilitation of Suspension Bridge Cables

Inspection évaluation et remise en état des câbles de ponts suspendus

Inspektion Beurteilung und Instandsetzung der Kabel von Hängebrücken

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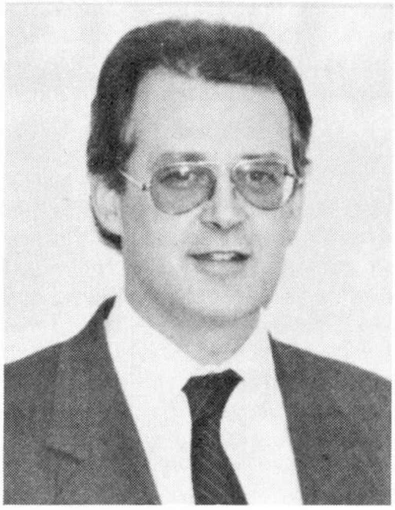

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SUMMARY

Recent inspections of the main suspension cables of the Williamsburg, Brooklyn (New York, NY), and I-74 (Bettendorf, Iowa) Bridges provide a sampling of the longevity and durability of three differing types of cable construction. The cables of each bridge will be rehabilitated in a manner dictated by the details of the original design.

RÉSUMÉ

Les récentes inspections des câbles principaux des ponts de Williamsburg et Brooklyn (New York, NY) et de la I-74 (Bettendorf, Iowa) fournissent un échantillon de longévité et durabilité de trois types de ponts suspendus différents. Les câbles de chaque pont vont être remis en état en respectant les détails des conceptions originales.

ZUSAMMENFASSUNG

Kürzliche Inspektionen der Hauptkabel der Brücken von Williamsburg und Brooklyn (New York, NY) und der I-74 (Bettendorf, Iowa) stellen eine Stichprobe für die Dauerhaftigkeit und das Langzeitverhalten dreier verschiedener Typen von Hängebrücken dar. Die Kabel jeder Brücke werden in einer durch die Einzelheiten des ursprünglichen Entwurfs bestimmten Art und Weise instandgesetzt.

1. THE THREE BRIDGES AND THEIR CABLES

1.1 Brooklyn Bridge

Brooklyn Bridge spans New York's East River, connecting Manhattan with Brooklyn Heights. It is the most famous of John A. Roebling's suspension bridges, completed in 1883 and now a national monument. The main suspended span is 486 m long and is flanked by two suspended side spans, of 283 m each. Each of its four main cables is comprised of 5358 galvanized steel wires, 4.67 mm in diameter. After spinning, the cables were compacted, coated with a thick paste of white or red lead, and then tightly wrapped with 3.8 mm diameter galvanized steel wire. This was then sealed with several coats of paint. The wrapped cables are 400 mm in diameter.

1.2 The I-74 Bridges

The I-74 Bridges are twin, nearly identical suspension bridges crossing the Mississippi River between Bettendorf, Iowa, and Moline, Illinois. The original bridge, now serving Iowa-bound traffic, was completed in 1935, and the newer structure now serving Illinois-bound traffic, was completed in 1959. Both have main suspended spans of 226 m, and suspended side spans of 113 m. Each bridge is supported on two cables, 241 mm in diameter, including wrapping wire. These cables are of a construction that markedly differs from the Brooklyn Bridge. Rather than building up the cross section with thousands of individual wires, these cables consist of 37 shop-fabricated galvanized structural strands, 31 of 38.1 mm diameter, and 6 of 25.4 mm diameter. Each strand contains 19 or 37 wires, twisted in layers. The strands are laid parallel to each other, roughly forming a circular cross section. Extruded aluminum fillers are added around the circumference to round out the surface, and the whole is tightly wrapped with 3.8 mm diameter galvanized wire, and covered with three coats of paint.

2. INSPECTION SAMPLING, AND TESTING

2.1 Brooklyn Bridge

Inspection of the Brooklyn Bridge cables was performed in two distinct parts: Wrapped areas between the anchorages; and the unwrapped individual strands within the anchorage chambers. The four cables were visually inspected end to end and based on conditions observed, an in-depth investigation was programmed.

2.1.1 Wrapped Portions

Two sections of the cable in the main span and one section in the Manhattan side span were unwrapped and split open with oak wedges. Because cable bands are located every 2.3 meters, it was first necessary to remove the suspender and cable band (or cable post) to provide enough free cable (about 7 meters) to penetrate with the wedges. It was found that the wrapping wire beneath the cable bands had begun to corrode significantly due to the tendency for moisture to lay between the ribbed surface of the wrapping and the smooth inner surface of the cable bands. The outer layer of main cable wire immediately below the wrapping and cable bands had corroded sufficiently to consume most of the zinc galvanizing and rusting of the wires had occurred. One wire had lost over 75 percent of its cross section and had broken. The remainder exhibited localized loss of material that is commonly referred to as "pitting" although in the strict technical definition no real pits were present (the technical definition of a pit is a defect that is at least as deep as its width at the surface). At the areas between cable bands, the surface wires were still in excellent condition. Dried red lead paste covered most of the surface, but in occasional spots powdery zinc oxide from the galvanizing coated the exposed wires.

Oak wedges were driven between the wires at four radial points (one point at a time) and the cable was penetrated approximately 15 cm, or almost to its center. It was found, even under the cable bands, that all corrosion had been limited to the outside layer of wires. From the second layer in, the original galvanizing was still in near-perfect condition.

Fifty seven wire samples were cut from the cables at various locations and sent to Columbia University's Carleton laboratory for testing. The cut wires were replaced with lengths of new wire spliced in using a combination of pressed-on and threaded ferrules.

The results from the laboratory indicated that the typical corroded wires had not lost measurable strength as compared to uncorroded wires, but it was apparent that the original material was not uniform and was of considerably lesser quality than modern bridge wire. Carbon content varied from 0.55 percent to 0.91 percent, whereas modern wire is generally in the range of 0.78 to 0.82 percent. Average tensile strength was 1,100 MPa, and the proportional limit was approximately 690 MPa. The most significant finding was that the original wire was of low ductility: reduction of area varied from practically zero to 26.5 percent; reduction of area for modern galvanized bridge wire will typically be on the order of 35%.

Fatigue testing results and microscopic examination of longitudinally sectioned wires provided the assurance that there was no evidence of stress corrosion cracking in the wires.

2.1.2 Cable at Anchorages

The initial inspection on the anchorages revealed serious corrosion and numerous broken wires were found both between the splay band and strand shoes, as well as at the back of the shoes, where concrete had been placed in contact with the wires. The confined space of the anchor chambers provided barely enough room for a man to pass between the strands and the chamber walls. It was impossible to determine the full extent of corrosion damage at the shoes or within close proximity to the splay band. It was apparent that extraordinary measures would be necessary in order to fully examine and evaluate the conditions at the anchorages. The possibility existed that entire strands were damaged beyond repair, and it was therefore decided to develop procedures and equipment to splice entire strands concurrently with developing a detailed program to continue the investigation.

First, the anchor chambers would need to be enlarged to provide working space; second, the existing splay bands would need to be removed to allow spreading of the strands for inspection at the splay points, and third; repair details for various possible conditions needed to be developed. With the assistance of the faculty and staff at Columbia University's Carleton Laboratory, a mock-up of a typical anchor chamber and splayed cable was constructed. Clamps, sockets, and jacking equipment were designed for the worst case scenario, in which entire strands would need to be cut, socketed, and reanchored in the field, something that had never been done before. New splay bands and strand spreader frames were designed.

Working in the mock-up, methods were tested for zinc-socketing of the strands in the horizontal, rather than the usual vertical position.

A contractor was awarded the contract to assist Steinman in enlarging the chambers, relocating the cable splay points, and if need be, cutting and replacing the deteriorated strands. It was decided to start at Cable B in the Brooklyn Anchorage. After the chamber had been modified, two temporary splay bands were installed spanward of the existing band, and the old band was removed. In a series of "leap frog" moves, the two temporary bands were moved up along the cable until they reached the location of a new permanent splay band about 4 meters from the original splay. Horizontal and vertical spreader frames were placed between the strands and the strands were gradually spread apart using specially designed hydraulic equipment. The placement of the new splay was such that the total length of each strand would be unchanged after the splay relocation was completed. Electronic strain gauges were used to monitor stresses in the anchor eyebars as the strands were spread.

Upon completion of the strand spreading and removal of concrete behind the strand shoes, a detailed inspection was made strand by strand. It was considered very fortunate by all involved that the serious corrosion and breaks were primarily confined to surface wires. Most of the wires were slightly corroded, or uncorroded, although much of the galvanizing zinc had been consumed by oxidation. The operations proceeded in the remaining seven anchorages and similar conditions were found. All in all, several hundred broken or seriously corroded wires were found, but these were repairable by splicing in new sections of wire, and no full strands needed to be replaced.

This work was completed in early 1987.

2.2 I-74 Bridges

Our assignment on the I-74 Bridges was to perform a close condition inspection of all cables and the superstructures of both bridges. The cables to be inspected included all handropes, main cables and all suspenders of both bridges.

2.2.1 Suspenders

All of the suspender ropes were inspected and as anticipated in bridges of this design with no center tie, one-way traffic, and increasingly heavy truck traffic, many of the shorter suspenders near mid span of both bridges contained cracked or broken wires. The mechanism which causes these breaks are fatigue related and caused by the differential longitudinal motion under live load of the main cable with respect to the top chord of the stiffening truss. The measured motion of 19 mm though relatively small, is sufficient to alternatively stress the suspender wires near the extremity of the suspender. The cyclical loading causes the wires to break over a period of time and the number of breaks occurring increases rapidly once the critical number of live load cycles has been reached. It has been recommended and accepted that center ties be installed on both bridges and that all suspenders with cracked wires be replaced.

2.2.2 Main Cables

The main cables of both bridges were inspected from anchorage to anchorage and included all associated cable bands, saddles and appurtenances.

The anchorages were inspected first and found to be in very good condition. No deterioration, rust blooms, moisture or staining were found on the splay saddles, splay strands, strand shoes or eyebar assemblies and no cracks or water were found in the chambers themselves.

The next step was to inspect the main cables between anchorages to determine if water had entered the cables and if so what damage had occurred as a result. Here we found that the paint on the cable wrapping was in poor condition with many cracks in the paint layers, some loss of galvanization and some rust on the wrapping wire. In addition, we found pop-outs of the bottom caulking and staining had occurred at many cable bands which is a sure sign that water under pressure or water under freezing conditions had been present in the cable. At random we removed some cable band bolts and found that no bolts from the top of any band was rusted, however, all bolts removed from the bottom of the bands were deteriorated with some section loss and the bolt chambers were filled with rust. Some of the rust and some of the bolts were wet. Next we removed some of the sealed covers from the tower saddles and the cable bent saddles to see if any leakage was occurring at these points. We observed that the sealed covers were watertight and that the cables across the saddles were in like new condition. The covers were replaced and resealed. Based upon our findings that water was entering the cable thru the many small openings in the paint protection at the wrapping wire it was decided to unwrap the main cables at various points between cable bands for closer inspection. The areas to be unwrapped were designated by us and were chosen to reveal varying types of cable exposure to water. Total length of unwrapping was 83 meters (horizontal projection) on the 4 main cables.

The unwrapping procedure began by first having the contractor install the work platform on both sides of the cable for the full length of the panel to be worked on.

To start the unwrapping, the wrapping wire was cut with a small electric grinder at a designated point where damage to the main bridge strands would not occur. Once the initial cut was made, the wrapping was unwound by hand, cut into pieces using a wire cutter and removed. Tie wire was placed around the cable to hold the aluminum fillers in place. With the wrapping wire removed, the strands and fillers were exposed.

Each of the 18 aluminum fillers was marked with an identification number as it was removed from the cable and placed in a special box made to hold the fillers. These fillers were then taken to the plaza area on the bridge where they were wire brushed clean and painted with neoprene paint.

With the aluminum fillers removed, the strands were inspected. Hard wood wedges driven between the strands along with a hydraulic wedge were used to spread the strands apart. A putty knife, steel awl, magnifying glass, flashlight, and a fiber optic device were used to inspect the strands. All of the strands, except where noted, were covered with a maroon colored paint. The paint had been applied before the strands were placed on the bridge.

In some locations, the strands were found to have white oxide formation along with, or sometimes without, deposits of ferrous rust. In cases where the ferrous rust was at a more advanced stage, "pitting" typical local corrosion was found. In order to determine if the individual wires had any defects such as cracks, the wires were carefully cleaned using a stiff bristle brush and a cleaner solvent to remove as much white oxide and ferrous rust deposit as possible. Once the wire was sufficiently clean, the wires were carefully examined by an engineer using a magnifying glass. No cases of visible wire cracking were found. Since the cables are composed of twisted wire strand, it was undesirable to remove wire samples for labratory testing, and the conditions found did not warrant further investigation.

Since the extent of damage was generally limited to localized areas, the condition was discovered in time to save the cables by applying a new protective wrapping of neoprene to preclude the entrance of water.

3. CABLE REHABILITATION PROGRAMS

3.1 Brooklyn Bridge

There were several factors that influenced the scope of rehabilitation required for the cables of the Brooklyn Bridge. In contrast to present practice, the cables had been wire wrapped continuously prior to installation of the cable bands and suspenders. While the original wrapping had admirably protected the cables throughout most of their length, the cable bands were trapping moisture and corrosion was taking place beneath them.

The bands were also prone to slipping down hill on the cable because their design precluded a secure clamping effect. The original red lead paste was dried out and did not afford the necessary sealing between the wrapping wires. It was also found during our general inspection that virtually all of the wire rope suspenders were seriously corroded near their lower sockets, and would need to be replaced, as would the diagonal wire rope stays.

The cable rehabilitation contract, designed and inspected by Steinman, includes the removal of all existing wire wrapping, cable bands, suspenders and stays. New bands of modern design and new suspenders will be installed, followed by the rewrapping and painting of the cables. The greater part of the cables will be rewrapped with galvanized wire bedded in a thick paste of red lead. At the sag points, where the cables pass below the roadway and are subject to splash by runoff water and deicing salts, the cables will be wrapped with 3mm thick neoprene wrapping material.

All eight anchorage chambers have been enlarged and now provide ample space for inspection and maintenance of the strands, which will remain in their newly spread configuration. All broken and badly corroded wires have been replaced with sections of new wire, using a technique developed by Steinman. Since past experience has shown that the cutting of threads on the existing wires is difficult and uncertain, a length of damaged wire is first cut out and to its two ends are spliced new wires using specially designed ferrules. These ferrules consist of a mild steel cylinder, approximately 10mm in outside diameter, bored to accept a hardened helical steel wire insert of slightly larger inside diameter than the original bridge wires. After inserting the end of one old and one new wire into the ferrule, a hydraulic press crimps the ferrule onto the wire, developing a splice that is 95 percent as strong as the original wire. The two mating ends of the new wires which have shop-cut threads, are then spliced using a ferrule with internal left and right hand threads. The threaded ferrules act like turnbuckles and permit the spliced wires to be stressed to a predetermined tension. After repair of the damaged wires, the cables are now more than adequate to carry the dead and live load of the bridge.

It is fully expected that the rehabilitated Brooklyn Bridge will serve New York City for at least another one hundred years.

3.2 I-74 Bridges

Our recommendations for protection of the cables was as follows:

Clean the loose paint from the cable exterior by wire brushing and wrap neoprene flashing completely around the cable along its entire length over the existing wrapping wire. The exposed neoprene wrapping surface should be painted for protection. The lead wool packing between the wrapping wire and the cable band will need to be removed and replaced with caulking in order to ensure a watertight seal.

It is not considered necessary to recommend complete removal of the wrapping wire because both the main cable and the wrapping wires are in relatively good condition. The major problem is that the lead-based paint on the exterior of the wrapping wire has failed, permitting water to enter the cables between the wrapping wires, and similar to other cables of this design, no red lead paste had been applied under the wrapping wire. Neoprene wrapping has been used on suspension bridges for the past 16 years with very good results. Neoprene wrapping placed over the original protection represents the best means of positively waterproofing the cables.

The cable bands should be reconditioned by removing the caulking from the bottom slot separating the cable band halves in order to permit any water which may leak into the cable to drain rather than being retained. It is also recommended that the lower cable band bolts be replaced with new ones and that the loose rust inside the cable band bolt housings be removed. This loose rust tends to retain moisture which is detrimental to the cable strands. A program of cable band bolt retensioning should also be performed at the same time.

4. CONCLUSION

These bridges can give the Engineering profession valuable insight into the longevity of different suspension bridge cable designs. Most notably, it can be seen that the Brooklyn Bridge, with its galvanized wires, red lead paste coating, and galvanized wrapping wire, performed almost perfectly for 100 years. The I-74 bridges, with galvanized strands, galvanized wrapping wire, but no paste sealer under the wrapping wire, will need relatively minor cable rehabilitation work after 50 years in service.

Inspection Program for the Lisbon Suspension Bridge

Programme d'inspection pour le pont suspendu de Lisbonne

Untersuchungsprogramm für die Hängebrücke in Lissabon

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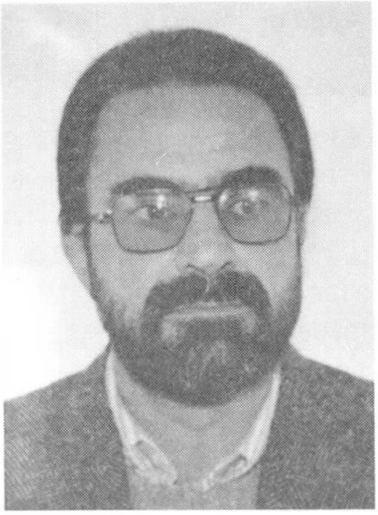
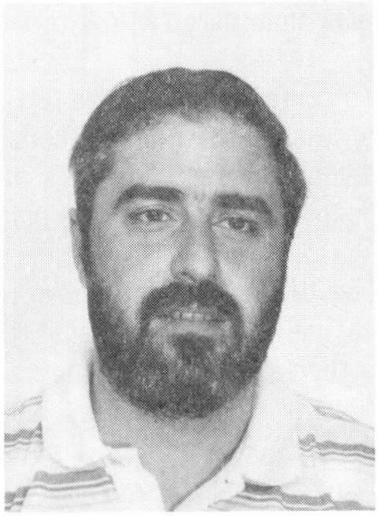

SUMMARY

The guidelines of the inspection program for the suspension bridge in Lisbon are presented. The program considered an overall inspection for corrosion and the analysis of the structural behaviour of several elements. The principal findings and the inspection techniques are presented.

RÉSUMÉ

On présente les lignes directrices d'un programme d'inspection pour le pont suspendu à Lisbonne. Ce programme consiste en une inspection générale de la corrosion et de l'étude du comportement de quelques éléments structuraux. Les résultats principaux et les techniques d'inspection sont présentés.

ZUSAMMENFASSUNG

Die Richtlinien eines Untersuchungsprogrammes für die Hängebrücke in Lissabon werden dargestellt. Dieses Programm besteht aus einer generellen Korrosionsuntersuchung und aus der Analyse des Verhaltens einiger Tragelemente. Die wichtigsten Ergebnisse und Untersuchungsverfahren werden beschrieben.

1. INTRODUCTION

The " 25th of April" bridge in Lisbon is one of the longest suspension bridges in the world with a total length of 2276m and a central span of 1013m. An observation program to study its structural behaviour was developed and implemented during its construction and carried on for several years after the bridge opening, in 1966.

Recently the Portuguese Highway Authority (J.A.E.) decided to widen the deck to carry six traffic lanes (presently it has 4 lanes). Simultaneously, studies are also being developed to introduce the railway traffic in the lower part of the deck, solution that was considered in the initial design of the bridge.

To come up with these projects, the implementation of a detailed inspection program of the structure was considered a priority task by the J.A.E. As a matter of fact during the last 23 years miner routine inspections have been undertaken, but now a deep assessment of the bridge was felt necessary.

Due to the unusual characteristics of this type of job a research was developed to define the guidelines of the inspection program. This is presented in this paper referring the main aspects to be considered in the inspection of the anchor blocks, towers, main cables, hangers, truss beam, joints and bearings. Based on the inspection, the main findings are also referred as well as some particular problems faced during the works.

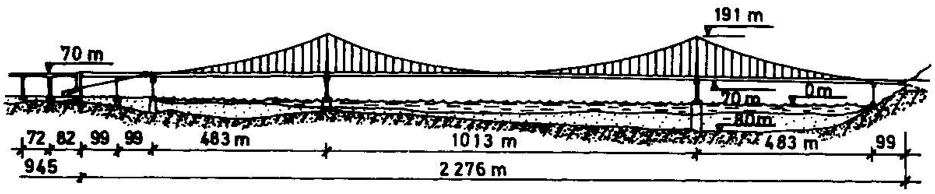

Fig. 1 - LISBON SUSPENSION BRIDGE.

2. THE SUSPENSION BRIDGE

The "25th of April" bridge is a steel structure with a total length of 2276m connected with a concrete approach viaduct (north side) with 945m. The steel structure includes the suspended structure with a 1013m central span and two 483m lateral spans, and a supported continuous stiffened truss with a south end span (99m) and two north end spans (2x99m) (Fig. 1).

There are seven supports along the bridge: the two abutments (P_1, P_7), the two main towers (P_3 , P_4), an intermediate column in the south side (P_2) and two other in the north side (P_5, P_6). The suspension structure is defined by supports P2 and P5 at which the main cable passes below the deck level. These columns are fixed at the base and slip free for the deck, at the top. The main towers are 191m high and are fixed at the base in concrete caissons which found 80m below water level.

The truss beam is, 10,6m high and 21m wide, suspended from the hangers (23m apart) about 70m above water level. Fig. 2 shows the actual cross section and the future situation with the train. The actual widening design considers the space between the border of the upper deck and the hangers.

The main cables have a diameter of Ø = 0,586 m and are made of 11 248 steel wire (Ø = 5mm - f_{yu}= 1560 N/mm^2). During construction the wires were tied in groups of 304 units, then compacted and tied with helicoidal wire and finally painted with anticorrosive paint. The hangers are connected to the main cables with two shell clamps tied with high strength bolts.

The structure was built with several types of steel. The connections between steel members were mainly done with high strength bolts.

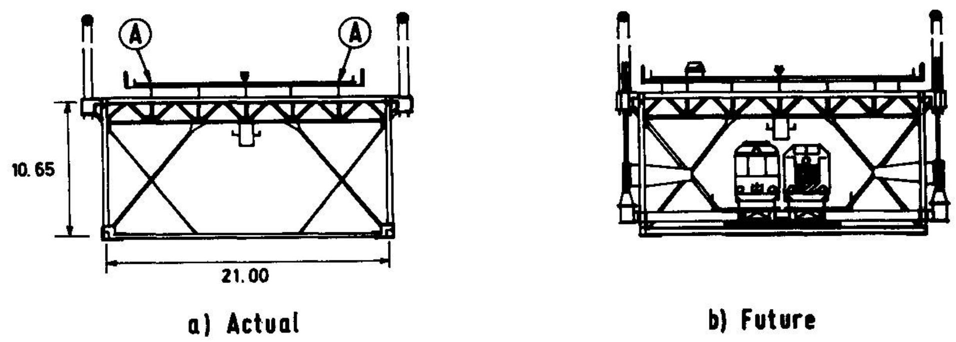

Fig. 2 - DECK CROSS SECTION.

3. THE INSPECTION PROGRAM GUIDELINES

The two main objectives of this program are:

- To perform an overall inspection for corrosion problems;
- To check the structural behaviour of the bridge elements.

The study began with a preliminary research about suspension bridge inspection techniques [1,2]. Next the program was developed considering the guidelines for the inspection of each main structural element. These are briefly referred.

3.1 Anchor Blocks

Inside the concrete cameras, where the cable anchors are placed, usually high humidity may occur. This may be associated to corrosion problems what leads to a careful inspection of cables and anchorage. The main structural problem associated with these elements is its movement along the time, what may lead to the collapse of the bridge. Topographic periodical inspections are highly recommended.

3.2 Columns

The steel bridge columns and the towers need a general inspection for corrosion. Special attention should be paid to:

- The base of the columns where the highest forces usually occur;
- The transverse cross beam in the top of the towers where unusual forces may occur due to asymmetrical loading;
- The connections of the cables to the top of the columns, to check for eventual slips.

3.3 Main Cables

The problems that may occur in the cables are the corrosion and cracking of the wires. A visual external inspection should be done along the cable with particular attention to the anchor zones and the lower part of the parabola where the water from rain converges and accumulates.

For the internal inspection of the cables there are presently two electromagnetic techniques, available [4]: The Foucault Current Method and the Induced Current Method. The first one measures the oscillations in the magnetic field of a solenoid placed around the cable, which is proportional to the oxidation of the wires. In the second method one measures the induced current that arises in an alternative magnetic field placed around the cable, due to its imperfection. This technique was developed to find cracked wires which are shown by tension peaks in the induced tensions.

3.4 Hangers and Cable Bands

The hangers corrosion is also an important problem, especially in the lower connection, but a visual inspection is usually sufficient. The structural problems associated with the hangers are the following:

- Inclination - inclined cables are associated with slips in the clamps;
- Hangers Tension - when connection problems occur, the tension may decrease. This can be checked measuring its vibrations frequency which is proportional to the tension [3];
- Longitudinal Profile of the Deck - irregular profiles of the deck are usually associated with suspension problems;
- Cable Bands - should be visually checked to detect slips. To check the clamping forces two methods are now under research: the ultrasonic measurement and the use of a hydraulic "bell" system that tensions the bolt from the nut side [1].

3.5 Deck Truss Beam

In this element the inspection should consider the corrosion and the local problems. Regarding corrosion, attention should be paid to water retaining details, zones near water gutters and closed sections where the internal inspection is also necessary. The local problems are associated with car damages in the structure, fatigue problems in welded connections (especially at midspan and support sections), and cracked or loose bolts in the connections.

3.6 Supports and Joints

The inspection of these elements should consider essentially their cleaning to guarantee a good behaviour. This can be observed during a day, checking the temperature movements. Eventual cracks in the supports may be detected by unusual noises under traffic.

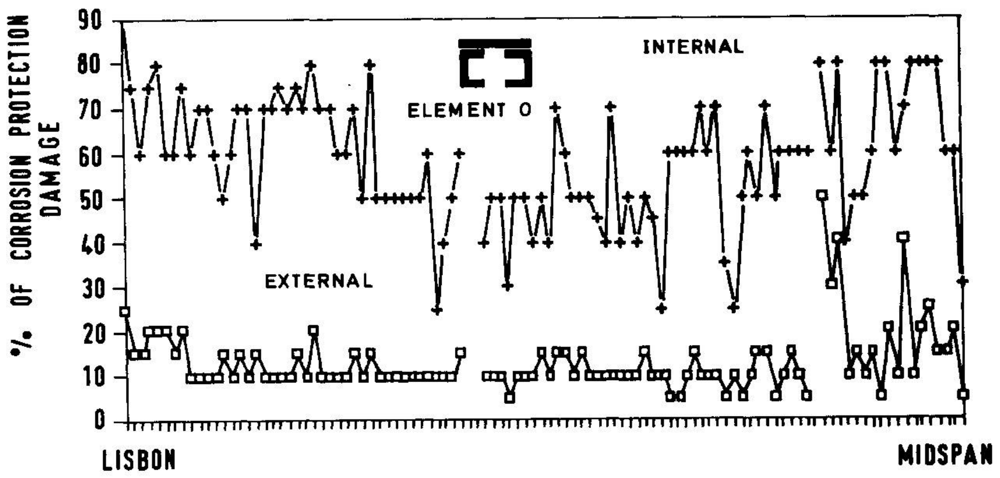

Fig. 3 - ANTICORROSION PROTECTION DAMAGE AT ELEMENT 0, ALONG THE SPAN.

4. THE INSPECTION RESULTS

At the first stage of the program, the corrosion inspection was implemented. At the columns only superficial corrosion was found, mostly at the interior of the base section. The concrete caissons were also observed and filmed under water to check for concrete degradation, but no relevant findings were obtained. At the cables hangers and cable bands, and with an external observation, only small spots of superficial corrosion were found. Due to the good external condition, an internal observation with electromagnetic techniques was not considered necessary at this stage. All the bars of the deck truss were also inspected and plots of the superficial corrosion along the bridge were drawn for each element (Fig. 3). Structural damage by corrosion was found only in a few nuts of the bolted connections which presented section reductions of more then 20% . It is estimated that about 2% of the deck bolts will need to be replaced shortly. During deck inspection some transversal bars located over the longitudinal girders (shown in Fig. 2 by A). It is estimated that 13% of the total number present this problems.

The second phase of the program considering the inspection of structural behaviour of several elements is now under development. The first problem is the rechecking of the tension on the cable bands bolts. The results from the ultrasonic measurement and the hydraulic bell system are being checked with those from a prototype to come up with an easy and reliable technique. To obtain the hangers tension, a cable prototype is also under research to calibrate the vibration method, considering different lengths of cables. Also checked were the hand rails cables, over the main cables. Their forces were measured to analyse their capacity to carry the loads of the future inspection system.

4. ACKNOWLEDGEMENTS

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Schäden und Sanierung von Brückentragseilen

Damage and Repair of Bridges Track Ropes

Défauts et réparations de câbles porteurs de ponts

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ZUSAMMENFASSUNG

Im Bereich von Verankerungen, Aufliegestellen und Hängerklemmen treten, infolge von Seilschwingungen und Ermüdungskorrosion, manchmal schon nach kurzer Betriebszeit, Drahtbrüche in der Aussenlage von verschlossenen Tragseilen auf. Durch Einpressen von neuen Z—Drähten und durch Löten können Drahtbrüche in aufliegenden Tragseilen, auch unter voller Betriebsbelastung, saniert werden. Hohe Kosten, die für den Austausch der schadheften Tragseile notwendig wären, sind dadurch vermeidbar.

SUMMARY

In the regions of anchorages, saddles and cable-clamps wire breaks can occur even after short periods of operation due to rope oscillation and fatigue corrosion. It is, however possible to repair these damages under full load of operation by pressing in new Z — wires and soldering. Thus the high costs of replacing damaged track ropes will be avoided.

RÉSUMÉ

Au droit des ancrages, des supports et des serres-câbles apparaissent parfois, même après une court période d'exploitation, des ruptures de fils dans la couche extérieure des câbles porteurs clos. La cause de ces ruptures est liée aux oscillations du câble et à une fatigue due à la corrosion. En serrant de nouveaux fils, du type z, et en les soudant, on peut réparer des ruptures de câble, même sous pleine charge d'exploitation. De cette manière, on peut éviter de grands frais pour l'échange de câbles porteurs défectueux.

1. EINLEITUNG

Infolge von Seilschwingungen, manchmal auch im Zusammenhang mit Ermüdungskorrosion, treten des öfteren, insbesondere in der Außenlage von verschlossenen Tragseilen, nach verhältnismäßig kurzer Betriebszeit, Drahtbrüche auf. Diese befinden sich bevorzugt im Bereich von Verankerungen und Aufliegestellen oder im Bereich von Hängerklemmen. (Fig. 1)

Treten die Drahtbrüche vereinzelt auf, so genügt eine Sanierung der klaffenden Bruchenden durch Plombieren mittels Kunstharz. Dadurch wird das Eindringen von Feuchtigkeit verhindert. Tritt Drahtbruchhäufung auf oder befinden sich die Bruchstellen in benachbarten Z-Drähten, besteht die Gefahr, daß Z-Drähte aus dem Seil treten und sich auf weite Strecken aus dem Verband schälen. Um dies zu verhindern, sind im Bereich der Schadensstelle Schraubklemmen zu montieren. (Fig. 2)

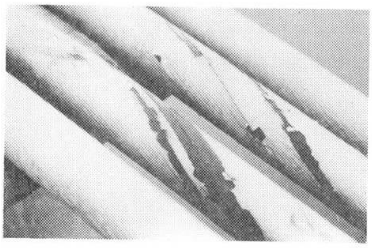

Fig.1 Drahtbruchhäufung in verschlossenen Tragseilen einer Hängebrücke im Bereich des Pylonsattels

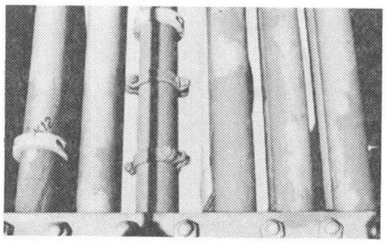

Fig.2 Drahtbruchhäufung in verschlossenen Tragseilen einer Hängebrücke im Bereich einer Hängerklemme

Tritt Drahtbruchhäufung im Bereich des Vergußkopfes auf, besteht vorerst keine Gefahr, daß die gebrochenen Z-Drähte aus dem Seilverband treten; die Krafteinleitung in den Vergußkopf wird jedoch so gestört, daß die weitere Zunahme der Drahtbrüche progressiv erfolgt.

Die Beurteilung von Drahtbrüchen in Brückenseilen bezüglich einer Sanierung erfolgt nach mehreren Gesichtspunkten. Es ist zu klären:

a) Ob der Seilschaden durch eine temporäre, örtliche Überbeanspruchung verursacht wurde, die durch konstruktive Maßnahmen oder durch eine entsprechende Wartung behoben werden kann.

b) Ob der Seilschaden durch Gewalteinwirkung entstanden ist und

c) Ob der Schaden bereits auf einen so hohen Dauerfestigkeitsverlust der Drähte zurückzuführen ist, daß die Tragseile innerhalb einer kurzen Zeitspanne den Zustand der Ablegereife erreichen.

Im letzten Fall ist eine Sanierung des Seilschadens nicht statthaft. Handelt es sich hingegen um einen örtlichen Seilschaden mit bekannter Ursache und entspricht der Allgemeinzustand der Tragseile, in bezug auf Seil- und Betriebsicherheit, den einschlägigen Bestimmungen, so ist eine Sanierung zulässig. Die Sanierung der Tragseile ist, wenn dies möglich, einem Austausch vorzuziehen, da die Erneuerung der Tragseile nicht nur mit hohen Kosten verbunden ist, sondern auch den Verkehr auf längere Zeit beeinträchtigt.

2. SANIERUNG EINES VERSCHLOSSENEN TRAGSEILES IM BELASTETEN ZUSTAND IN DER SEILRECKANLAGE

Die Sanierung von Drahtbrüchen in verschlossenen Tragseilen wird schon seit Jahrzehnten praktiziert. Insbesondere müssen Tragseile von Personenseilbahnen nach Gewalteinwirkung, z.B. nach Blitzeinschlägen, saniert werden. Zu diesem Zwecke wurden früher die Tragseile entspannt und die Sanierung der gebrochenen Z-Drähte im unbelasteten Zustand vorgenommen. Im letzten Jahrzehnt wurden auch aufliegende Tragseile im belasteten Zustand mit nachhaltigem Erfolg saniert. Auf diese Weise konnten erhebliche Kosten, vor allem aber auch Zeit, eingespart werden. [1] [2] [3]

Tragseile von Hängebrücken, die gebündelt oder gefächert angeordnet sind, können nicht einzeln entspannt werden. Aus diesem Grunde ist die Sanierung einzelner, schadhafter Tragseile unter Betriebsbelastung oder unter jener Belastung vorzunehmen, die bei unbelasteter Brücke gegeben ist.

In der Seilreckanlage der AUSTRIA DRAHT Ges.m.b.H., kann die Sanierung bei jeder beliebigen Belastung, auch entsprechend den Bedingungen im Bauwerk, simuliert werden. Die dabei zu messenden Werte hinsichtlich des Drahtes, wie Wegstrecke zur Beseitigung der Drahtüberlänge nach dem Löten und hinsichtlich des Tragseiles, wie Belastungs-Dehnungs-Diagramm und Elastizitätsmodul, geben Aufschluß über die erforderliche Sanierungslänge und über das Verhalten des sanierten Tragseiles unter Betriebsbedingungen.

2.1 Vorbereitung des Versuches

Um das zehn Meter lange verschlossene Tragseil, 63 mm Durchmesser, belasten zu können, wurde es an beiden Enden mit Vergußköpfen versehen. Die äußere Z-Drahtlage weist in Seilmitte zwei nebeneinander liegende Drahtbrüche auf. (Fig. 3)

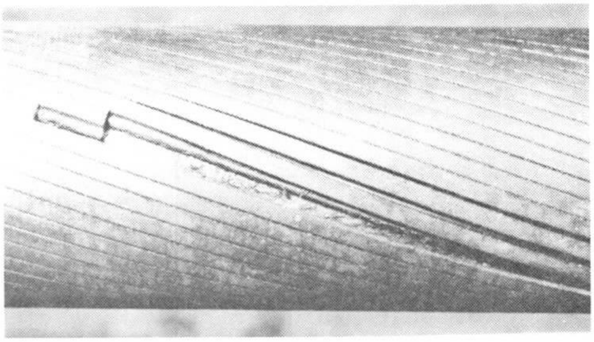

Fig.3 Verschlossenes Tragseil, 63 mm Durchmesser, mit zwei nebeneinander liegenden gebrochenen Z-Drähten, Belastung des Seiles in der Seilreckanlage: 1430 kN

Nach dem Einsetzen der Seilköpfe in den Kuppelwagen (Fig.4) und in den Klemmenwagen (Fig.5) wurde eine Last von 1430 kN, das sind 30 % der rechnerischen Seilbruchlast, aufgebracht. Mittels Einrichtung zur Konstanthaltung konnte die Belastung des Seiles während der Sanierung der beiden Drahtbrüche auf gleichem Niveau gehalten werden.

2.2 Versuchsdurchführung

Im ersten Arbeitsgang wurde das Bruchende des Drahtes Nr.1 aus dem Seilverband gehoben und der Z-Draht in Richtung Kuppelwagen aus dem Seilverband geschält. In einer Entfernung von 560 mm, das entspricht einer Seilschlaglänge, wurde der Z-Draht durchtrennt, geschäftet und mit einem neuen, 2800 mm langen Z-Draht, hart verlötet.

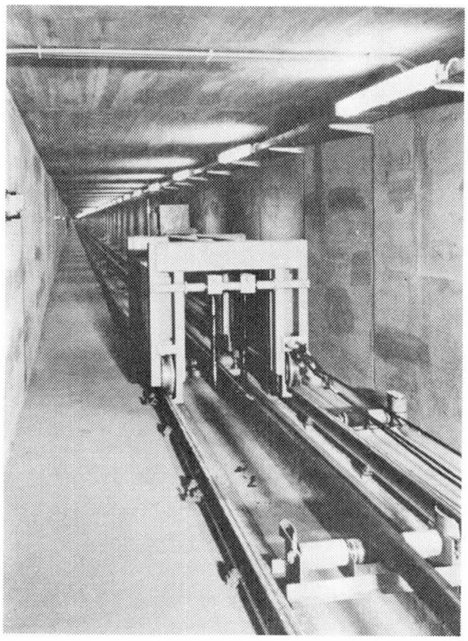

Fig.4 Seilreckanlage, 400 m lang, mit Einziehwagen, Kuppelwagen im rückwärtigen Teil des Tunnels

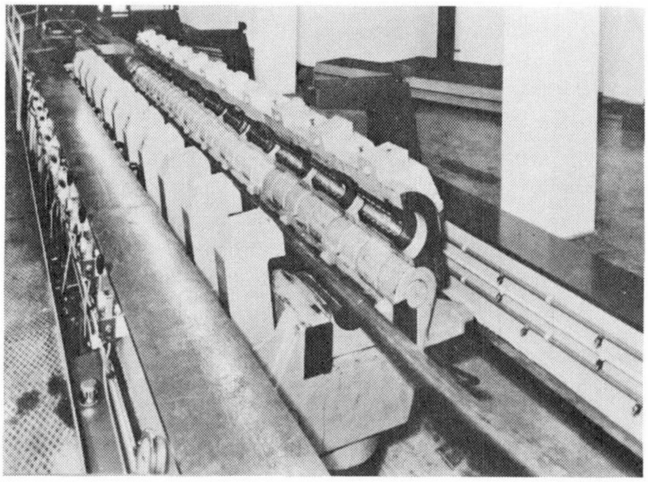

Fig.5 Klemmenwagen mit elf hydraulisch betätigbaren Klemmbacken

Im zweiten Arbeitsgang wurde der neue Z-Draht mittels einer gut passenden Schraubklemme in den Seilverband gepreßt und zwar vorerst bis zur ursprünglichen Schadensstelle. Im weiteren Verlauf der Sanierung wurde das zweite Bruchende des Z-Drahtes Nr.1 aus dem Seilverband gehoben und in Richtung Klemmenwagen, auf eine Länge von 2240 mm, aus dem Seilverband geschält. Anschließend wurde der neue Z-Draht von der ursprünglichen Schadensstelle mittels Schraubklemme in den Seilverband gepreßt. In einer Entfernung von 2800 mm von der ersten Lötung wurde die zweite Lötung mit kurzer Drahtüberlänge vorgenommen. (Fig.6) Nach Behandlung der Lötstelle mit Spezialwerkzeugen wurde im dritten Arbeitsgang die Drahtschlaufe mittels Schraubklemme bis zum Verschwinden in die Außenlage des verschlossenen Tragseiles gepreßt. (Fig.7) Hierzu war eine Wegstrecke von ein Meter Länge erforderlich. Die Gesamtlänge zur Sanierung eines gebrochenen Z-Drahtes beträgt in diesem Falle 3800 mm, das sind ca. sechs Seilschlaglängen. Mit der Sanierung des zweiten Drahtbruches wurde in gleicher Weise wie beim Drahtbruch Nr.1 verfahren.

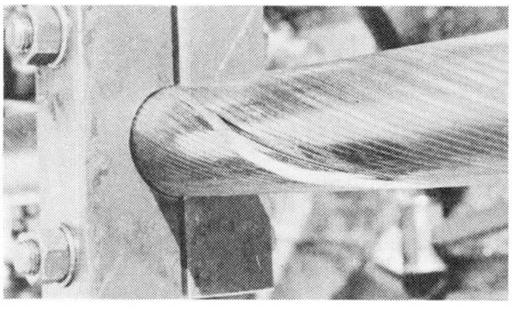

Fig.6 Einpressen der Drahtüberlänge (Schlaufe) in den Seilverband mittels einer Schraubklemme

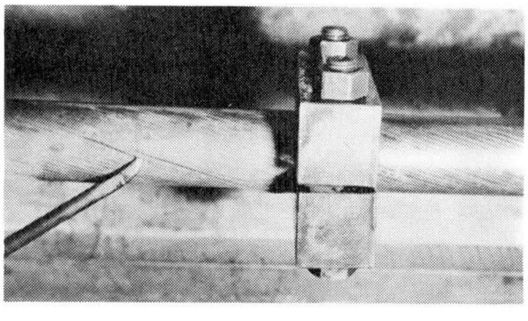

Fig.7: Seilzustand nach dem Einpressen der Drahtüberlänge in den Seilverband, Entfernung von der sanierten Bruchstelle: 2800 mm

Aus Fig.8 ist der Seilverband im sanierten Bereich ersichtlich. 'Alle Z-Drähte der Außenanlage liegen fest im Seilverband. Unmittelbar nach der Sanierung tragen die eingepreßten und gelöteten Z-Drähte nicht voll mit. Ein weitgehender Spannungsausgleich zwischen den Z-Drähten tritt jedoch nach mehrmaliger Be- und Entlastung auf.

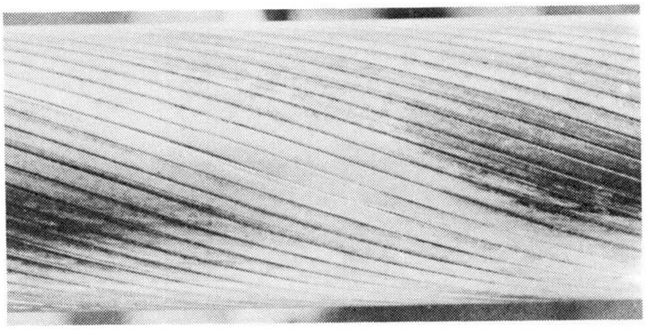

Fig.8 Verschlossenes Tragseil, 63 mm Durchmesser, Seilzustand im sanierten Bereich

2.3 Beschreibung der Seilreckanlage [4]

Die Seilreckanlage wurde 1970 im Werk Wien als Tunnelreckanlage, vier Meter unter Hüttenflur, errichtet. (Fig.4) Als Besonderheit der Anlage ist hervorzuheben, daß der Reckvorgang zu jeder Zeit bei konstanter Temperatur und unabhängig von den Außenbedingungen erfolgen kann.

Die wichtigsten Daten der Seilreckanlage:

Maximale Reckkraft:	5000 kN
Länge des Tunnels:	400 m
Hubweg des Reckkolbens:	3500 mm

Die Reckkraft wird mittels eines hydraulischen Reckzylinders aufgebracht. Je 12 mit Feinzinklegierung gefutterte Klemmbacken ermöglichen das Recken von Seilen bis zu 100 mm Durchmesser und 40 Tonnen Stückgewicht. Die Krafteinleitung erfolgt über hydraulisch betätigte Klemmbacken. (Fig.5) Dadurch ist es auch möglich, Seile mit einer Länge von mehr als 400 m vorzurecken.

3. SANIERUNG VON AUFLIEGENDEN VERSCHLOSSENEN TRAGSEILEN EINER HÄNGEBRÜCKE UNTER BETRIEBSBELASTUNG

In den verschlossenen Tragseilen, 72 mm Durchmesser, einer Hängebrücke mit 325 m Spannweite, wurden nach 18-jähriger Aufliegezeit eine große Anzahl von Drahtbrüchen festgestellt. Drahtbruchhäufung trat insbesondere im Bereich zweier Hängerklemmen auf, wo die Tragseile mittels Stahlplatten und Bügelklemmen eingespannt und gepreßt sind, wodurch es in Verbindung mit Schwingspannungen zu einer örtlichen Überbeanspruchung kam. Die Hängerklemmen befinden sich im Vorspannfeld der Brücke und sind nur 12 m vom Pylon entfernt. (Fig.2) Nach Überprüfung des Seilschadens konnte festgestellt werden, daß die Bruchenden der Z-Drähte die Form des Dauerbruches aufweisen. Ermüdungskorrosion trat nicht auf.

Da der Allgemeinzustand der Tragseile befriedigend war, wurde deren Sanierung durch Einpressen neuer Z-Drähte im Schadensbereich unter Betriebsbelastung beschlossen. Während der Sanierungsdauer, die drei Wochen in Anspruch nahm, mußte der Verkehr auf eine Fahrbahnhälfte eingeschränkt werden.

3.1 Lötstellenplan

Im Bereich einer Hängerklemme wurden in 6 von 12 Tragseilen insgesamt 24 Drahtbrüche festgestellt. Die Lage der Drahtbrüche in den einzelnen Seilen ist aus dem Lötstellenplan ersichtlich. (Fig.9) Die Verteilung der Lötstellen wurde so festgelegt, daß der Abstand zwischen zwei benachbarten Lötstellen 700 mm beträgt. Im Bereich der Hängerklemme wurde eine lötstellenfreie Zone von 4 m geschaffen. Aufgrund der Drahtbruchverteilung ergaben sich im zu sanierenden Bereich folgende maximale Einziehlängen bzw. Sanierungslänge: Seil Nr. 11: Einziehlänge 10,3 m und Länge des sanierten Bereiches 16,6 m.

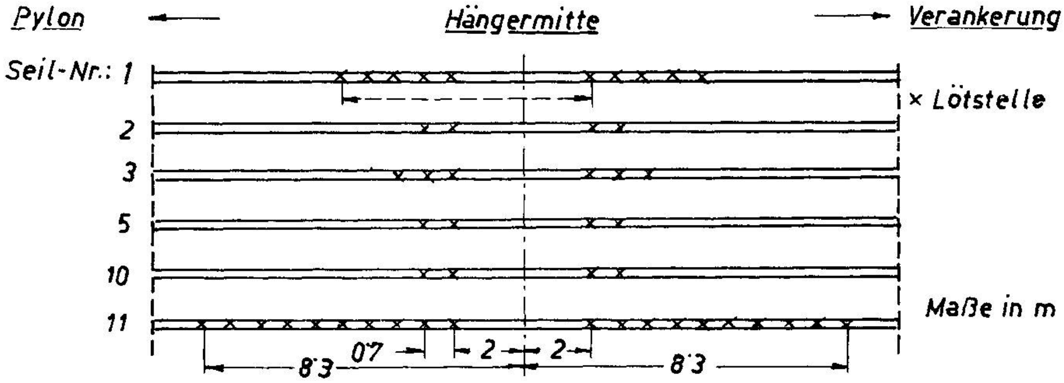

Fig.9 Lötstellenplan zur Sanierung der verschlossenen Tragseile im Bereich einer Hängerklemme

3.2 Sanierung der Drahtbrüche durch Löten und Versetzen der Stoßstelle

Diese Methode wird dann angewendet, wenn sich die Schadensstelle in unmittelbarer Nähe eines Pylons oder in Nähe der Verankerung befindet, so daß für das Vertreiben der Drahtüberlänge nicht genügend Wegstrecke zur Verfügung steht. Es wurde aus diesem Grunde jeweils ein Ende des einzupressenden neuen Z-Drahtes durch Hartlötung mit einem gebrochenen Z-Draht verbunden; das zweite Ende auf 200 mm Länge ausgeglüht und auf Stoß in den Seilverband gepreßt. Außerdem wurde das Drahtende mit Kunstharzkleber bestrichen und gegen Eindringen von Feuchtigkeit geschützt.

Vor dem Einpressen der neuen Z-Drähte wurde die Seiloberfläche durch Sandstrahlen gereinigt. Das Einpressen der Z-Drähte erfolgte wie im Kapitel 2.2 beschrieben. Die in die Tragseile gepreßten Z-Drähte liegen seit der Sanierung, die 1984 stattfand, noch immer fest im Seilverband. Eine zusätzliche Absicherung der sanierten Seillänge ist nicht erforderlich. Fünf Jahre nach der Sanierung zeigen die verschlossenen Tragseile keine nachteiligen Veränderungen. Der Erfolg der angewandten Sanierungsmethode ist nachhaltig.

Entsprechend der durchgeführten Methode können auch schadhafte Tragseile von Schrägseilbrücken saniert werden. Voraussetzung dafür ist, daß die Tragseile frei zugänglich sind und eine Arbeitsplattform errichtet werden kann.

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Long-term Anticorrosion Protection for Guys of Cable-Stayed Bridges

Protection à long terme des câbles de ponts haubanés

Dauerhafter Korrosionsschutz für seilverspannte Brücken

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SUMMARY

A very durable multiple anticorrosion protection proposal is presented to be applied especially on parallel cable elements of stayed bridges or similar structures where, by a relatively higher initial cost, an evident increase in reliability and useful life of cables is achieved having an enormous safety reserve, reducing control and maintenance expenses that result in a lower final cost.

RÉSUMÉ

On propose d'appliquer une protection anticorrosive multiple de longue durée aux câbles parallèles des ponts haubanés, ou aux structures similaires, lorsque, pour un coût initial relativement plus élevé, on obtient une augmentation évidente de la fiabilité et de la durée d'utilisation des câbles, tout en disposant d'une énorme réserve de sécurité et en obtenant une réduction des frais de contrôle et d'entretien conduisant à un coût final plus bas.

ZUSAMMENFASSUNG

Ein sehr dauerhafter Korrosionsschutz für seilabgespannte Brücken oder ähnliche Bauwerke wird beschrieben. Durch höhere Baukosten wird eine wesentliche Erhöhung der Dauerhaftigkeit und der Lebensdauer der Kabel erreicht. Daraus resultiert eine grosse Sicherheit mit entsprechender Verminderung der Inspektions- und Unterhaltskosten, wodurch die Gesamtkosten geringer ausfallen.

1. INTRODUCTION

It is not easy task to conceive a structure from the viewpoint of its durability. Studies on the subject show a pronounced structure duration dispersion [1] and a certain contradiction between the frequency of the failure cases and the theories dealing with their reliability [2]. Though there exists already a definite trend towards tackling design from a probability viewpoint to solve questions concerning durability -and also safety and serviceability- it must be admitted that the material failure precise nature is not known.

Within a context so conceived, a multiple anticorrosion protection is proposed that seeks a durability as long as the stayed bridge useful life. This type of bridge, recognized as an economical, reasonable, aesthetical, lasting solution, especially efficacious for 200 to 500 m spans, has not offered a satisfactory "status quo" with respect to cable durability. During past years there have appeared cases of corrosion and deterioration in guys of important bridges in Europe, U.S.A., Latin America and Japan [3], [4], [5], [6], that lead to think that there is not an adequate coherence between the decisive structural importance these cables have and the protection safety and durability for which the most qualified stayed bridge pioneers, designers and constructors are crying out, clearly emphasizing the need for a robust and reliable corrosion protection system for the stay tendons.

2. ESSENTIAL IDEA OF THE SISTEM PROPOSED

The main objective is having a protection the duration of which approaches the bridge expected useful life (conventionally, 75 years). The design is based on the conviction that owing to the materials deterioration laws phenomenology, with respect to anticorrosion it is not possible to expect spectacular solution centered on a magic product or method, so to obtain a very long duration protection recourse must be had to a highly reinforced protection. High polymer materials have been chosen considering that high molecular weight enables them for lengthy duration, especially if suitable precautions are taken.

The central idea -the system fundamental key- is to protect a protective element considered essential; in this case a high density polymer inner pipe (HDPE) or similar to which should be guaranteed a sort of "hibernation" aided by other elements that besides acting also as anticorrosion protection, insulate the inner pipe from temperature and weather. Waterproofing should be paid as much attention to as temperature insulation.

3. DESCRIPTION OF THE PROTECTION

The system proposed (Fig. 1) is composed of a HDPE inner pipe (3) circling the tension elements bundle; another HDPE pipe (6), light or white coloured; an injection between pipes (4) and another injection (1) within the inner pipe (3) both of an elastomeric or plastic material or eventually portland cement with polymers and a two layered wrapping or otherwise only one tape complyind with the same purpose. The first helps as a fastener of the whole, and the external one, white or light coloured functions as a protection against UV rays, IR radiation, oxigen and ozone and as a temperature reducer. Also, if vibration produced in the cable by the wind are expected to be significant, it is advisable that external side of the wrapping be corrugated, scarified, ribbed or creased in such a direction that once in place it shows an aerodynamically oriented pattern for dissipating the vibratory energy.

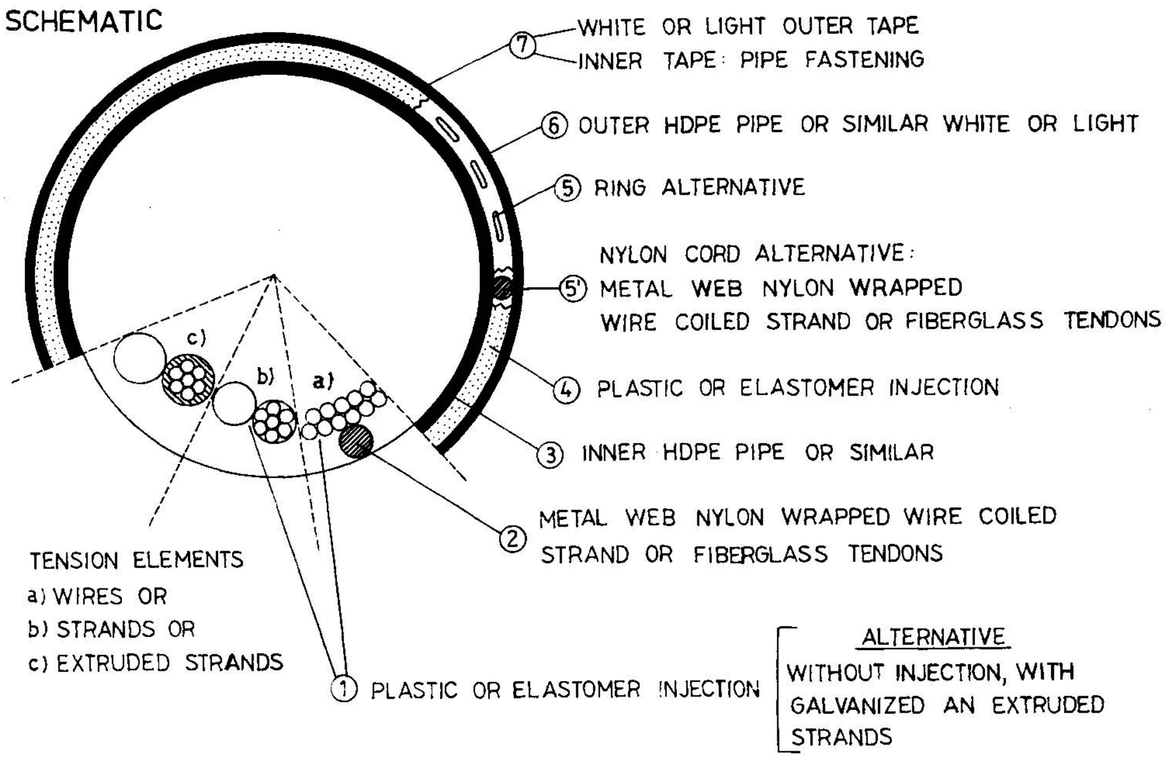

Fig. 1 - Cross section of the proposed cable and alternatives

The spacing between pipes may be obtained with rings (5) perforated for the injection to get through and welded to the inner piper only but in contact with the outer pipe, whitout adherence, with the object of allowing the longitudinal movement of the protection top part. The adhesion of the ring, that may be made of any suitable plastic material, can be achieved by the plastic welding update techniques developed nowadays. Though the polythylene thermal stability makes brief heating possible near the softening point without causing any trouble -provided that it does not occur simultaneously with mechanical loads- if it is desiderable to avoid a certain temperature range, a metal web nylon wrapped wire coiled strand (5') may be used over the pipe (or fiberglass tendons as those used for posttensioning concrete bridges), similar to the one used to fasten the wire bundle (2) acting also as spacer. This strand (2) helps -together with rings and the other strands (5')- in keeping the cable circularity the objective of which is to prevent additional interferences in future surveying performed by magnetic induction or similar method. If strands are chosen a tension elements (b) and (c) in Fig. 1, care should be taken that the fasteners and or spacers do not damage the HDPE pipe, the steel or the single tape wrappings.

In the case of very lengthy cables and where -owing to the weight- the deflection requires lightening, the guy diameter can be reduced by eliminating strand (2) and making a direct extrusion on the bundle; or -in case strands are used,- injection (1), provided that the strands be extruded and galvanized individually. In any of these alternatives, the remaining protection (from 4 to 7) should be added to secure a long lasting useful life, according to the above mentioned concepts.

4. PROPOSED SYSTEM ADVANTAGES AND PECULARITIES

It is considered that the system described offers the following characteristic advantages: 1°) The plurality of the different component material, implies extreme safety, as a cause of corrosion is not apt to attack simultaneously different materials such as those of the proposed system with success; 2°) ageing or deterioration is notoriously retarded in the inner layers which greatly lengthens its useful life and consequently that of the cable; 3°) there is a large availability of time to change the outer protection without risk for the steel; 4°) spot accidental causes (notches, plastic components defects) dangerous in systems with less elements, lose importance in a multilayered system; 5°) independently from the tension steel anticorrosion properties, the protection emphasis should be placed on the steel "external" elements sum and synergy. This approach permits an absolute liberty in the choice of the tension elements proper. On this concern, it must be taken into account that steels suitable for tensioning have suffered a decrease in their response to strain and fatigue resistance owing to treatments applied directly on same (hot galvanizing or previous treatments such as sanding, phosphatizing and chromium plating) [7], [8]; 6°) without detriment to the bridge being correctly designed for vibration and fatigue, the proposed outer wrapping roughness makes more effective the cable antivibration response; 7°) the selection of a sum of differential thicknesses necessary for the impermeability an the decrease of the thermal gradient, as substitute for only one thickness, allows replacements by laters in case of deterioration, far off from the risk of the "all or nothing"; 8°) plastic injections capacity for deformation, expansibility or elastic resiliencie secures continuity and weather tightness, since they fill all voids and hollows and readjust in the presence of cable deformation; 9°) plastic flexibility, positioning of the cable with all its protection avoiding "in situ" injections, notoriously increasing quality levels and implying that during construction higher loads should not be incorporated, avoiding in this way stress checking tests under the urgencies and difficulties imposed al this stage; 10°) the multilayered system provides high shock absorption and the cuts and flattenings that may be caused by handling and mouting are circumbscribed to a periphery for away from the protection nucleus; 11°) high polymer injections or fillings permit obtaining mixes that under tensile stresses, for example, only shows very small depth fissures (0,03 to 0,05 mm) that are far from water penetration limits (0,1 to 0,2 mm) a behaviour highly superior to that of the rigid injections; 12°) the possibility of producing a cable entirely factory or "in situ" made, permits the control of the injections pressure in order to make it small enough so that it may not affect the pipes long duration desired, especially the inner one; decrease of the two thermodynamic coordinates (pressure and temperature) and absolute protection the inner pipe has against UV rays and other weather phenomena, are the main factors that permit forecasting a useful life similar to that of the bridge.

The importance -for the duration of the HDPE or similar pipe- of reducing pressure and temperature (they are variable and intermittent Δt) is verified inmediately when observing these materials characteristics curves based on plastic deformation and relaxation test and that relate temperature, duration, and triaxial stress originated by internal pressure [9]; 13°) the nylon strands do not leave any imprint on the HDPE pipes or any other high resistance plastic and do not imply restriction to the mobility of the elements they come into contact with; 14°) the proposal for the most external of the pipes is that it should be ligth coloured and treated against UV rays in spite of the outer wrapping having the same properties. This arrangement implies further safety in case that due to neglect in surveying there may come long period of time in which no wrapping deteriorations are detected; 15°) the proposed system can be easily rehabilitated. There is no problem with the wrapping and any pipe(s) section is replaced with half round pieces of the same material welded "in situ"; 16°) relaxation and creep may be reduced if certain precautions are taken and selection made. Recent investigations carried out in Japan [10] show that if the combination PWS (parallel wire strands) plus Hi-Am ("cold" mix for anchorage that melts only at 110°C) is adopted, creep and relaxation reach values of only 3,7 % respectively, while for LCR (locked coil rope) plus Z (Zamak type metal mix anchorage or similar) melting at 350 thorough 450°C, the values reach magnitudes of 13,8 and 10,3 % respectively.

As for steel pure relaxation in cables, it may be reduced if a considerable insulation against temperature is used, as the one provided for the protection. This relaxation depends on temperature and the initial stress and also on stress cyclic variation; a phenomenon that acquires some importance owing to the great stress oscillation amplitude. It must be considered that in some regions and seasons of the year, temperature on the surface of a great number of cables reaches up to 70 to 80°C. Other investigations made in relation with the steam curing influence on prestressing steels, over the mentioned temperature range (anisothermal Test) [11] , [12] , show that steel relaxation may be increased from 3,7 to 16 % above the one measured at the conventional 20°C temperature (isothermal Test). Notwithstanding the defferences that may be pointed out between the influence of the steam curing duration and the day-night cyclic Δt ; of the extrapolations used by researchers and that if certain steels such as the "stabilized" are used, a better response to relaxation is achieved (though these steels are more sensible to corrosion), it is important to remark that, anyway, the sum of the stresses during mounting and/or those originated by cyclic variation plus effect of the mentioned temperatures may induce relaxation that agree very little with the tensional demands supported by the stayed bridges, since to its temperature susceptibility are added higher demands imposed by the sustained increase of the main spans and the ever more sophisticated design of the deck transversal sections.

For this reason, a protection blockading the arrival of significant temperatures to the steel always implies an improvement -no matter its quantum- in relaxation decrease. There is another advantage to be added: high polymer injection imply no restriction to wire deffered deformation, which facilitates the possibility of the tension element total loss more accurate calculation, this characteristics being more important than the eventual restrictions -the evaluation of which is controvertible- that may present rigid injection system or other type of cables. All the factors that have been mentioned encourage the consideration that any improvement of the cables with references to deformation may be the reason that will make possible -as requirements increase- another step in the evolution of these bridges or of other stayed structures; 17°) offer great additional safety if sudden or undervaluated effects appear; 18°) have a satisfactory behaviour in the presence of wide range of climates; 19°) minimize the temporary protection problem; 20°) retightening, if considered possible, made whitout generating interference; 21°) offer an important reserve in the presence of fire, intentional damage and vandalism; 22°) minimize time between structural clousure and bridge opening to service; 23°) part of this arrangement -from (4) to (7), Fig. 1- may be thought of as a long lasting overprotection able to protect a wide range of existing tension members; 24°) the protection being highly reinforced, it is only logical to expect from it a high reliability and consequently be able to reduce the control usual periodicity. The resultant savings, only in this item, throughout the cable useful life, largely compensate the cost of more than one protection as proposed.

From the description of the system presented and the analysis of the advantages that have been pointed out, it is considered that the system complies with the objective of obtaining a long lasting protection ranking with the cable stayed bridge hierarchy and importance.

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Durability of Bridge Expansion Joints

Durabilité des joints de dilatation des ponts

Dauerhaftigkeit von Fahrbahnübergängen

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SUMMARY

In recent years, various failures of expansion joints of highway bridges occurred very often causing inconvenience to traffic and further deterioration to girders and bearings. In order to improve the durability of joints, the authors re-examined the data to be used in the design of joints, that is, the vehicles weight exceeding the standard weight, its distribution between axles, impact coefficient to be applied and contact pressure intensity between tires and road surface. The authors have also carried out field measurements of stresses in the joints induced by traffic loads, and discussed the results to be obtained.

RÉSUMÉ

Récemment, de multiples ruptures de joints de dilatation de ponts routiers occasionnèrent de fréquentes perturbations du trafic et des déteriorations ultérieures aux poutrelles et aux coussinets antifriction. Afin de tester la durabilité des joints, les auteurs ont examiné à nouveau les données utilisées dans la conception de joints, à savoir la prise en compte du poids d'un véhicule lorsque celui-ci excède le poids standard, la distribution du poids entre les essieux, les coefficients d'impact à utiliser et l'intensité de la pression de contact entre les bandages des roues et la surface de contact. Les auteurs ont ainsi réalisé des champs de mesure des forces dans les joints sous charge de trafic et ont discuté les résultats obtenus.

ZUSAMMENFASSUNG

In den letzten Jahren verursachten Schäden an Bewegungsfugen von Autobahnbrücken Verkehrsbehinderungen und Folgeschäden an Brückenlagern und -trägern. Zur Verbesserung der Dauerhaftigkeit der Fahrbahnübergänge wurden die Bemessungsannahmen untersucht (Schwertransporte, Achslastverteilung, Stossfaktoren und Reifenpressungen). Es wurden auch Feldmessungen der in den Fugenkonstruktionen auftretenden Spannungen durchgeführt und mit den Berechnungsresultaten verglichen.

1. INTRODUCTION

A bridge expansion joint is subjected to the direct loading of moving vehicles and, in recent years, has caused many cases of damage as a result from increasing traffic volume as well as vehicle size, thus presenting itself as one of the major problems for maintenance of highway structures.

The facts are not sufficiently reflected in the design of expansion joints, and the design method involves many ambiguous points concerning the acting load intensity and the load applying mechanism. Furthermore, in the design, effects of fatigue should be given the due consideration.

In this situation, in order to improve the durability of expansion joints, the establishment of the design method seems to be indispensable. From this point of view, the authors investigated weights of actual running vehicles, impact coefficient and contact pressure intensity on the joint surface, and performed loading tests including the measurement by a stress histogram analyzer under the actual traffic, to confirm the stresses generated in expansion joints.

This paper presents results of these investigations and tests. From the results, various data which will promote the improvement of durability of expansion joints in the future have been obtained.

2. WHEEL WEIGHT

The Specification for Highway Bridges [1], Japan Road Association, adopts T-20 loading as the vehicle load (Fig.1). Based on this loading, the Manual on Bridge Expansion Joint Systems [2] specifies the wheel weight acting on an expansion joint as 78.4kN (8tf).

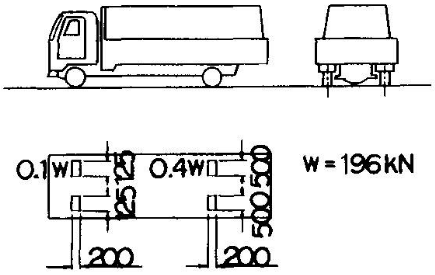

Fig.1 T-20 Loading [1]

Results of investigation of the vehicle load [3], carried out on urban expressways in Japan, are shown in Table 1 and Table 2. From the results, the maximum tandem axle weight was 436.4kN of a trailer. Maximum value of tandem axle weight is surmised as 490kN (50tf) for calculation in regard to safety.

Table 1. Mixed ratio of car type [3]

car type		axle form	mixed ratio	
large sized truck	2-axles	front axle, rear axle	not loaded	1.22
			loaded	1.02
			over loaded	0.01
	3-axles	tandem axle	not loaded	3.72
			loaded	6.04
			over loaded	0.05
	trailer		not loaded	1.02
			loaded	1.18
medium sized truck			17.88	
passenger car			67.86	

If the weight ratio between tandem axle is 1:1.7, the maximum wheel weight, composed of 1 to 4 tires, is calculated as 154.3kN. Therefore, it is desirable that the largest load of 156.8kN (16tf) is to be applied, as the static wheel weight for the verification of ultimate state.

Table 2. Average and maximum weight of axle axle of over loaded vehicle [3]

Axle form	Average weight of axle (kN)	Max. weight of axle recorded in 24hrs (kN)	Max. weight of axle over six years (kN)
2 axles	131.4	176.5	193.2
3 axles	236.6	306.0	389.3
trailer	190.5	338.3	436.4

Moreover, from Table 1, it seems that the adequate load should be decided, as the wheel weight for the verification of fatigue limit state, based on the further investigation of over loading.

3. IMPACT

The impact acting on an expansion joint can be estimated to be larger than those acting on other parts of bridge, because of direct loading application. In the design of expansion joints, the value of 1.0 is customarily used as the impact coefficient.

In order to confirm the validity of this value, the authors studied the effects of road surface condition and running speed on impact and the effects of over loading rate on impact. The results are shown in Fig.2 and Fig.3. The impact is larger on a poorer road condition and at a higher running speed, but if the running speed exceeds 50km/h, the impact tends to become smaller. According to the results of tests at real bridges (Table 3), the values of impact coefficient near expansion joints range between 0.5 to 0.8.

From the above, the largest impact coefficient is evaluated to be about 0.8, but considering abnormally over loaded running vehicle and the occurrence of level difference near expansion joint, it is desirable to set the possible maximum coefficient of impact at 1.0.

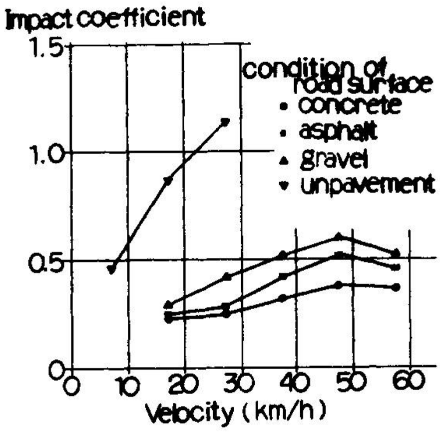

Fig.2 Impact coefficient due to condition of road surface and velocity

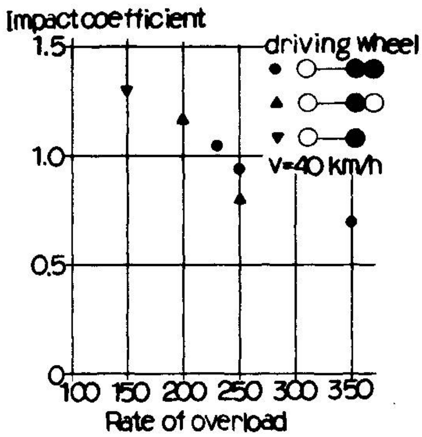

Fig.3 Impact coefficient due to rate of over load (unpaved road)

Table 3. Impact coefficient measured at real bridge

Joint type	Static load(kN) Mid axle	Static load(kN) Rear axle	Maximum joint level difference (mm)	Vehicle speed (km/h)	Max. working load (kN) Mid axle	Max. working load (kN) Rear axle	Impact coefficient Mid axle	Impact coefficient Rear axle
Finger type Joint	73.5	71.0	4	10	87.5	81.0	0.19	0.14
				20	111.0	100.9	0.51	0.42
				40	121.3	103.8	0.65	0.46
				60	132.3	112.3	0.80	0.58
Finger type Joint			2	10	86.0	81.0	0.17	0.14
				20	100.7	93.1	0.37	0.31
				40	102.9	90.3	0.40	0.27
				60	119.1	112.3	0.62	0.58

※Level difference is between two parts of joint

4. CONTACT PRESSURE INTENSITY BETWEEN TIRES AND ROAD SURFACE

A tire of vehicle contacts the road surface as a plane, and the vehicle load is transmitted to the road surface or expansion joint as the contact pressure of a tire. It is considered that the contact pressure is affected by the tire load, air pressure in tire, roughness of road surface and running speed of vehicle. The authors took notice of the influence by the tire load among these factors.

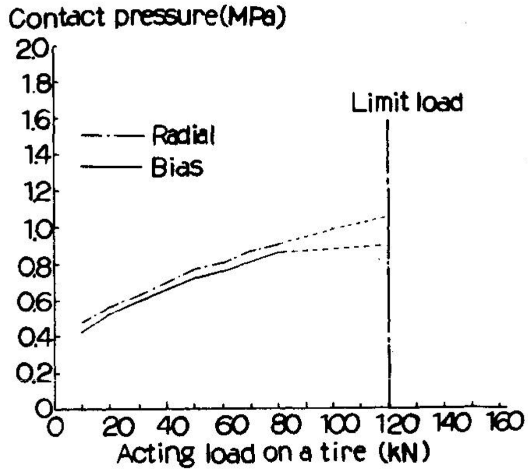

Fig.4 Acting load and contact pressure relation

Generally, if the tire load increases, the width of contact area changes only very slightly, but the length of contact area changes greatly. The relation between the contact pressure and the acting load of a tire, derived by the authors, is shown in Fig.4. The contact pressure gradually levels off at the tire load exceeding 78.4kN (8tf), and turns out the maximum pressure 1.1MPa at the tire load 120kN. Because if the tire load exceeds 120kN, the tire blows out.

From the above, the possible maximum intensity of contact pressure is to be 1.1 MPa at the tire load 120kN (including the impact force).

5. LOADING TESTS

5.1 Expansion Joint used for Tests

To confirm the loads acting on the expansion joint and the stresses generated in joints by practical vehicles, the authors carried out field tests using an expansion joint installed on a new bridge. The expansion joint used for the tests has a form as shown in Fig.5. This is made of casted aluminium alloy with a tensile strength of 270Mpa and equivalent to NF Standard A-S7G06 [4], and designed with the safety factor based on the Specification of Japan Light Metals Association [5], to decide the allowable stress of the material (Table 4).

In the design method of this joints, a wheel weight is replaced by the contact pressure of a tire which is assumed to act on the expansion joint (Fig.6).

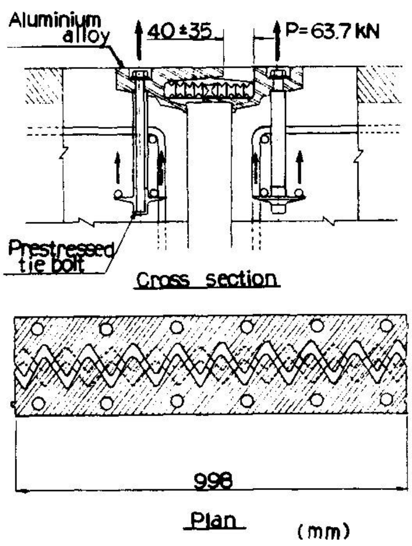

Fig.5 Used joint in loading tests

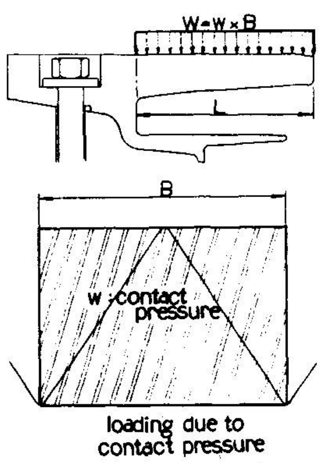

Fig.6 Loading method in design

The wheel weight 78.4kN (8tf), impact coefficient 1.0 and the contact area of wheel 200×500mm are set at the values conforming to the Specification for Highway Bridges [1] and the Manual of Bridge Expansion Systems [2]. As a result, the design contact pressure is the value of 1.6MPa.

This design contact pressure is larger than the possible maximum intensity 1.1 MPa verified in Chapter 4. The design maximum stress due to this contact pressure can be calculated 77MPa, and has a sufficient margin of safety against the allowable stress. Also, fatigue is not surmised to pose any problems since the fatigue limit of the material is about 78~86MPa as shown in Fig.7.

Table 4. Allowable stress of material

Designation		Factor of safety	Stress (MPa)
Tensile strength		———	270
Stress in 0.2-permanent strain		———	260
Yield strength		———	195
Allowable stress	Tension	1.85	105
	Compression	1.85	105
	Bending	1.85	105
	Shear	1.85×√3	60

5.2 Test Results

The static loading tests and the moving loading tests were carried out using a dump truck with a gross weight 200kN (20.4tf) as shown in Fig.8. The static loading tests were conducted for two cases of loading (Fig.9), using a front tire of the dump truck. The largest generated stress in the expansion joint was 18.0MPa, lower than 25% of the design maximum stress 77MPa (Table 5). Also, the stress was lower than 50% of design stress 38.5MPa without impact.

The contact pressure of the tire inversely calculated from the generated stress 18.0MPa was the value of 0.36MPa, which was smaller than the contact pressure of 0.48MPa calculated from the real weight and measured contact area. It can be assumed that a larger portion of the wheel weight was supported by the concrete slab behind the joint due to the higher stiffness.

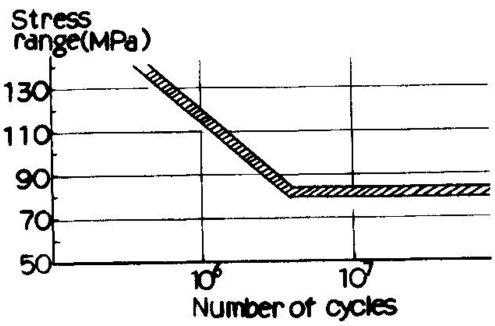

Fig.7 S-N Curve of aluminium alloy (A-S7G06) [4]

In moving loading tests, the speed of vehicle was changed in 6 steps from a very slow to 60 km/h. The largest generated stress was 21.3 MPa, and the largest impact coefficient was 0.25 at the speed of 60km/h (Table 6). The coefficient was smaller than the test result verified in Fig.2 of Chapter 3.

Immediately after the new bridge was opened to traffic, the measurement by a stress histogram analyzer was carried out for 24 hrs under the actual traffic load. The largest generated stress was 33.0MPa (Fig.10). The stress was 1.8 times the largest stress

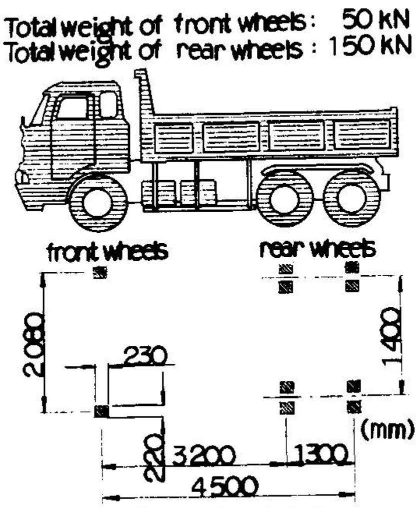

Fig.8 Load vehicle

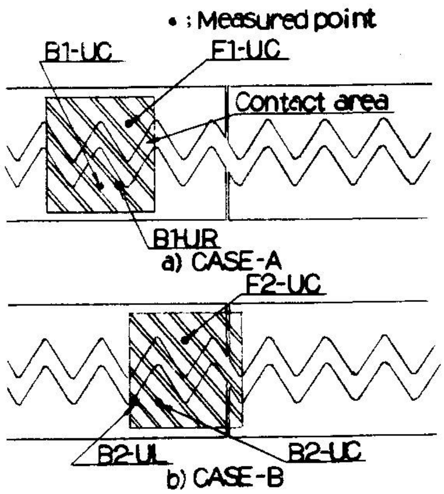

Fig.9 Loading condition

Table 5. Static loading test results

	Generated stress(MPa)					
	B1-UC	B1-UR	F1-UC	B2-UC	B2-UL	F2-UC
CASE-A	14.8	13.2	14.0	—	—	—
CASE-B	—	—	—	18.0	15.1	15.0

Table 6. Running vehicle loading test results

	CASE-1	CASE-2	CASE-3	CASE-4	CASE-5	CASE-6
Vehicle speed (km/h)	very slow speed	10	20	30	40	60
Impact coefficient	0.07	0.16	0.16	0.16	0.22	0.25

in the static loading tests and 1.6 times that in the moving loading tests. From this result, the presence of over loaded vehicles was surmisable.

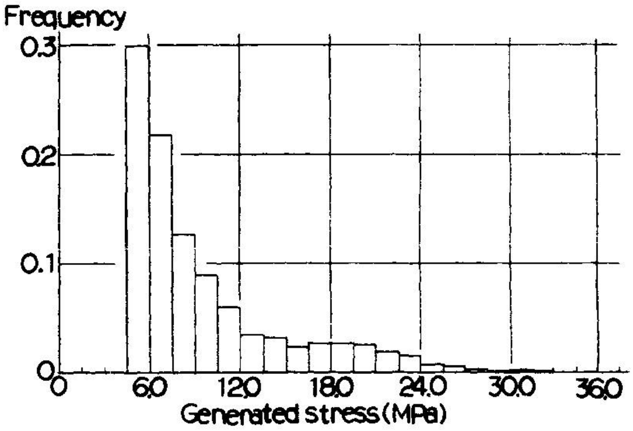

Fig.10 Stress frequency measurement

6. CONCLUSIONS

From the results described above, the following conclusions may be drawn :

(1) It is desirable that as the static wheel weight for the verification of ultimate state, the largest load of 156.8kN (16tf) composed of 1 to 4 tires is to be applied, based on the investigation records of loads of large size vehicles, and that as the wheel weight for the verification of fatigue limit state, the adequate load should be decided, based on the further investigation of the actual condition of over loaded vehicles.
(2) The impact coefficient acting on an expansion joint can be assumed to be about 0.8 at the largest. However, considering the occurrences of abnormally over loaded vehicles and level differences, it is desirable that the value of 1.0 is to be used as the possible maximum coefficient of impact.
(3) It can be assumed that the possible maximum intensity of contact pressure is the value of 1.1MPa at the tire load of 120kN (including the impact force), taking into consideration the fact that the tire blows out if the tire load exceeds 120kN.
(4) The expansion joint used for the tests, designed with the contact pressure of 1.6MPa in accordance with the specification and the manual in Japan, has enough margines of safety compared to stresses generated in the loading tests, and the design method can be assumed to be appropriate to secure the sufficient durability both in the ultimate strength and in the fatigue strength.

In order to verify the durability of other expansion joints with similar structures as the aforementioned joints, various tests are being performed on actual bridges as well as in laboratories including fatigue tests.

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Etanchéité d'un simulateur de souffle à grand gabarit

Abdichtung eines Windkanals grossen Querschnitts

Watertightness of a Large Wind Tunnel

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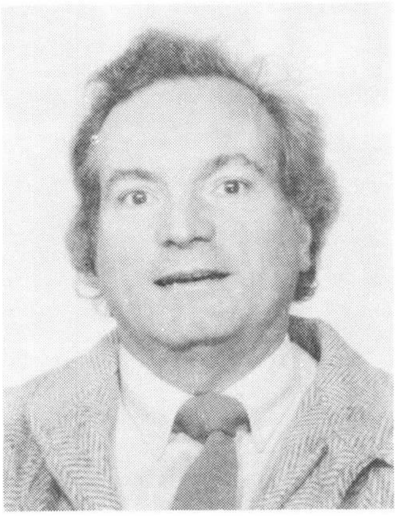

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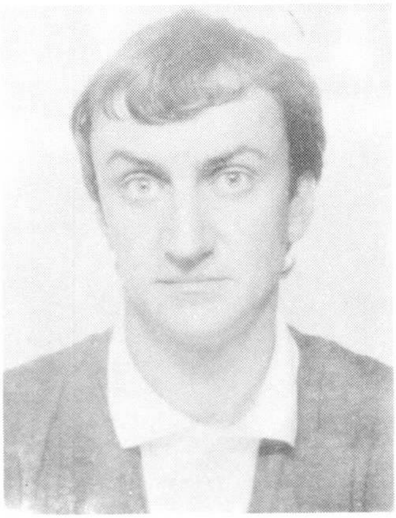


RÉSUMÉ

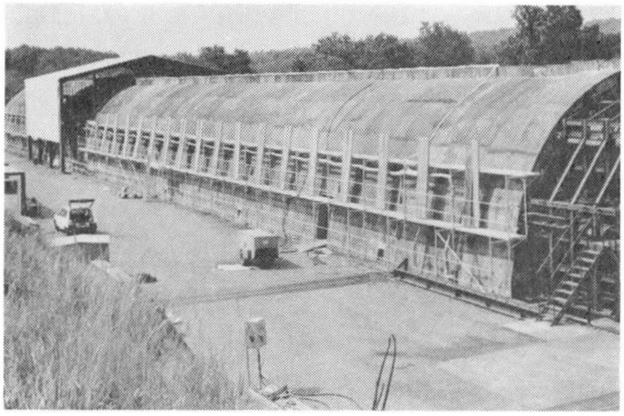
Un simulateur de souffle à grand gabarit (veine tubulaire de 105 m de long et 12 m de diamètre, en béton précontraint) présente des problèmes liés à l'absence d'étanchéité sur l'extrados de la structure. La protection de la précontrainte nécessite une imperméabilisation de la veine après traitement spécifique des joints entre les 14 éléments constitutifs. En raison de la configuration géométrique assez complexe et du délai très court imposé, le Laboratoire Central des Ponts et Chaussées propose la réalisation d'une étanchéité par pulvérisation d'élastomère de polyuréthanne haute qualité.

ZUSAMMENFASSUNG

In einem Windkanal grossen Querschnitts (Rohrlänge 105 m, Durchmesser 12 m, aus vorgespanntem Beton) zeigen sich Probleme mit der Wasserdichtigkeit. Zur Erreichung der erforderlichen Dichtigkeit ist eine Imprägnierung sowie eine spezielle Abdichtung der Fugen zwischen den 14 Bauwerksteilen erforderlich. Aufgrund der komplizierten Geometrie und kurzen Bauzeit schlägt das Zentrallabor des Strassenbauamtes eine Abdichtung mit einem pulverförmigen Polyurethane-Elastomer vor.

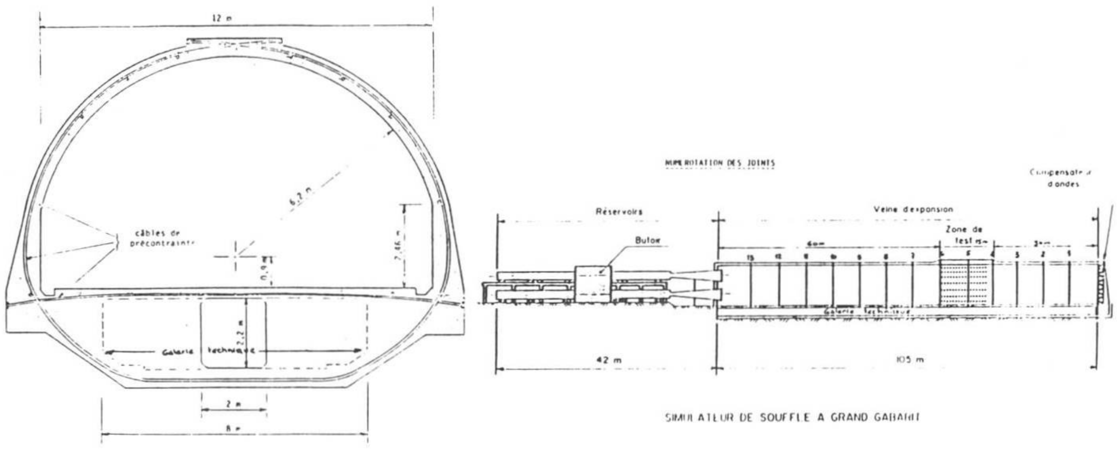
SUMMARY

A large wind tunnel (size: length 105 m, diameter 12 m) constructed of prestressed concrete, is giving problems with respect to watertightness. To achieve the required watertightness it is necessary to apply grouting as well as a special sealing of the joints between the 14 structural elements. Due to the complicated geometry and the tight time schedule imposed for the remedial work, the Central Laboratory for Bridges and Highways has proposed sealing the structure with a polyurethene elastomer in powder form.



1. GENERALITES

Le simulateur de souffle à grand gabarit est une veine en béton précontraint destinée à reproduire les effets mécaniques d'une explosion nucléaire : effets d'une onde de choc et de souffle provoquant des surpressions maximum de 1,4 bar pour un déplacement d'air estimé à 800 km/h.
La veine est formée par 14 éléments semi-circulaires précontraints transversalement par des câbles FU 4-600 de SEEE.
Un hourdis intermédiaire également précontraint transversalement sert de plancher à l'intérieur de la veine dégageant sous sa face inférieure une galerie technique destinée au stockage du matériel.
La longueur de l'ouvrage est de 105 m et sa largeur intérieure au niveau du plancher est de 12 mètres.



SIMULATEUR DE SOUFFLE A GRAND GABARIT

Afin de simuler une veine infinie, l'extrémité opposée au système générateur du souffle est équipée d'un compensateur d'ondes muni de volets réglables.
Celui-ci devant résister au souffle généré est "arrimé" à des tenons solidaires de câbles longitudinaux précontraints traversant la veine (36 câbles FU 4-600 de SEEE).
Ceux-ci sont tendus de manière à ce que l'effort de poussée exercé par le souffle sur le compensateur soit repris par l'effort de tension antagoniste des 36 câbles longitudinaux.
Un coefficient théorique de sécurité de 0.5 est retenu pour tenir compte d'éventuelles pertes de tension par frottement et fluage.
L'ensemble des câbles longitudinaux est injecté à la graisse CONDAT TRACTA 1391 pour que l'effort de tension -dû au souffle- mobilise la totalité du câble.

2. DESORDRES CONSTATES

2.1 Joints

La mauvaise liaison entre les différents éléments de la structure a entraîné
-des épaufrures aux zones en contact (phénomènes thermiques)
-des infiltrations d'eau importantes percolant à l'intérieur
-un délavage de la graisse ressortant des manchonages de gaines avec pour conséquence l'entraînement du produit et la souillure des parements

Sur ce dernier point, une étude approfondie a permis de constater que la graisse présentait un ressuage important (séparation d'huile dans des conditions déterminées). Sous l'influence de pressions internes le fluide lubrifiant peut migrer progressivement à l'intérieur du réseau et se séparer avec formation de poches d'huile. Ce phénomène est d'autant plus accentué que les interstices entre fibres sont grands et que la viscosité du fluide est faible.
L'incidence sur la protection des aciers est très grande car c'est la phase huile qui contient les agents nobles de la graisse que sont les additifs anti-corrosion.
Les ouvertures réalisées sur 4 gaines n'ont cependant pas permis de mettre en évidence un défaut de remplissage global mais il est certain que les surpressions dans les gaines ont entraîné la dissipation de l'huile et non du savon ce qui est préjudiciable à la pérennité des aciers.

2.2 Absence d'étanchéité

L'absence d'étanchéité superficielle sur la veine en béton a eu pour conséquence une érosion de celui-ci par l'eau de pluie. Cette eau, en s'infiltrant au travers de la structure attaque la chaux du béton (ciment) et produit les coulures de calcite constatées sur les parements au niveau des fissures.
Si cet aspect d'érosion n'est pas, en lui-même dramatique, bien que l'effet de la carbonatation puisse entraîner une oxydation des armatures superficielles et des éclats de béton, l'action de l'eau qui s'infiltre au niveau des cachetages de la précontrainte transversale (câbles anneaux) peut-être préjudiciable dans l'hypothèse où celle-ci pourrait pénétrer à l'intérieur des gaines et corroder les armatures de précontrainte.

3. PROPOSITIONS DE REPARATION

Afin de remédier aux désordres engendrés par les circulations d'eau et aux éventuels risques que celles-ci font courir à la précontrainte (transversale essentiellement) il a été décidé de procéder à l'étanchement extérieur de la veine après préparation spécifique notamment au niveau des joints entre les éléments qui la composent.

Les caractères spécifiques imposés à cette étanchéité sont les suivants :
-Surface à couvrir : 2300 m2 en 10 jours
-Mise en oeuvre sur support présentant des surfaces
  . hétérogènes (aspérités - arêtes vives)
  . horizontales inclinées ou verticales
  . comportant de nombreux éléments métalliques en saillie
-Résistance aux ultra-violets et infra-rouge et aux intempéries (chape non recouverte)
-Insensibilité aux tensions de vapeur c'est à dire bon coefficient de diffusion (effets de la vapeur d'eau de surface et des surpressions dues aux tirs)
-Bonne résistance à l'allongement au niveau des joints (dilatation thermique) pour des températures comprises entre -30°C et +50°C
-Très bonne adhérence sur béton et acier

Compte tenu des impératifs fixés, tant du point de vue "technique" que du point de vue "délai" (rappelons que la surface à couvrir est un 1/2 cylindre de révolution d'axe horizontal présentant un nombre important de pièces métalliques en saillie, d'arêtes vives etc...) les solutions classiques d'étanchéité ne pouvaient pas donner satisfaction que ce soient les chapes minces (feuilles) ou épaisses (asphalte).

C'est ainsi que la recherche d'une solution nouvelle a été entamée. Et c'est sur proposition du Laboratoire Central des Ponts et Chaussées qui venait de terminer un chantier expérimental d'étanchéité sur ouvrage d'art à base de polyuréthanne, que le Laboratoire Régional de Toulouse a envisagé de proposer cette solution aux responsables du Centre d'Etudes de Gramat.
La majorité des impératifs qui avaient été fixés, était théoriquement satisfaite par l'application d'un revêtement de la gamme BAYTEC de la Société BAYER : le BAYTEC 310-309 qui est un élastomère de polyuréthanne de haute qualité à deux composants constitués de polyétherpolyols et de diisocyanate - diphénylméthane.
L'application du produit a été confiée à l'entreprise SERP de ST Georges de Reneins (Rhône). Celle-ci s'effectue par pulvérisation au cours de laquelle se fait le mélange des deux composants.
Celui-ci est contrôlé et corrigé en permanence grâce à un ordinateur qui intègre en temps réel les paramètres essentiels comme la température, les rapports de débit des différents composants, etc...
Le produit prend en moins de 30 s une consistance cireuse puis élastique au bout de 20 mn Il faut cependant attendre 3jours pour obtenir l'optimum des propriétés mécaniques ou chimiques.

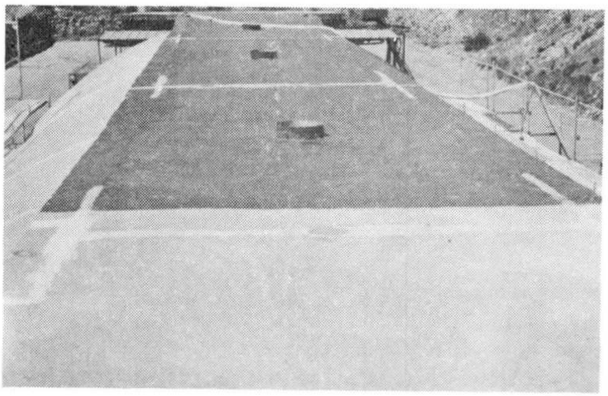


Photo n°1

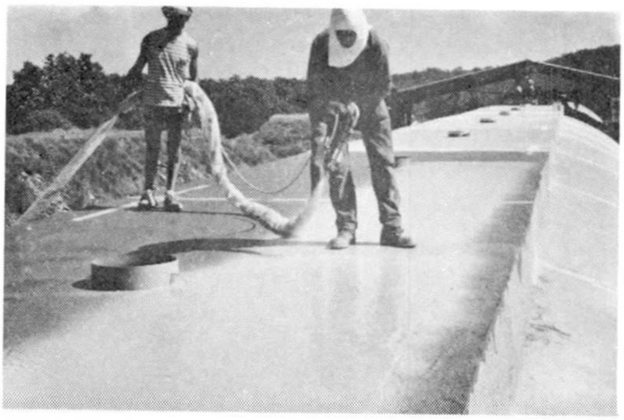
Préalablement à l'application du BAYTEC 310-309 une couche de fond, destinée à fixer la poussière et améliorer l'adhérence du polyuréthanne est mise en oeuvre : SOLYPRIM B de SOLYCA, qui est un système polyuréthanne compatible monocomposant durcissant par réaction avec l'humidité de l'air. Cette couche de fond doit être recouverte dans un délai compris entre 8h et 24h.

4. TRAVAUX DE REPARATION

4.1 Préparation

Nous n'insisterons pas sur les préparations classiques de ce type de travaux à savoir le sablage du béton, les reprises d'épaufrures, la protection des aciers apparents, etc...
Plus spécifique par contre était la préparation des joints. Il a été décidé de les injecter avec une résine époxydique souple (J24 de SRS) après nettoyage des traces d'huile et calfatage intérieur et extérieur. Ainsi, une bonne continuité d'ensemble a été redonnée à la structure sans avoir à craindre au niveau des joints de problèmes dûs aux phénomènes thermiques.

4.2 Application du BAYTEC Photo n°2



Lorsque les préparations ont été réalisées sur une zone assez grande, l'application du primaire Solyprim B et du BAYTEC ont pu commencer. Le délai d'intervention entre les 2 produits a été en moyenne de 16 h (application du primaire la veille), le BAYTEC nécessite une application sur 2 couches (2 passages espacés d'environ 20 à 30 s) d'épaisseur unitaire 2 mm.

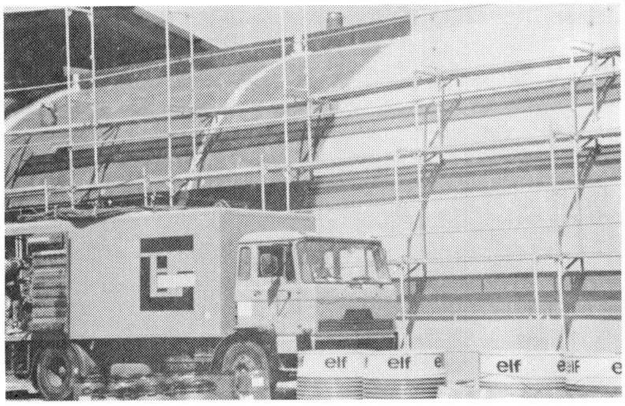


Photo n°3

Le matériel très sophistiqué utilisé pour la préparation du mélange et son dosage vers la pulvérisation est entièrement embarqué dans un camion autonome-photo ci-contre- Ce matériel est composé de pompes équipées de débit-mètres plongées dans les conteneurs de chaque produit. Chaque pompe est reliée au pistolet projeteur par des flexibles. La température des produits, la température extérieure, l'hygrométrie ambiante et du support sont en permanence mesurées et saisies par un micro- ordinateur qui gère ainsi en fonction de ces paramètres le dosage de chaque composant.

Cette haute sophistication du matériel est imposée par la technicité de l'opération. En effet, une modification "hors tolérance" de la composition du mélange peut entraîner un changement complet de la texture et de la structure du produit pulvérisé avec perte de ses qualités. Néanmoins, l'expérience d'une anomalie de fabrication -statistiquement prévisible sur un chantier important- a permis de constater la facilité de réparation pour revenir à un niveau de service encore très supérieur à celui obtenu avec d'autres types d'étanchéité.

L'anomalie constatée, s'est présentée sous la forme d'un bullage apparaissant en surface et ne concernant que la deuxième couche. La surface incriminée située au sommet de la veine était de l'ordre de 50 m2. La cause de bullage ou cloquage (qui enlevait totalement l'adhérence entre les 2 couches) a été trouvée dans un déréglage de la composition du BAYTEC suite à une partielle obturation du conduit d'un des 2 composants. L'ordinateur, équipé d'un système d'alarme en cas de déréglage n'a pas réagi car le système de mesure utilisé au début du chantier ne donnait pas accès au débit mais à la vitesse seulement qui, elle, n'avait pas varié (des modification ont été par la suite apportées).

Ainsi, en présence d'une zone défectueuse, il a fallu prendre les dispositions nécessaires pour redonner un bon niveau de service à celle-ci.

-L'enlèvement de la couche défectueuse a pu être réalisé parfaitement là où apparaissaient les cloques mais très partiellement là où aucune cloque n'était visible avec cependant suspicion de défaillance du BAYTEC. Dans tous les cas, la 1ère couche n'a pu être enlevée tellement l'adhérence au béton s'est avérée élevée.

-L'application, du primaire Solyprim B s'est donc faite sur une surface repréparée parfaitement saine.

-L'application définitive du BAYTEC, qui constituait la dernière phase de la réparation n'a posé aucun problème et les essais d'adhérence qui ont été effectués "in fine" sur la zone défectueuse ont donné une résistance moyenne à l'arrachement à 6 jours d'âge de 1.2 MPa ; la valeur la plus faible (0.9 MPa très ponctuellement) a été constatée lorsque la rupture a eu lieu -comme cela était suspecté lors de l'analyse des anomalies- entre 1ère et 2ème couche de la 1ère projection. Jamais une rupture ne s'est opérée entre la 1ère et la 2ème projection ce qui prouve l'adhérence parfaite que l'on obtient lors d'une superposition.

La totalité de cette zone a ainsi été parfaitement réparée et il est raisonnable de penser qu'aucun problème particulier ne l'affectera dans l'avenir. Pour s'en persuader et à titre de comparaison, l'adhérence des chapes traditionnelles sur le béton doit être de 0.4 MPa d'après le STER 81 qui est la recommandation française en vigueur.

4.3 Justification de la qualité de l'étanchéité

Dans toute réparation d'ouvrage, l'objectif a atteindre est que le mal dont souffre l'ouvrage soit éliminé par suppression de ses causes. Réparer sans avoir ce souci en tête n'est que leurre. Dans le cas présent, la réussite de la remise en état de l'ouvrage passait par la certitude d'obtenir un revêtement parfaitement étanche -y compris aux nombreuses parties métalliques en saillie- sur une structure à laquelle on aurait redonné une continuité (traitement des joints). Malgré les pannes insidieuses, qui ont entraîné la réparation d'une zone, la qualité du revêtement d'étanchéité appliqué sur l'ouvrage est remarquable. Il n'en faut pour preuve que :

-l'adhérence exceptionnelle du produit sur le béton (supérieure à 1.5 MPa) et surtout sur l'acier (parties métalliques en saillie) qui garantit l'impossibilité à une goutte d'eau de s'infiltrer à l'arrière de ces remontées et de pénétrer sous la chape.
-la résistance à l'allongement (traction simple) mesurée sur les éprouvettes confectionnées sur le chantier (essais Laboratoire Régional des Ponts et Chaussées d'AIX EN PROVENCE).

TEMPERATURE	ALLONGEMENT A RUPTURE	CONTRAINTE RUPTURE
20°C	290 %	5.6 MPa
- 10°C	> 62 % *	> 3.1 MPa *
50°C	> 70 % *	> 2.6 MPa *

* La capacité des enceintes climatiques à - 10°C et 50°C pour l'allongement, ne permettaient pas d'aller au-delà de ces valeurs, mais il est probable que les valeurs à rupture à - 10°C et 50°C auraient été voisines de celles à 20°C

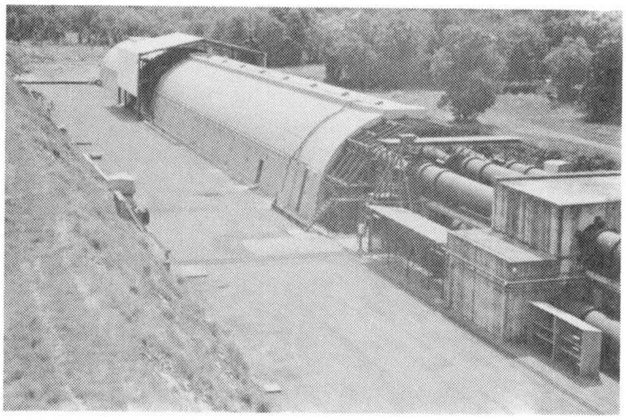


Photo n°4

Quant à la tenue dans le temps de ce type de produit, l'expérience acquise par BAYER sur des ouvrages similaires avec les mêmes conditions d'exploitation, ainsi que les essais de vieillissement accéléré effectués en laboratoire permettent de penser que la pérennité de la réparation sera excellente.

5. CONCLUSION

En conclusion, s'il est clair que la qualité du revêtement d'étanchéité est largement dûe aux caractéristiques peu communes du BAYTEC, il est néanmoins très important de rappeler, qu'en tant que Technique de pointe, ce type d'application nécessite un équipement très sophistiqué aussi bien en ce qui concerne l'appareillage de projection, que le système de gestion informatique en temps réel du mélange des produits en fonction des paramètres critiques hygrométrie et température.
L'amélioration des performances et de la fiabilité du matériel, qui est par ailleurs le souci permament des entreprises qui emploient ces techniques de pointe, concourra à éliminer ce que nous appelions le risque statistique d'occurence d'anomalie et par conséquent à faire de cette technique, un outil remarquable pour résoudre les problèmes d'étanchéité des ouvrages de génie civil et des ouvrages d'art en particulier.

Langzeitverhalten hochbelasteter unbewehrter Elastomerlager im Ingenieurbau

Long Term Behaviour of Plain Elastomeric Bearing Pads under High Stresses

Durabilité des appuis en élastomère non renforcés soumis à un effort élevé

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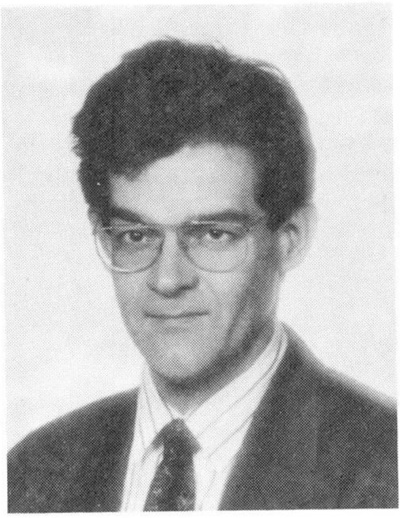

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ZUSAMMENFASSUNG

In bestimmten Anwendunsgsfällen werden unbewehrten Elastomerlagern höhere Pressungen zugewiesen, als sie nach den geltenden Baubestimmungen zulässig sind. Die dauerhafte Lagerfunktion unter solchen Beanspruchungen liegt ausserhalb des bisherigen Erfahrungsbereiches. Es wird das Spektrum des erwartbaren Dauerstandverhaltens marktüblicher Elastomerqualitäten aufgezeigt und es werden Beurteilungskriterien für das Langzeitverhalten formuliert.

SUMMARY

Higher stresses are sometimes assigned to plain elastomeric bearing pads than allowed in the construction codes. The durability of the bearing function is outside our range of experience. Based on the commercially documented elastomeric qualities the spectrum of creep and damage is given as well as hints to estimate long term behaviour.

RÉSUMÉ

Dans certains cas pratiques des appuis en élastomère non renforcés sont soumis à des pressions plus élevées que celles permises dans les réglements en vigueur. La durabilité et l'efficacité des appuis dans de telles conditions ne sont pas connues actuellement. L'étendue des durabilités annoncées pour des qualités d'élastomères commercialiés est présentée. Des critères de jugement du comportement à long terme sont formulés.

1. EINFÜHRUNG

Wirtschaftlichere Fertigungsmethoden im Ingenieurbau erfordern kostengünstige, montagefreundliche und wartungsfreie Lagerausbildungen, die bei niedriger Bauhöhe koinzident ggf. Verschiebungen und Verdrehungen erlauben. Traditionelle Mörtelbettausbildungen führen zu vergleichsweise "starren" Bettungen, die bei Bauteilverdrehungen infolge Kantenpressungen Schäden am Mörtelbett und an angrenzenden Bauteilen hervorrufen können. Stählerne Konstruktionen bedingen Lastkonzentrationen an Roll- und Kippelementen sowie deutlich höhere Bauhöhen und höheren Kosten- und Montageaufwand. Durch die elastische Verformbarkeit unbewehrter Elastomerlager werden innerhalb bestimmter Grenzen Abweichungen von der Ebenheit und Schiefwinkligkeit zu lagernder Bauteildruckflächen ohne kritische örtliche Spannungsspitzen ausgeglichen. Infolge der relativen Volumenkonstanz des inkompressiblen Werkstoffs führen Vertikalverformungen solcher Lager zu Lagerausbreitungen im Lagerspalt bis zum Erreichen eines Gleichgewichtszustandes (Reibungsschluß). Wegen der nur unzutreffend erfaßbaren Reibungsverhältnisse an den Kontaktflächen werden sehr vorsichtige zulässige Beanspruchungen formuliert [1] .

Bei Lagerungen im Hochbau wird den Elastomerlagern über die gesamte Lebensdauer des Bauwerks eine vergleichsweise hohe ständige Pressung bei vorwiegend ruhend beanspruchten Bauteilen zugewiesen. Pulsierende Beanspruchungen sind wegen der Gefahr des Wanderns aus dem Lagerspalt nicht zulässig. Üblicherweise wird die Eignung eines elastomeren Werkstoffs für seinen Verwendungszweck in Kurzzeitversuchen nachgewiesen. Für die Langzeitbeanspruchung unter ständigen sehr hohen Pressungen existieren jedoch auch in anderen Ingenieurbereichen keine gesicherten Erkenntnisse.

Für Lagerungen, bei denen im Falle der Überbeanspruchung oder des Ausfalls der Lager die Standsicherheit des Bauwerks nicht gefährdet ist [2] , oder in Stützenstößen, werden den Lagern häufig höhere mittlere Pressungen als nach den geltenden Baubestimmungen zulässig zugewiesen [3, 4] . Dabei stehen im ersten Fall wirtschaftliche Kriterien, im zweiten Fall vorwiegend konstruktive einschränkende Randbedingungen im Vordergrund.

2 WERKSTOFFE FÜR LAGERUNGEN IM HOCHBAU

2.1 Elastomerqualitäten

In der Bundesrepublik Deutschland hat sich im bauaufsichtlichen Bereich des Hoch- und Brückenbaus der Elastomertyp CHLOROPREN-KAUTSCHUK (CR) durchgesetzt. Seit Beginn der 80er Jahre sind kostengünstigere Elastomerqualitäten auf der Basis von ETHYLEN-PROPYLEN-DIENPOLYMEREN (EPDM) auf dem Markt, vereinzelt wurden für diese Werkstoffe bauaufsichtliche Zulassungen erteilt. Im Normentwurf für die bauliche Durchbildung und Bemessung unbewehrter Elastomerlager [3] sind EPDM-Qualitäten als Werkstoff nicht vorgesehen, und für die bereits zugelassenen EPDM-Qualitäten gelten nach [4] geringere zulässige Beanspruchungen. Für solche Lagerungen, bei denen bei Ausfall der Lagerfunktion die Standsicherheit nicht gefährdet ist, ist nach einem geeigneten Nachweis auch die Verwendung anderer Qualitäten nicht ausgeschlossen [2] .

2.2 Werkstoffverhalten von Elastomeren unter hoher Beanspruchung

Elastomere Werkstoffe bestehen aus weitmaschig vernetzten Molekülketten. Verformungen bewirken Relativbewegungen der Ketten und innere mechanische Beanspruchungen der Vernetzungsstellen. Innerhalb bestimmter Dehnungen weisen Elastomere zeitabhängig reversibles elastisches Verhalten auf, bei Entlastung gleiten die Polymerketten langsam in den Ursprungszustand zurück. Dagegen führen hohe mechanische Beanspruchungen zu bleibenden plastischen Verformungen infolge chemorheologischer Effekte wie

- irreversible Umstrukturierung der Moleküle
- Bruch von Polymerketten oder Vernetzungsstellen
- Mikrohohlraumbildung als Folge von Molekülkettengleiten oder -bruch
- Ablösen der Polymermatrix von den Füllstoffpartikeln.

Veränderungen in der chemischen oder physikalischen Struktur äußern sich u. a. in der Abnahme der Bruchspannung, in irreversiblen Deformationen oder in der Beschleunigung des Kriechens. Solche Veränderungen sind zeitabhängig und können in der Regel im Kurzzeitversuch auch unter verschärften Versuchsbedingungen nicht beobachtet werden.

3. EXPERIMENTELLE UNTERSUCHUNGEN

Für die baupraktisch orientierte Fragestellung der Eignung und der zulässigen Beanspruchung von CR- und EPDM-Elastomerqualitäten unter hohen Beanspruchungen wurden an allen derzeit auf dem deutschen Markt befindlichen normgerechten oder bauaufsichtlich zugelassenen Elastomerqualitäten Dauerstandversuche durchgeführt. Tabelle 1 gibt Aufschluß über die zugrunde liegenden Versuchsparameter.

Elastomertyp	EPDM						CR				
Elastomer-qualität	1	2	3	3.1	3.2	4	1	2	2.1	3	4
Lagerfläche A mm^2	100 x 100				100 x 200				200 x 200		
Lagerdicke t mm	5						10				
Drehwinkel α	0						0.3 t/a				
mittlere Pressung N/mm^2	20				40				60		

a : kleinere Lagerseite bzw. Seite rechtwinklig zur Drehwinkelachse

Tabelle 1: Übersicht über die Parametervariationen (nicht vollständiger Faktorversuch)

Für die spezielle baupraktische Fragestellung ist die interaktive Wechselwirkung zwischen Lagerverformung und Reibungsschluß an den Betondruckflächen von besonderer Bedeutung. Als Kontaktflächen wurden daher stahlgerahmte Feinbetonscheiben verwendet /5/.

4. BEURTEILUNG DES LANGZEITVERHALTENS

4.1 Kriechneigung

Unter hoher Belastung stellt sich bei Elastomeren nach bestimmter Belastungsdauer eine mehr oder weniger ausgeprägte Zunahme der Kriechgeschwindigkeit ein, ähnlich wie dies bei anderen Werkstoffen in der Nähe der Zeitstandfestigkeit beobachtet wird. In den Bildern 1a und 1b sind die Kriechkurven aus allen Versuchen differenziert nach den Elastomertypen EPDM und CR dargestellt. Zur Formulierung trendmäßiger Aussagen bleiben an dieser Stelle einzelne Parameterausprägungen unberücksichtigt.

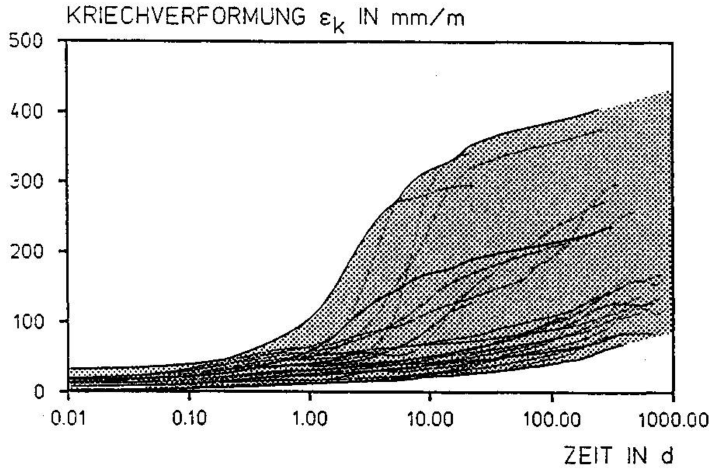

Bild 1a: Kriechkurven aus allen Versuchen mit EPDM-Qualitäten

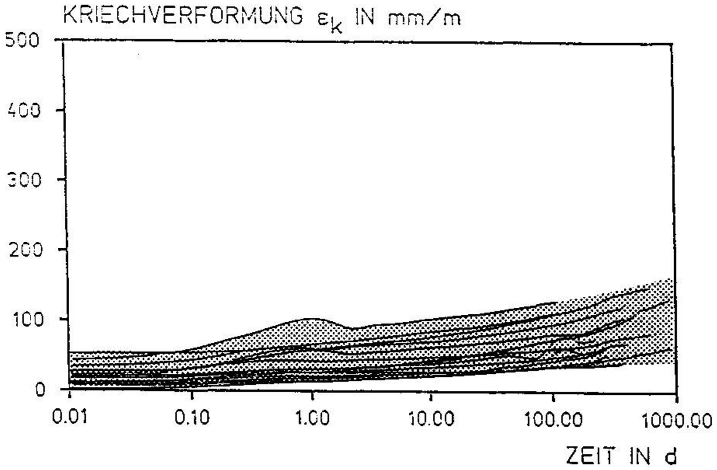

Bild 1b: Kriechkurven aus allen Versuchen mit CR-Qualitäten

Die Bilder 1a und 1b illustrieren, daß

- EPDM-Qualitäten gegenüber solchen aus CR stärkere Kriechneigung aufweisen. Dies gilt für die Kriechverformung und -geschwindigkeit gleichermaßen.
- EPDM-Qualitäten innerhalb der zugrunde liegenden Parametervariation einen relativ großen, CR-Qualitäten dagegen einen engen Streubereich aufweisen.
- nach einer Belastungsdauer deutlich länger als 100 Tage insbesondere EPDM-Qualitäten einen progressiven Kriechanstieg zeigen.

Ein progressiver Kriechanstieg deutet auf Versagensvorgänge in der Mikrostruktur hin, die nach bestimmter Beanspruchungsdauer zu visuell erkennbaren äußeren Veränderungen des Lagerkörpers führen.

4.2 Schädigungsgrad der Lager und maximale rechnerische Schubspannungen

Aus visuell an den Lagerkörpern beobachtbaren Schädigungsmerkmalen

- Riss
- Oberflächentextur
- Ablösen der Randschicht
- Gefügezerstörung

kann entsprechend der Wichtigkeit des Merkmals für die dauerhafte Lagerfunktion ein sogenannter "Schädigungsgrad" der einzelnen Lagerprobe gefunden werden. Die Vergleichbarkeit der Versuche mit unterschiedlichen Parametern kann durch eine rechnerische maximale Schubspannung, die von der mittleren Pressung, der Lagerfläche, der Lagerdicke und vom Drehwinkel abhängt [6] , herbeigführt werden. Diese Schubspannung ist in starkem Maße von der Lagergeometrie (Formfaktor) abhängig. In Bild 2 ist der Zusammenhang zwischen Schädigungsgrad und Schubspannung für eine EPDM-Qualität mit hoher Kriechneigung (EPDM 1) und eine CR-Qualität mit geringer Kriechneigung (CR-1) zusammengefaßt.

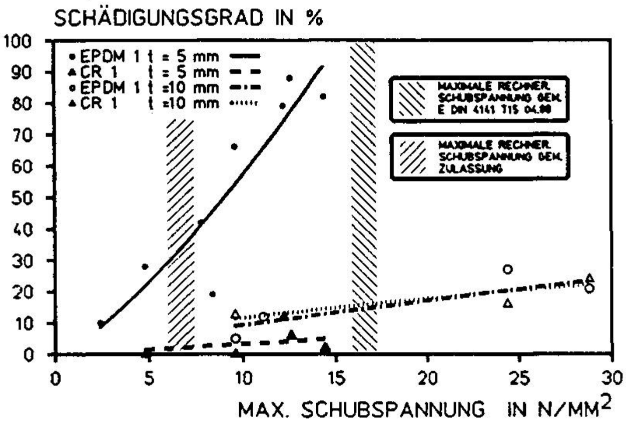

Bild 2: Schädigungsgrad in Abhängigkeit von der maximalen Schubspannung EPDM 1 und CR 1

In das Diagramm sind die aus den zulässigen Beanspruchungen der Zulassungen [3] bzw. der Norm [4] resultierenden maximalzulässigen Schubspannungen eingetragen. Danach führen Beanspruchungen oberhalb derjenigen der Zulassungen bei dünnen EPDM-1 Lagern zu Schädigungen, die die dauerhafte Lagerfunktion in Frage stellen. Aufgrund der speziellen Definition des Schädigungsgrades kann ein Schädigungsgrad bis 25 % für die Lagerfunktion noch als akzeptabel angesehen werden. Innerhalb der normgerechten Beanspruchung zeigen dünne CR-1 Lager vernachlässigbare Schädigungen. Bei 10 mm dicken Lagern aus EPDM 1 und auch bei 5 mm dicken Lagern aus CR 1 werden oberhalb der zulässigen Beanspruchungen der entsprechenden Norm Schädigungen an der Akzeptanzgrenze registriert.

4.3 Beurteilungskriterium für das Langzeitverhalten

Eine im Bauwesen geläufige Beschreibung des Kriechverhaltens ist die Extrapolation nach Ross; das einfach handhabbare Verfahren scheint auch für Polymere geeignet [7] . Das Verfahren unterstellt eine hyperbolische Kriechfunktion und gilt für einen Bereich mit konstanter Kriechgeschwindigkeit. Dies trifft bei der Mehrzahl der vorliegenden Kriechkurven bis zu einer Belastungsdauer von einem Tag zu (Bilder 1a und 1b). Aus diesem Zeitintervall wird ein die Kriechneigung charakterisierendes Endkriechmaß bestimmt. Die Dauerhaftigkeit der Lagerfunktion kann wesentlich durch eine aus

- dem Endkriechmaß (aus dem Zeitintervall bis zu einem Tag)
- dem Schädigungsgrad (in %)

abgeleitete Rechengröße, dem "Gütewert" (GW) beurteilt werden. Dieser Gütewert wird nach folgendem Zusammenhang ermittelt:

GW = (Endkriechmaß/Schädigungsgrad) . 100.

In Bild 3 ist der Zusammenhang zwischen dem Gütewert und der maximalen rechnerischen Schubspannung für die EPDM 1 und die CR 1 Qualität in Abhängigkeit von der Lagerdicke aufgetragen.

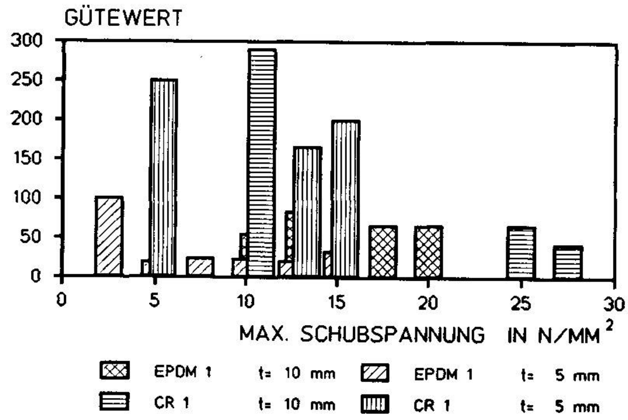

Bild 3: Gütewerte für EPDM 1 und CR 1 Lagerproben

Die Darstellung illustriert, daß EPDM 1-Lager unabhängig von der Lagerdicke innerhalb eines Schubspannungsbereiches zwischen 5 N/mm² und 15 N/mm² Gütewerte kleiner 50 annehmen. Für die dauerhafte Lagerfunktion unter hohen Beanspruchungen muß ein Gütewert größer 75 erwartet werden.

5. BAUPRAKTISCHE SCHLUßFOLGERUNGEN

Aus den vorliegenden Untersuchungsergebnissen können nachfolgende Hinweise für die Verwendung unbewehrter Elastomerlager unter hohen Beanspruchungen gewonnen werden:

- CR-Qualitäten weisen ausreichendes Dauerstandverhalten bis zu mittleren Pressungen von 20 N/mm² auf.
- EPDM-Qualitäten zeigen innerhalb der zugrunde liegenden Versuchsparameter relativ große Streuungen in der Kriechverformung und kleine Gütewerte auf. Innerhalb geringer Beanspruchungen muß bei diesem Elastomertyp unter Umständen mit hohen Schädigungen des Lagerkörpers gerechnet werden. Solche Qualitäten sollten nicht ohne besonderen Nachweis für hochbeanspruchte Lagerungen verwendet werden.

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