Conference Proceedings

NDT assessment and new systems in prestressed concrete structures
proceedings of the First Workshop of COST 534 on NDT Assessment and New Systems in Prestressed Concrete Structures

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First Workshop of COST 534

NDT assessment and new systems in prestressed concrete structures

13 October 2004
Swiss Federal Institute of Technology, ETH Zurich
Switzerland
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Preface

Prestressed concrete structures form an important part of Europe’s physical infrastructure for transportation, energy production, nuclear power plants, water and waste-water treatment and buildings. Especially for the transportation infrastructure, most major large span bridges (roads and railways) and also segmental bridges are post-tensioned structures. This method of construction has now been used for 50 years. The increasing age, the environmental impact and increasing traffic load may lead to problems regarding the durability of post-tensioning tendons as documented by a small, but significant number of failures and collapses all over Europe.

The COST action 534 “New Materials and Systems for Prestressed Concrete Structures” started in December 2002. According to the memorandum of understanding five working groups are active in materials research topics important for durability and safety of new and existing prestressed concrete structures (see www.cost534.com). One of the declared scopes of the new COST 534 action is to interact and to exchange information between COST and the civil engineering world. For this reason the first COST 534 workshop “NDT assessment and new systems in prestressed concrete structures” was held together with the fib/IABSE workshop on “Durability of post-tensioning tendons” at the Swiss Federal Institute of Technology (ETH) Zurich. Many of the COST participants took the opportunity to attend sessions on „Design“, „Materials and Construction“ and „Maintenance, Assessment and Rehabilitation“ based on the fib draft report „Durability Specifics for Prestressed Concrete Structures“ closely related to the activities in COST 534.

The third day of the workshop was organized by COST 534 and dedicated to two main topics out of the working program: “New assessment methods” (WG 3) and “New Systems” (WG 2). In the morning session progress and problems in the NDT methods for assessment of existing structures were presented and discussed, focusing on acoustic monitoring and on magnetic and sound methods. In the afternoon session a new post-tensioning system using plastic ducts and electrically isolated tendons and the experience so far was presented.

The organisers express their sincere thanks to the experts acting as speakers or chairmen and to all participants for the discussions allowing further progress to be made in this important field of prestressed concrete construction. They also gratefully acknowledge the financial support obtained from the COST / ESF office in Brussels and of the COST office Switzerland.

It is hoped that this first COST 534 workshop will be a success not only from a scientific point of view but that it will also allow the participants to enjoy these days in Zurich.

Bernhard Elsener           Edoardo Proverbio
Group leader WG 2          Group leader WG 3
Chairmen of the organising committee
Proceedings of the first workshop of COST 534 on
NDT Assessment and New Systems in Prestressed Concrete Structures
13. October 2004, Swiss Federal Institute of Technology, ETH Zurich, Switzerland

Host

The first COST 534 Workshop “NDT assessment and new systems in prestressed concrete structures” is held at the Swiss Federal Institute of Technology, ETH Zurich, Main Building, Rämistrasse 101, located in the centre of Zurich. See also www.ethz.ch.

Organising Committee

B. Elsener  Swiss Federal Institute of Technology ETH Zurich, Institute of Building Materials, Materials Chemistry and Corrosion (IBWK)
E. Proverbio  University of Messina, Faculty of Engineering
M. Nick  Swiss Federal Institute of Technology ETH Zürich, Institute of Building Materials, Materials Chemistry and Corrosion (IBWK), Secretary

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Sponsoring Organisations

European Science Foundation COST Office, Brussels
Swiss Federal Office for Education and Science, COST Office, Berne
Swiss Federal Institute of Technology, Zurich

The organisers express their sincere thanks to the sponsors for their financial, logistic and technical support.
Program

Tuesday, 12 October 2004
19.30 Workshop Dinner

Wednesday, 13 October 2004 in the Main Building, Room HG F1
08.30 Registration / Coffee
09.00 Welcoming address B. Elsener
   What is COST 534 R. Polder (chairman of MC)

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How to reach the Main Building of the ETH Zurich

Zurich can easily be reached by plane or by train. Arriving by plane, we recommend to take the train connection from the airport directly to Zurich main station (approx. 10 min). From the main station the hotels and the conference venue can be reached by public transport or taxi.

Location of ETH in the City Centre of Zurich

From Zurich Main Station you arrive in a few minutes with tram number 6 directly in front of the ETH Zentrum (Main Building). The information desk is situated on your left as you enter the building.

Workshop venue: ETH Main Building, Rämistrasse 101
Lecture room : HG F1 in the main building
Acoustic emission monitoring of reinforced concrete elements - an overview

Leszek GOLASKI
Kielce University of Technology, Poland

1. Introduction

The advantages of AE technique as NDT method are:

- Volume inspection
- Localization of deteriorated zones
- On line data evaluation
- Intensity of deterioration development estimation
- Detection of damages that are under development only

However, AE method characterizes the defect severity with the help of parameters of acoustic emission (AE) signals that are produced by defect development. Therefore interpretation of results is complicated and experienced staff is necessary.

The scheme of AE signal is shown in Fig. 1. Its parameters are:

- Counts number
- Amplitude
- Signal duration
- Rise time
- Signal strength
- RMS voltage

![Fig. 1 AE event and its parameters](image-url)
On the basis of AE activity following additional parameters are determined. They are:

- Felicity ratio (LOAD ratio)
- CALM ratio
- Historic Index
- Severity

Felicity ratio (FR) – The ratio of the load during subsequent loading at which detectable AE is present to the previously applied maximum load.

LOAD ratio - it means acronym: Ratio of load at the onset of AE activity to previous load.

CALM ratio - it means acronym: The number of cumulative AE activity during the unloading process to that of the last maximum loading cycle.

Historic Index at time t and Severity are given by equations:

\[
H_{(t)} = \frac{N}{N-K} \sum_{i=1}^{N} \frac{S_{oi}}{\sum_{i=1}^{N} S_{oi}}
\]

\[
S_{fr} = \frac{H_{(t)}}{J} \sum_{m=1}^{m=J} S_{om}
\]

where:
- \(N\) – number of AE events
- \(K\) – a constant that depends on \(N\)
- \(S_{oi}\) – strength of the \(i\)-th signal
- \(J\) – experimentally determined constant

Historic Index compares the strength of recent AE signals to the strength of all of them. A knee on \(H_{(t)}\) vs. time diagram may indicate a new damage mechanism. It is independent of dimension of element. Severity is defined as the mean strength of a number of signals with the largest strength.

To estimate the structure integrity and damage evolution usually the AE parameters as below are used:

- Beginning of AE activity or knee on AE summation diagrams
- Large amplitude AE events appearance
- Number of large amplitude AE events
- High energy AE events
- AE activity during load hold
- RMS voltage
- Summation of events duration
- Felicity ratio
- CALM ratio
Historic Index

Severity

Evaluation is on a per channel basis mostly. AE parameters are presented in different type of diagrams: summation, intensity, points etc. vs time. The EA parameters are evaluated separately or in multi-parameters space.

AE technique is applied for integrity evaluation of different structures. For some structures special systems have been developed. Except integrity evaluation the systems make possible to estimate the residual strength and remaining life of structures. The basic technique is AE however, if necessary, other NDT techniques are applied. So far AE systems were developed for:

- FRP tanks/vessels
- Bottom of on ground storage tanks
- Pressure vessels

The Committee on Acoustic Emission from Reinforced Plastics (CARP) Division of the Technical Council of The American Society for Nondestructive Testing has developed first evaluation system [2, 3, 4, 5]. This system serves as a model for the next two and other procedures. However, it is worthy to notice the caution of CARP procedure authors that "...it is very difficult to apply laboratory generated data as a quantitative measure of acoustic emission sources in full size structures." and "Moving the technology from the laboratory to the field proved to be difficult...." [4]. Validation of procedures are necessary and to perform this large database is necessary, where results of field examination of structures are stored. These databases should contain signal patterns typical for specific damage mechanisms, reference signals etc. Unfortunately these database are confidential.

Our study shows that for bridges the obstructions in laboratory data transfer to the field tests can be much greater in comparison to composites.

AE in nondestructive testing of reinforced concrete has been used for more than 40 years [6]. Most of experiments were made on model beams and other laboratory tests pieces. Limited number of prestress girders was examined. On the basis of these results two procedures for concrete structures monitoring by acoustic emission have been developed. These procedures are:

- "Procedure for Acoustic Emission Monitoring of Prestressed Concrete Girders" by The Ferguson Engineering Laboratory, The University of Texas and accepted by Texas Department of Transportation 2001 concerning prestress girders
- Recommended Practice for In Situ Monitoring of Concrete Structures by Acoustic Emission, NDIS 2421, Japanese Society for Non-Destructive Inspection 2000 for concrete structures.

In the first one the deterioration is evaluated using Felicity ratio and AE event amplitude. Four categories of AE activity are distinguished and corresponding four levels of deterioration significance:

- Not significant
- Minor
- Warning
- Serious (amplitude >75dB, Felicity ratio<0.6).

![Diagram of CALM and LOAD ratios](image)

In the second procedure deterioration significance is evaluated with the help of both CALM and LOAD ratios. The criteria are presented in Fig 2. In this figure the position of dot lines are given approximately.

These procedures employ only two among possible AE parameters. However, they perform valuable approaches because deterioration criteria and demands concerning instrumentation and test performance are formulated.

### 2. Damage evaluation by acoustic emission technique

There is poor database concerning AE monitoring of bridges. Since 5 years some efforts in this direction have been made at Kielce University of Technology. At this time complementary AE tests have been performed on material samples, model beams and full-scale girders. Our selected results are given in Journal of Acoustic Emission [7]. In this paper an overview concerning AE method application, possibilities and results concerning laboratory tests, full-scale prestressed girders and field tests on bridges are given. All below presented results are from KUT laboratory.
2.1 Model beam tests

Damage testing of model beams can provide valuable general information concerning AE behavior of concrete structures under different loading conditions. The models we have used were concrete beams (1500x200x100) reinforced with 4 rebars (10 mm in diameter). The nominal failure load was 70 kN. Below given results concern damage evolution during incremental loading cycles in three point bending. In each cycle maximum load was hold at constant level during 4 minutes.

AE parameters that were recorded are:
- Number of events
- Amplitude
- Duration
- Signal strength

The results are demonstrated in Figs. 3, 4, 5. In Fig 3 event sum and load are plotted vs. time. The beginning of AE took place at P = 12.3 kN. It is worthy to notice AE activity during load hold. AE vanished at the beginning of unloading. During first two subsequent loading cycles AE started at higher load than maximum load in previous one. Thus Felicity Ratio (FR) in first cycles were greater than 1. During first load holds AE usually increased with time. When model is unloaded the AE activity decreases and next disappears. With the number of cycles increasing the AE during unloading disappear at relatively lower load. AE event summation provides important information concerning damage development. However, to learn more about deterioration that takes place during loading additional AE parameters should be recorded. Usually they are amplitude and duration. The amplitude and duration point diagrams are shown in Fig 4 and Fig 5 respectively for selected loading cycle.
While changes in amplitude are insignificance with load, duration increases more than 100 times. Significant changes in duration may indicate changes in damage mechanisms. Usually the sources of AE activity with long duration are friction between concrete and reinforcement.

In the tested concrete reinforced model beams AE activity takes place during all loading holds. It is in contrary to composite structures, where AE is vanishing during first few (usually 2) minutes. This AE behavior demonstrates that deterioration processes in this beam take place at constant load too.

It should be noticed that the results of AE testing might be influence by numerous factors. For example: sensor location, sensor characteristic, not precisely defined “beginning of AE activity”, rebars position (they can work as a waveguides) etc.
2.2 Stress corrosion of FRP

AE technique application for corrosion damage testing of on-ground oil reservoirs and oil pipelines has been reported in a number of papers. However, corrosion of steel reinforcement in concrete elements and cracking of concrete structures due to corrosion were not studied extensively by AE and only selected papers are available. Evaluation of corrosion by AE in laboratory scale was performed by Ohtsu and Tomola [8]. Their researches demonstrate that corrosion process of steel rebars produces AE events that can be detected. Moreover this technique can detect initiation of stress corrosion of rebars, initiation of crack in concrete and crack propagation acceleration.

Our studies mostly concern stress corrosion of FRP that can be used for reinforcement of deteriorated concrete structures. Stress corrosion takes place both in acid and alkaline environment although the patterns of corrosive pits are quite different [9]. Damages, that take place as a result of stress corrosion, reduce composite stiffness and thus decay the reinforcing effect in reinforced concrete structures by FRP. For bridge owners and users it is important to know the change of FRP compliance and the durability of composite reinforcement. For these purposes we used AE analysis. The study was performed on glass fiber epoxy composites immersed in corrosive liquid with pH8 and pH12. This concentration corresponding to that, which are meet in fresh and corroded concrete. The samples were tensioned with constant load and AE events were recorded up to failure. The elongation as a result of corrosion was recorded too. A relationship between elongation and AE activity can be seen in Fig 6.

A relationship between changes of compliance and AE events summation was observed with acceptable statistic as it can be seen in Fig 7. This relationship depends on environment. Because the pH concentration in concrete during service of bridges is not known, such relationship for all tested samples were developed. However, for this relationship the statistic is not so good. Time to
samples fracture and AE event summation were used to evaluate the durability of FRP samples under environment and stress conditions. It was stated that while time to fracture was characterized by significant dispersion, the dispersion of event summation up to fracture was pronounced lower.

![Fig.8 Typical AE signals and their FFT from a) composite cracking and b) concrete cracking](image)

The tests were made on composite only. In fact reinforced concrete with the help of FRP under stress corrosion condition produces AE from both concrete and composite. To separate FFT of AE signals can be used. Typical signals and their FFT from concrete and composite are demonstrated in Fig. 8.

This study was performed on composite samples and concrete samples only. Experiments on concrete reinforced by FRP are under preparation on larger samples.

### 2.3 Selected results from girder testing

The result given below concern girder that was designed as continuous beam with supports within its span. The girder length was 18.4 m and its height was 1 m. It was reinforced by 16 wires. Parts of some wires, as it is shown in Fig. 9, were unstressed. The scheme of loading that is shown in Fig. 9 was changed in relation to designed one. Eleven AE sensors were located on the bottom surface of girder and zonal location was applied. Numbering of channels coincide with numbering of zones and AE sensors. The beam was loaded in 6 sequences (load is given in kN): I - (0 - 157 - hold - 0); II - (0 - 314 - 0); III - (0 - 195 - 0); IV (0 - 308 - 0); V (0 - 602 - hold - 0) and VI (0 - 682 i.e. collapse). In each sequence load was applied in some steps ~15 kN each. During intervals between loading the girder was examined visually to detect the cracks.
AE activity started at low load level. The strongest AE took place in zone 5 and 7. It should be noticed that this "noisy zones" produced AE signals with amplitudes > 70 dB during loading and > 60 dB during load hold, even when the load was equal ~10% of failure loading. Typical AE activity at higher load (~60% of loading capacity) is shown in Fig. 10 while the AE for last but one loading sequence (632.5 kN) is shown in Fig. 11.
In Fig. 10 first 160 sec. concern load hold while time interval of 160-240 sec corresponds to load increasing. In zones 5, 6, 7 and 8 amplitude exceeded 80 dB. Load increasing (Fig.11) did not change quality of AE activity. In this loading sequence stronger AE appeared in Channels 2, 3 and 10. Generally some increasing in strength of signals was observed. However, number of hits in time unit might to be lower at higher load. In this particular girder it was difficult to indicate, on the basis of AE analysis, the load level, at which severe damage took place. Undoubtedly AE indicated that the deterioration process took place at very low load and exhibited the evolution of damage with load in zones of girder.

Usually when strong AE took place during a loading step, subsequent loading step produced lower AE. Generally with load increasing the amplitude and energy of AE signals increased however, this was not a continuous process. Frequently AE was seen in the form of high-energy AE signals "eruption". This eruption more distinctly can be seen in duration vs. time point diagrams.

First tiny cracks took place during IVth loading sequence in zones 3 and 9 at 308 kN. Cracking was accompanied by single AE event with amplitude ~80 dB. Next 9 cracks appeared in the load range from 308 kN to 340 kN (load sequence V). Further 23 cracks took place in the load range 340-422 kN. At higher loads number of cracks didn't change although their length increased with load. The crack distribution is shown in Fig. 12.

The results of examination of this girder indicate that parameters, which usually are taken for evaluation (amplitude, Felicity ratio) exceeded their critical values at comparatively low load. On
the other hand the energy parameters of recorded AE signals are rather low. Therefore, it is reasonable for damage severity evaluation to consider more AE parameters.

Fig. 12. Crack distribution in tested girder at load > 340 kN

2.4 Field-tests on bridge

We tested 7 full-scale concrete reinforced structures. They were old and new bridges, bridges before and post repairing, bridge during overloading and concrete reinforced elements of industrial building. Below there are reported selected results concerning prestressed and post tensioning bridge.

The prestressed bridge was monitored during service. Because of high attenuation 11 AE sensors were located on bottom surfaces of girders as it is demonstrated in Fig. 13. For damage localization zonal location was applied.
The bridge was monitored during 1 hour under heavy traffic. For damage evaluation number of AE events, their duration and amplitude were taken into consideration. As example the AE amplitude point diagrams for each zone are shown in Fig. 14.

On the basis of these AE parameters three levels of damage were selected: minor, warning and serious. They are indicated in Fig. 13. The main criterion was AE events sum however, two other parameters were considered too. Zones, where events sum exceeded twice the average value for all zones, are considered as seriously damaged. Warning damage are zones, where events sum exceeded the average value for all zones. In this classification the pattern of amplitude and duration vs time diagrams and staff experience were taken into consideration too. Visual examination did not revealed cracks however, the surface layer of concrete in seriously damaged zones was strongly corroded.

Fig. 14: AE sensor location and damage level in girder

Fig. 14. AE amplitude point diagram vs time for prestressed girder
Visual examination of post-tensioning bridge exhibited a number of different types of damages. Most of them took place as a result of tendons corrosion. AE monitoring was performed under traffic and under proof load, equals the loading capacity of bridge. The lorries drive slowly on smooth surface and on surface with obstacles. Next the cars were stopped abruptly on the bridge. At the end two lorries were situated on the bridge. During all of these loading AE was recorded and zonal location was applied. It should be noticed that most of visually detected damage did not produced strong AE.

In Fig. 15 AE amplitude and energy point diagrams that were recorded during stationary load with two lorries are shown. After two minutes eruption of very strong AE was recorded. High energy is a result of long duration of signals. The zone, in which this strong AE took place was examined and no cracks could be seen. The AE parameter analysis suggests that a new crack between cable and concrete took place and next propagated inside the beam.

![Graph](image)

Fig. 15. AE amplitude and energy point diagram recorded during examination of post-tensioned bridge under constant load.

Generally the damage processes that produces AE signals eruption indicated in duration or energy point diagrams can produce serious deterioration. This deterioration can be danger for structure. This AE behavior usually cannot be detected in amplitude point diagram.

2.5 Intensity analysis

Our experiences with AE application as NDT technique indicate that limited information are available when amplitude, Felicity ratio and event summation are considered only. Promising seems to be analysis that employs intensity parameters: Historic Index and Severity parameters that are defined by equations (1). Both indexes depend on signal strength and thus on amplitude and duration. To evaluate damage level of composite structures these intensity parameters are
plotted in $S_r - H_s$ intensity chart. This chart (shown in Fig.17) is divided into 5 area corresponding to different damage levels. Bottom area characterizes zone without damage while upper area indicates major structural damages. Higher values of Historic Index may indicate new damage mechanism. At high $H_s$ comparatively low $S_r$ may detect serious damage.

There is question, if these parameters can describe damage evaluation in concrete reinforced structures. Some positive experiences are given [7]. However, more data on concrete structures that differ in structure, reinforcement, materials properties and damage level are necessary.

Below there are discussed $H_s$ and $S_r$ that were calculated at different loading level of above described model beam and full scale girder.

The intensity parameters for reinforced concrete model beam were calculated in selected time intervals during two last loading cycles. Results are given in Table I while Fig. 16 shows positions at which these parameters were calculated.

**Table I.** Values of Historic Index and Severity in dependence on time for model beam

<table>
<thead>
<tr>
<th>Point No</th>
<th>Time interval [sec]</th>
<th>Historic Index</th>
<th>Severity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>8108.82 8136.35</td>
<td>1.64</td>
<td>0.537</td>
</tr>
<tr>
<td>2.</td>
<td>8136.39 8199.43</td>
<td>0.956</td>
<td>1.358</td>
</tr>
<tr>
<td>3.</td>
<td>8497.70 8560.07</td>
<td>5.666</td>
<td>2.282</td>
</tr>
<tr>
<td>4.</td>
<td>8560.07 8707.86</td>
<td>0.295</td>
<td>2.443</td>
</tr>
<tr>
<td>5.</td>
<td>9393.08 9446.10</td>
<td>3.343</td>
<td>1.659</td>
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<tr>
<td>6.</td>
<td>9446.12 9592.75</td>
<td>0.616</td>
<td>1.952</td>
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<tr>
<td>7.</td>
<td>9495.91 9700.84</td>
<td>1.594</td>
<td>2.245</td>
</tr>
<tr>
<td>8.</td>
<td>10032.83 10481.76</td>
<td>1.55</td>
<td>3.983</td>
</tr>
</tbody>
</table>

![Diagram](image)

Figure 16: Load and strength signal sum vs time for model beam; the number indicates places where $H_s$ and $S_r$ were calculated
It can be noticed that Historic Index and Severity increased with time during loading hold although their values are comparatively low. At this time serious damages of model could be seen. This suggests that for concrete reinforced by rebars the values of critical intensity parameters are lower than for prestressed structures.

The intensity parameters for examined girder, for selected loads and channels, are given in Table II.

**Table II.** Intensity parameters for girder in selected zones

<table>
<thead>
<tr>
<th>Point number</th>
<th>Load [kN]</th>
<th>Channel</th>
<th>Historic Index</th>
<th>Severity</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>600,0</td>
<td>2</td>
<td>5,06</td>
<td>9,22</td>
</tr>
<tr>
<td>2</td>
<td>632,5</td>
<td>10</td>
<td>28,94</td>
<td>1,73</td>
</tr>
<tr>
<td>3</td>
<td>681,2</td>
<td>10</td>
<td>0,33</td>
<td>23,13</td>
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<td>4</td>
<td>681,2</td>
<td>7</td>
<td>45,07</td>
<td>31,27</td>
</tr>
</tbody>
</table>

The data from Table II are inserted into intensity chart that was developed for composite [4]. This chart is presented in Fig.17. The data for zone 7 (point No 4) correctly predict the failure of girder. However, it should be noticed that \( H \) strongly depend on time range position. Shifting this position a little noticeable changes in Historic Index can be obtained. Therefore precise instruction concerning this calculation is necessary.

![Fig.17. Intensity chart for tested girder. Inserted numbers concerns data indicated in Table II.](image)

The intensity chart shows that with increasing Historic Index the critical value of Severity is decreasing. There is particular value of \( H \) when comparatively low \( S \) indicates serious damage that makes impossible safety service of structure.

AE behavior of these model beam and girder indicate that the intensity chart developed for composite structure cannot be used in present form as an universal damage criterion. The tests that are given above show that \( S \) at failure for concrete reinforced by steel bars are lower in
comparison with $S_r$ value for prestress full-scale girder. Nevertheless it seems that intensity approach is a valuable method for damage level evaluation.

3. Conclusion

Concrete reinforced elements were examined by AE technique to reveal and localize defects, and damage evolution. Laboratory scale examination was performed on samples/models and full-scale girders, while field tests were performed on bridges of different structures. Generally these experiments approved the usefulness of AE method for deterioration evaluation of concrete reinforced structures. AE activity may be used to localize the damage zones and to estimate the intensity of damage development in each zone. Multi-parameter analysis of AE signals may help to identify the mechanisms of damage.

AE technique was used to evaluate stress corrosion of glass reinforced composites that can be used as a reinforcement of concrete structures. Stress corrosion of glass fibers was detected both in acid and alkaline environment. It was proved that AE event summation could be employ to evaluate changes in stiffness of composite and service life prediction. Differences in FFT of AE signals from concrete and composite may make possible to separate AE signals from composite and concrete. Further study on concrete reinforced by glass reinforced plastics under influence of stress and corrosive environment is desired.

AE examination of full-scale girders can locate and estimate the category of damage however, this evaluation should be made with caution. This is because AE activity can depend on factors, which influence on AE sources are not fully recognized. These factors for example are: type of reinforcement, concrete microstructure and properties, loading mode, residual stresses etc. The performed experiments show that it is difficult to select uniform quantitative criteria for damage evaluation.

The field tests performed on full size structures confirmed the usefulness of AE technique as a tool to localize the damaged zone and to evaluate the severity of defects. Only defects under development or defect initiation are indicated.

Laboratory generated data have limited application for defect severity assessment in full size structures because of scale effects. In addition the laboratory data are taken from new samples and elements that usually are specially prepared, while the field tests are performed on real old structures containing different defects that interact. Therefore laboratory tests on old samples and old beams are desired. In laboratory testing the loading is displacement controlled while real structures are load control. Differences in loading control may influence AE activity, especially at high loads close to failure load. These factors make difficult to move criteria developed on samples for defect severity evaluation in field tests of old concrete structures.

AE data recorded during all performed tests on bridges suggest that intensity parameters (Historic Index and Severity) can be taken as a quantitative measure of acoustic emission sources and to serve as a measure of deterioration of structures. To develop this measure a validation should be made on real structures. A database have to be created, where complete data that are recorded
during AE monitoring of different concrete reinforced bridges (not only prestressing), are stored. This database is organized at TU Kielce.

**Literature references**

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The Use of Acoustic Monitoring to Manage Concrete Structures

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Abstract

Concrete and steel are widely used in civil structures. Both are excellent acoustic transmitters. In many structures tensioned wire elements are used. However, tensioned wire can be vulnerable to corrosion. To reduce the probability of corrosion sophisticated protection systems are used. To confirm that the design strength is available through time, extensive inspection and maintenance regimes are implemented.

Due to severe problems observed at later stages on several structures on a world wide basis, technologies have been developed to monitor continuously prestressing failures and concrete cracking on key structures.

This paper presents two case studies of an acoustic monitoring technology which uses widely distributed sensors to detect and locate wire failures using the energy released at failure. The technology has been used on a range of structures including post-tensioned concrete bridges, suspension bridges, buildings, Prestressed Concrete Cylinder Pipelines (PCCP) and Prestressed Concrete Containment Vessels (PCCV), where it has increased confidence in structures and reduced maintenance costs. Several projects have been completed on civil engineering structures: several suspension bridges in the USA, prestressed bridges (internal and external prestressing) throughout Europe etc...

This paper describes this type of technologies, details of two cases showing, how the information given by this tool can be used as a management tool to mitigate significantly structural maintenance and repair costs. One application is on a viaduct in the Netherlands the other is on a bridge in Belgium.

1. Acoustic monitoring system

1.1 Introduction

Continuous acoustic monitoring has been used since 1994 to monitor failures in bonded and unbonded tendons in post-tensioned structures, where it has shown major benefits in confirming the performance of structures, increasing Client confidence and reducing maintenance costs. To extend the application of this technology to the monitoring of concrete cracking required that
effectiveness of the principles and methods was evaluated for each structural type.

For acoustic monitoring technology to function in a particular environment it must be shown that the signals generated by wire failure can be detected above general noise levels and distinguished from events which are not of interest. Furthermore, to assess the structural implication of each event it is generally important to be able to locate the source of each emission. Provided with high quality data of this type, the engineer can appraise a structure with knowledge of the actual failures in damaged elements, and their location, in the entire structure over the monitoring period. The alternative, to base the assessment on a physical inspection at a sample of locations, leads to uncertainty when for practical and economic reasons the number of inspection points is limited. Monitoring the entire structure may so reveal failures not detectable by a conventional investigation.

In many applications the acoustic data is transmitted over the Internet for processing and analysis. After processing and quality control checks, the data can be made available on a secure section of a web site, allowing owners rapid independent access to their database of results. The technology is useful in providing cost-effective long-term surveillance of both unbonded and grouted post-tensioned structures.

1.2 Development of continuous acoustic monitoring

The principle of examining acoustic emissions to identify change in the condition of the structural elements is not new. However, until recently, continuous, unattended, remote monitoring of large structures was not practical or cost-effective. The availability of low-cost data acquisition and computing hardware, combined with powerful analytical and data management software, resulted in the development of a continuous acoustic monitoring system described hereafter, which has been successfully applied to unbonded post-tensioned structures in the world since 1994.

Corrosion of the steel strands in these post-tensioned structures has become a concern for designers and owners. As with grouted post-tensioned bridges, the extent of corrosion is not known, primarily because of the difficulty of identifying corrosion due to the inaccessibility of the corrosion sites, the lack of external evidence and the limited spatial coverage of intrusive inspections.

The continuous acoustic monitoring system uses the distinctive acoustic characteristics of wire breaks to separate them from other acoustic activity on a structure. Using a combination of instrumentation, data acquisition and data management, it is possible to identify events, as well to locate the failure and time of failure.

This concept allows the non-destructive identification of broken strands, so that these strands can be replaced periodically as part of a long-term cost effective structural health program. In addition, an understanding of the condition of the steel wire elements allows the life of the structure to be extended.

A typical system includes an array of sensors (Figure 1) connected to an acquisition system with
coaxial communication cable. The sensors are broadband piezo-electric accelerometers fixed directly to the concrete slab. Sensor location can be put anywhere on the slab can be detected by at least four sensors per 60 square meters for fully grouted slabs up to tendons. Multiplexing techniques are able to acquire the acoustic noise signals on 32 acquisition channels.

Using several characteristics of the acoustic events including frequency spectrum it is possible to classify wire breaks and to reject environmental noise.

By analyzing the time taken by the energy wave caused by the break as it travels through the concrete to arrive at different sensors, the software is able to calculate the location of the wire break, usually to within 300 - 600 mm of the actual location. Independent testing showed the system to be 100% correct when spontaneous events classified as "probable wire breaks" were investigated. Figure 2 shows a typical acoustic response to an unbonded wire break at a sensor 10.0 m from the break location. Figures 3 and 4 illustrate how the system locates events.
The site acquisition systems download all data automatically using the Internet to the Paris processing centre. This allows the cost of data transfer to be minimized. All data can be viewed by the owners team directly on the secure web site. This allows the owner to review areas of concern in parallel with the generation of routine reports. Various levels of alarms can be triggered semi-automatically using e-mail, automatically voice activated phone alarms, etc.

2. Monitoring of wire breaks in grouted post-tensioned bridges

Acoustic monitoring has been used in a wide range of applications including suspension and cable stay bridges, and pipelines. The technology has also been applied on many posttensioned concrete bridges around the world.

2.1 Acoustic monitoring on the Cadettenkamp viaduct in Breda (Netherlands)

Cadettenkamp viaduct in Breda is a viaduct on the A27 highway and crosses the railway connection between Breda and Tilburg. The A27 is highly used highway for cargo trucks. It is a three span structure with 4 box girders and has a total length of 75 m and is 25 m wide. The structure built in the 70's has classical internal grouted post-tensioned tendons in the walls of the boxgirders. Due to extension of the traffic capacity in terms of an extra lane. The viaduct will be strengthened by 4 external grouted post-tensioned cables.

The old prestressed cables are not in very good condition due to their brittle character. The risk that the extra load will damage the old cables is still present after installing the 4 posttensioned cables. To be able to follow the condition of the old cables under the new load characteristics, the client has decided to instrument the viaduct with the acoustic monitoring system. The system will safeguard the structure and identify and locate the damage on the tendons if it occurs.

106 acoustic sensors were placed on both sides of each box. The sensors are connected to a 32
channel acquisition unit. The sensors were placed on the outside of the boxgirders. This together with a railway passage under the viaduct made the access very difficult. The installation above the railway was done in 2 night shifts during a very short time period where the railroad was out of service. Due to these difficulties the installation including tests and set-up covered 4 weeks.

Figure 6: Test impact signal on different sensors
2.2 Acoustic monitoring system on a bridge in Kortrijk (Belgium)

The bridge over the canal Bosuit-Kortrijk is a bridge on the E17 highway. It is a three span structure with 4 box girders and has a total length of 210m and is 40 m wide. The structure built in the 60's is a cast-in-situ cantilever bridge and has classical internal grouted prestressed tendons in the walls of the box girders. Due to visual and detailed inspections question marks were placed on the condition of the tendons in the outer box girders that are heavily loaded with cargo traffic.

To obtain information on the condition of the tendons in these areas, the client has decided to instrument the bridge with the acoustic monitoring system. The system will safeguard the structure and identify and locate the damage on the tendons if it occurs.

144 acoustic sensors were placed on both sides of one box and on only one side of the other box. The sensors are connected to a 32 channel acquisition unit. The sensors were placed inside the box girders. Due to the easy access the installation including tests and set-up covered only 2 weeks.

The system will monitor the bridge for 1 year with an option for longer.

References

Advanced Acoustic Emission Monitoring of Concrete Structures

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1. Introduction

Acoustic Emission has been known about for over a 100 years and its use in monitoring and testing has steadily increased for a number of reasons. Advances in electronic equipment, starting with the development of the first oscilloscopes, aided the pioneers, followed by the almost exponential growth in computing power in recent decades. Today systems have the processing speed to allow measurement of more than 1500 waveforms per second per sensor. These advances coupled with the associated cost reductions have allowed a vast increase in applications. Furthermore, the enhanced power and speed of computers allows a multitude of features to be extracted from the emission allowing a detailed signature of different types of damage to be identified and stored. Examples of such signatures are fatigue crack growth in steels, damage in fibre reinforced composites, tensile fracture of concrete, micro fracture of corrosion product (rust) and wire fracture in ropes.

Figure 1 Risk Based Inspection (RBI) cycle of maintenance using AE.

Acoustic Emission will always be an experience-based technology. In order to understand the AE signals you receive from the material in real structures it is important to be able to correlate the data to experiences where the damage process has been studied in greater depth. All successful applications like aircraft fatigue monitoring, corrosion monitoring of petrochemical tank floors and crack
Detection in metals have been due to a thorough understanding of the relationships between AE and damage growth. It is this knowledge that allows AE to be used confidently as a non-invasive monitoring tool identifying different types of damage using sensors mounted on the external surface to assess the structure. Today AE is widely used in the oil and chemical industry. The results of monitoring, even for short periods of time, can quickly identify defects and prioritise structures for maintenance, even giving indications of when further action is required. This form of maintenance is termed Risk Based Inspection (RBI) and its periodic use is shown in Figure 1.

RBI utilises AE to determine if a structure is undergoing actual active damage and, if so, to locate its position and severity. Using additional information such as strain or load the conditions under which damage occurs can be identified. RBI acknowledges the fact that AE should be used as a tool to identify structures or areas of concern which then can allow cost effective focusing of further non-destructive testing techniques and analysis (such as fracture mechanics) to determine the severity of the defect. It is at this point that the decision is made whether to repair the defect or to re-monitor at a later date or to monitor continuously. This strategy of short term monitoring structures both allows early detection of defects and aids the development of a cost effective priority based maintenance strategy depending on the actual damage and its significance in the structure.

With the increasing applications and awareness of AE in the world-wide civil engineering industry this paper goes on to profile the use of AE in different construction materials, structural elements and strategies, in three case studies.

2. Local Area Monitoring of Steel Corbels for Fatigue Cracking

The Queenhill Viaduct (M50, UK) consist of two independent concrete box girder viaducts carrying traffic to and from South Wales from Herefordshire. Each carriageway consists of approximately 30 metre spans joined by a half joint, Figure 2. Within these half joints are two steel corbels on which the bearings sit. The corbels underwent numerical fatigue analysis and were found to be nearing the stage when fatigue cracks might initiate. In particular the fillet welds on the plate identified by bold black lines, Figure 3, were particularly prone.

The corbels are cast into the half joint leaving only very small amount of their structure exposed, in addition to this the top corbel sits tightly on the bearing, leaving a gap of 40mm between the two concrete spans. The combination of these two factors greatly limited the inspection of the corbel, with the majority impossible to assess for fatigue cracking even with non-destructive testing techniques such as radiography or ultrasonics. Acoustic Emission was proposed as the technique that could identify and locate any active fatigue cracking.
Figure 2: Corbel site with scaffold access into the joint. Mobile laboratory can be seen at the base of the right hand column.

Figure 3: Corbel design. Dashed line denotes area accessible, bold line denotes area with suspected fatigue cracking.
2.1 AE Monitoring of the corbels

Piezoelectric sensors were mounted in a local array on the two upper and lower corbels of the west bound (pairs “A” and “B”) and east bound (pairs “C” and “D”), to provide planar location of active cracking in the hidden welds shown in bold lines in Figure 3. The two triangular arrays were linked so that any emission from bearing movement was distinguishable by both its location and from the signal features, which are very different to emissions from crack growth. Location checks using a Hsu-Neilsen source (simulating an AE source using the fracture of a pencil lead) determined that location accuracy was very precise, both on the bearing and the vertical plate. High frequency sensors were used to limit environmental noise (which is present at lower frequencies) from passing vehicles from corrupting the data acquired.

From previous experience of monitoring motorway bridges for cracks (Watson et Al, 2000) it had been established that one day’s traffic provided sufficient fatigue cycles to conclusively identify active defects, so monitoring was carried over a 17 hour week day period, from 05:00 to 22:00 hours.

2.2 Results of AE corbel monitoring

AE data was collected from all four of pairs corbels, with some of the results presented in Figure 4. A small number of emissions were detected from the corbels, however they were judged to be insignificant. Focused AE sources were detected in the region of the bearing block. The significance of sources was determined from an extensive database accumulated during 8 years of AE monitoring fatigue tests at Cardiff University (Holford et Al, 1996).

The activity of the source over the course of the day, figure 5, provides information on the bearing behaviour.

AE monitoring of the corbels was successful, allowing assessment of the concrete encased corbel with only very limited access to its surface. Hsu-Neilson source location was found to be accurate both on the corbel surface and the bearing. Monitoring showed no indication of fatigue crack activity during the test, the only located activity originated from the bearing block.
3. **Condition assessment of concrete half joints**

3.1 **Introduction**

Within the UK trunk road and motorway network there are approximately 10,000 bridges, of these over eighty percent are predominantly concrete. The majority of these bridges were constructed during the late 1960’s and 1970’s during the rapid expansion of the UK’s road network, and incorporated many new design features, which increased the speed of construction. During this period reinforced concrete was considered to be a material that would require little or no maintenance over their design life.

In hindsight however this is far from true. Today the UK’s engineers have inherited deteriorating concrete structures (some rapidly deteriorating), which have suffered from prolonged chloride attack, exaggerated by poor construction such as insufficient cover and poor water proofing. This has reduced the capacity of concrete bridges to resist increasing bridge loading, and in some cases has lead to bridges failing structural assessments. One particular concrete design detail that has been found to be concern is the half joint.

Figure 5: Located AE source activity over the day of monitoring from the bearings
The design consists of a supporting nib, which can either be continuous e.g. supporting slabs or multiple smaller ones e.g. supporting beams. The half joint allows the use of precast slabs or beams, which are craned into position resting on bearings on the half joint nib. The combined load from the beam and live load is transferred into the nib which carries much of the shear and tensile loading by means of high density steel reinforcement, which may include post tensioning.

Defective expansion and articulation joints can allow the ingress of water carrying chlorides from road salts, which percolate and collect in the nib. Over time the build up of chlorides can lead to deterioration and reduction of the steel section, sometimes in discrete locations, and cause a subsequent loss of shear and tension resistance in structures. One of the first indications of this occurring can be the spalling of cover concrete and structural cracking within the nib. It is of great importance that bridge engineers are able to identify the presence of this type of damage as early as possible to allow suitable remedial work and management of the structure. It is also desirable to accurately assess the condition of individual half joints to identify areas of damage and to rank these in terms of severity, allowing the prioritisation of actions and maximise cost efficiency.

Following on from the development of successful condition assessment and damage location techniques for steel bridges, Physical Acoustics Limited were approached by consultants W.S.Atkins and the UK Highways Agency, to carry out an trial to evaluate the potential of Acoustic Emission as an assessment method for half joints.

3.2 The Borrowbeck bridge

A bridge in Cumbria was suggested by UKHA Area 19’s Maintaining Agent’s, Mouchel and Amey Joint Venture (soon to be known as Amey Mouchel), as an excellent subject for a limited trial. Borrowbeck bridge is an under bridge just south of Junction 38 on the M6 motorway in the North West of England, which had been extensively scaffolded to allow a detailed inspection of its northern/southern hammerheads and half joints. The bridge spans a gully from a mountain brook and an access road to a farm and is shown in Figures 1 and 2.

The bridge consists of two spans to the abutments from the north and south hammerheads with a central drop in span, shown in Figure 3. All span supports are half joint nibs which each support two precast beams, either concrete box beams or pre-tensioned I beams. The northern and southern hammerheads are post-tensioned concrete box girders that have the cable anchorage’s located in the nib of the half joint, shown in Figure 4. This contributes significant compression strength to the half joint to counter shear and tensile forces.
As part of the assessment of the structure extensive chloride samples were taken from the soffit and half joints. Analysis of the samples indicated chloride levels up to 2.5% of the concrete cement content, with levels of 0.32% to 2.2% at 80mm depth. Many of these areas coincided with visual evidence of damage such as delamination, spalling and surface cracks.

From this information Mouchel Amey identified eight half joints nibs for assessment by AE monitoring, considered to be in a variety of conditions ranging from good, to those having evidence of damage. Details of the condition of these joints are shown in Table 1.
3.3 AE condition monitoring of half joints

The procedure developed by Physical Acoustics Limited for assessment of half joints follows on from the extensive work of Physical Acoustics Nippon and Kumamoto University, Japan, incorporating new research into concrete fracture signal behaviour and damage energy source analysis carried out at Cardiff University. The procedure utilises two monitoring strategies: Zonal and 3-dimensional Local monitoring. Both strategies were complimented by structural displacement and ambient temperature monitoring, which allows correlation of AE activity from damage sources with load conditions and environmental changes.

The first strategy used is Zonal assessment. This comprises of a single sensor mounted on the half joint nib soffit that detects signals originating from active damage sources within the joint. It has been determined that this sensor is capable of detecting a 0.5mm pencil lead break on the surface of the structure at distances greater than 6m. This gives the Physical Acoustics AE system sufficient sensitivity to detect signals from microfracture of concrete, with energies in the region of $10^{-12}$ Joules, ~10,000,000 times smaller than post-tension wire fracture. The individual Zonal monitoring sensor is able to assess a large volume of structure during normal traffic loading. Analysis of this data after the removal of any extraneous noise determines the significance and intensity of damage mechanisms within the joint and the conditions under which they are caused. Comparison of the data from individual sensors allows condition ranking of areas of slab joint or the nibs of beam half joints. This facilitates the prioritisation of structures and areas for further AE monitoring, if required.

The second strategy employed in half joint assessment is 3-dimensional monitoring. This strategy seeks to acquire detailed information about unique sources within the joint, which includes source intensity and severity. This was achieved through correlation with controlled crack growth experiments and attenuation modelling at Cardiff University. The damage and growth conditions together with approximate location are displayed graphically in either 2 or dimensions.

<table>
<thead>
<tr>
<th>Half Joint Location</th>
<th>Suspected Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>Good</td>
</tr>
<tr>
<td>B6</td>
<td>Good</td>
</tr>
<tr>
<td>B7</td>
<td>Good</td>
</tr>
<tr>
<td>E1</td>
<td>Cracked, visibly corroded</td>
</tr>
<tr>
<td>E6</td>
<td>Cracked</td>
</tr>
<tr>
<td>E7</td>
<td>Cracked</td>
</tr>
<tr>
<td>F6</td>
<td>Cracked</td>
</tr>
<tr>
<td>F7</td>
<td>Cracked</td>
</tr>
</tbody>
</table>

Table 1: Suspected half joint condition
3.4 AE assessment of Borrowbeck bridge half joints

Testing was carried out according to Physical Acoustics Half Joint Monitoring procedure, according to PAL ISO 9002 Quality System. A Physical Acoustics “DiSP” Acoustic Emission testing system was used with special sensors optimised for detecting microfracture in concrete. The displacement of the half joint soffit was monitored throughout the test and logged with AE data by the DiSP system together with temperature. This allowed correlation of environmental conditions with AE data. The positions of the probes are shown in Figure 3.

Each of the eight joints identified were Zonally monitored using one AE data sensor mounted on the soffit of the half joint. Soffit mounted sensors make the procedure and results from Borrowbeck transferable to other types of structures such as slab half joints. The sensitivity of the system was set to detect signals with 1/10,000 of the energy of an 0.5mm pencil lead break at 0.5m distance. The pencil break or “Hsu-Nielsen” source is the field calibration standard, as per BS EN 1330-9:2000. Based on Cardiff research, this level of sensitivity exceeds that required for locating the early stages of concrete microfracture. All monitoring was carried under normal traffic flow.

Figure 5: Position of the sensors on the half joint nib during local monitoring

Following Zonal monitoring of the eight half joints, AE data was analysed in order to identify the most damaged joint. Eight sensors were mounted on this half joint, 3 sensors on the soffit, 3 on the external side of the joint and 2 on the vertical end face of the joint to the right hand side of the outer beam, as shown in Figure 5. Sensors were mounted as previous, calibrated and location performance checked against artificial surface sources.
3.5 AE half joint test results

Study of signal attenuation found that the “Hsu-Nielsen” field calibration source could be detected at the other side of the half-joint nib with a sizeable amplitude, which was considered to be low attenuation for a concrete structure of this size, geometry and structural reinforcement.

AE signals from the beam and half joint bearing were filtered out, leaving only emissions from the half joint itself. A summary of some of the results after data processing and noise removal is presented in Figure 6. The half joint location supporting beam E1 was found to be the most active, accounting for 67.3% of all the signal energy from all monitored half joints. Its position is shown in Figure 3, under the outer beam on the North bound carriageway. This correlated with high chloride levels, up to 2.5%, identified from concrete samples taken from this half joint. Half joint E1 had suffered extensive visible spalling due to corrosion, and was considered to be one of, if not the most degraded half joint on the structure. Half joints at the south end of the southern hammer head which support beams B1, B6 and B7, were identified by the Maintaining Agent as being of sound condition. These were found to emit very low levels of damage present or occurring during the monitoring period.

![Figure 6: Percentage of cumulative energy released from each half joint](image)

The emission from the most damaged half joint nib, E1, was directly related to displacement (load). As a vehicle passes over the rear of the table the displacement becomes positive due to the cantilever effect of the table over the pillar, reversing when past the pillar support. The stress then increases rapidly on the half joint as the vehicle passes over it and on to the beam, and emission primarily occurs as stress increases at this point. Some emission occasionally occurs on unloading, this is normal behaviour known as crack closure noise. Signal analysis of data from half-joint E1 (and others) indicates that the signals are from micro-fracture of concrete and secondary crack closure emissions. This is supported by Cardiff University research, Figure 7 shows a typical waveform due to tensile microfracture of concrete.
Data from Half Joint E1 was analysed to determine the location of active damage sources present. The location of emission was calculated by triangulation, from the time arrival of signals, a similar method to earthquake epicentre location. The 3-dimensional location effectively "looks inside" concrete structures, in this case the majority of emission appeared to originate from two locations, shown in Figure 8. The first is a focused source, located approximately 1.6m from the end face within the half joint nib in an area of high shear stress. The second is located at the outer surface of the nib which was near an area of that had suffered extensive cover spalling due to corrosion of the
reinforcement shown in Figure 8 inset. Location accuracy depends upon the position of the source inside the array, and the signal wavelength, in this case it is expected to be within +/-0.1m. Through correlation of displacement of this half joint nib and the AE activity, it was possible to confirm that most of this activity occurred at maximum load, suggesting that they are indeed damage mechanisms. The characteristics of the located emissions were found to have distributed energy levels, averaging $10^{-13}$ joules, peaking at $10^{-11}$ joules. This indicates active micro-fracture concrete.

### 4. Conclusions

AE monitoring of the corbels was successful, allowing assessment of the concrete encased corbel with only limited access to its surface. Hsu-Neilson source location was found to be accurate both on the corbel surface and on the bearing. Monitoring showed no indication of fatigue crack activity during the test, the only located activity originated from the bearing block.

The Physical Acoustics Half Joint monitoring procedure, as used on Borrowbeck Bridge, has proven to be practical for use on motorway bridge half joints under normal traffic conditions, and has demonstrated its ability to identify and rank their condition. 3-dimensional monitoring has provided detailed source analysis, together with location information on the most badly deteriorated joint, achieved with normal traffic loading.

### 5. Acknowledgments

This presentation is based on two previous papers:
The author would like to acknowledge and thank the following: United Kingdom Highways Agency, UKHA Area 19 Maintaining Agents Mouchel-Amey Joint Venture Area 19 Maintaining Agents, especially Mr.H.Roper and Mr.D.Pollard for all their valuable assistance in this trial. Physical Acoustics would like also to thank Dr.A.W.Davies, Dr.K.Holford and Mr.T.Bradshaw at Cardiff University’s School of Engineering for their continuing assistance in development of structural monitoring techniques.

6. References


SAMCO - Structural Assessment, Monitoring and Control. Lessons learned from monitoring of post tensioned bridges

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Abstract

It has been recognised that networking adds value to the fragmented research activities in the field of Health Monitoring. Structural Assessment, Monitoring and Control has gained importance in our complex technical environment. The European Union appreciates the development and supports activities in these fields for the benefit of society. The SAMCO Network is established, on top of a number of high profile research and development projects, to concentrate the knowledge and to stimulate further development. A database is established to provide raw data to researchers free of charge and a help desk shall answer queries to the subject. The end users forum shall help to close the gap between the developers and users of new technologies in the field. It is intended to keep this network as open as possible to enable interested parties to join and contribute, but in particular benefit from the activities. The invitation to join is open to anybody world wide.

1. Acoustic monitoring system

People desire trivial solutions because they make our life easier. Problems are there to be solved is a common saying in the language of marketing. Advertising, which influences our life considerably provides permanently answers without asking questions. Bertrand Russell said: "A good question already includes the answer." The general public demands that everything works perfectly. Teachers, mechanics, and doctors provide answers and trivialise our world. This cannot yet be said from researchers, which are often misunderstood.

They often stand for another reality, which has nothing to do with our daily environment. Citizens ask questions, also in such areas where the general believe is, that everything is solved. Public relation work is required to close this gap. Furthermore researchers are encountered in their golden cage, and their solutions are very often not understood because they use a wording, which does not exist in every day's language. Ideally we guess why a research subjects might be important, but general we don't understand the background.

Knowing these facts it makes sense to invest in all means of dissemination of results of R&D work in an understandable way. The public shall be made aware of the important work done and
the consequences from new development. It is taken for granted that everything works perfectly, but the hard effort behind it is not recognized and awarded. For the field of structural assessment monitoring and control the SAMCO Network (www.samco.org) shall be a platform for these activities. Information shall be provided in a most understandable way and shall support the research community with the necessary raw material. It is our understanding that by giving away raw data for free, we will achieve a better environment to carry out our work. We therefore encourage everyone to use the material and to help in further dissemination.

2. Lessons learnt from the first 2 years of Operation

The SAMCO Network started to operate in October 2001. Since then 3 workshops and a Summer Academy has been organized on the subject. The benefits of the initiative are already visible. Among others the following deserve special attention:

- The integration of sectors, namely construction, energy, nuclear, environment, automotive, aerospace and production industry sectors, brings new ideas and approaches to the solutions. The initiative has been appreciated by the concerned engineers.
- The integration of all players, namely industry, small and medium enterprises, governments and institutions, the academics, research laboratories and society, increases the awareness of the players from the problem side to the scientific solution.
- The critical mass achieved by integration helps to raise sufficient funding for useful development on medium and large scale. It further overcomes the problem of limited markets for exploitation.
- The collection of data and the subsequent free distribution to interested research groups fertilised a cooperation and exchange of ideas, results and approaches. The idea of free exchange of all data has been picked up and will probably lead to different approaches in the research community.
- The dissemination of information is one of the weak points of community research. The new media databases and portals make it possible to supply the community with the updated information online.
- The cross fertilisation of works through events such as special workshops or summer academies showed to be of interest to the various industries and researches. Particular evolving technologies are benefiting from the contacts established.
- The identification of key players (mapping) helps to identify the relevant contacts in the market and the relevant authorities. Never the less it has been difficult to approach the very fragmented sector.
- Harmonisation, benchmarking and standardisation is only possible, if a critical mass of institutions and researches are behind the technology. The application in the market is widely depending on applicable standards, which can be drafted, supported or distributed.
- The knowledge of the current practice helps to support initiatives on practical application of newly developed methods. The database creates references that allow to overcome eventual
hurdles in the procurement process.
- The international contacts, which can be established to connect research initiatives and thematic networks help to coordinate the international progress of works. This section has been shown to be particular fruitful.
- The definition of visions for future research activities might lead to the development of entirely new research programs and strategies, which can be included into framework programs like those of the European Union.
- Programs like those of the European Union

It has been observed, that the expected benefits really materialized during the operation of the network. It is interesting enough to know that the use of these benefits, particular the database, is not at all limited to Europe. More than 60 % of the users are from countries outside the European Union. As survey among the users of the information has shown very satisfactory results. It will be interesting to see, what the response becomes, when the network has reached it full size.

3. Structure of the European SAMCO Network

Structural Assessment, Monitoring and Control have gained importance in our complex technical environment. The European Union appreciates the development and supports activities in these fields for the benefit of society. The SAMCO Network is established to concentrate the knowledge and to stimulate further development. A database is established to provide raw data to researchers free of charge and a help desk shall answer queries to the subject. The end users forum shall help to close the gap between the developers and users of new technologies in the field.

Technology Management in the field of construction is a precondition for the successful market introduction of new and innovative developments. The long way from the idea to the commercial application shows hurdles consisting of a lack of references, a lack of codes and recommendations, as well as the insufficient information and awareness of decision makers. Today's monitoring technology supports science and development and subsequently conquers practical applications. Never the less any new technology raises further questions and enables practical solutions for given problems. Monitoring has successfully conquered the laboratory work, then the production facilities and is now progressing into every aspect of human life. The management of the infrastructure is one of the young fields of monitoring, assessment and structural control. To focus the activities the subject of bridge management will be introduced, that allows application in one of the most developed fields.

3.1 Problem Statement

Construction is traditionally a conservative business. As it represents almost 10% of GNP in Europe any improvement has considerable positive effects on the European development, growth and employment. To acknowledge this fact the role of construction related research projects within the European framework programs has been considerably strengthened over the years. Many
outstanding results have been achieved. Anyhow the exploitation of the results has faced the following problems:
- Conservative End Users in the construction business are not sufficiently aware of research and development achievements made trough EU projects.
- Many sophisticated solutions are not applied due to a lack of reference projects or demonstration cases.
- The engineering community is too diversified and uses too many different approaches (lack of benchmark tests).
- Many promising technological approaches lack success due to the lack of raw data or opportunities to get this expensive information.
- Complex technical matters are often seen from diversified point of view. It takes a whole life - whole costs approach to model reality and assess the risks.
- Education is not picking up new technologies fast enough to improve the knowledge of End Users and engineers sufficiently for the acceptance of new ideas.
- Key question of structural dynamics like a consistent definition of damping are not answered by the ongoing research and development projects.
- Experimental methods are little used by the designers due to insufficient knowledge and lack of access to the facilities.
- There are no standards, codes or recommendations for the new technologies. This makes positive decisions for innovative projects by authorities unlikely. The political risk is contra productive.
- Recent earthquakes (Le. Turkey, Taiwan, Greece in 1999) showed that society and engineering is insufficiently prepared for those incidents.
- General dissemination and exploitation problems are experienced by many of the research consortia.

3.2 Objectives
To offer solutions to above mentioned problems the following objectives are defined for the intended Network:
- To create a centre of knowledge and reference at JRC in Ispra, Italy.
- To carry out benchmark tests and distribute the raw data freely from a database established at JRC.
- To work out a recommendation as a basis for a code for monitoring, assessment and control of structures.
- To define necessary steps for a better handling of earthquake loads and the related structural response.
- To provide information about the experimental testing capabilities and allow a big audience to see the tests, use the capacities and learn from it.
- To disseminate the idea of a whole system - whole life approach in structural engineering.
- To address the specific situation of the transportation sector, particular the railways, where huge investments are foreseen in Europe.
- To generate an environment where reference projects can be developed and exploited.
- To organise summer academies for improvement of the education situation.
- To define the needs for further research and development as well as the enhancement of optimal consortia and solutions targeted on the next framework program.
- To convince conservative owners by demonstration projects and material.
- To create a certification agency to help to overcome duplication and costly parallel development.
- To compare the European knowledge, standards, technologies and testing techniques with non-European countries.
- To improve the existing bridge management in Europe.
- To offer newest monitoring and assessment technologies to bridge management.
- To create a feedback from bridge management to the structural engineers.

3.3 Means

The means to achieve these objectives are:
- Establishment of a database at JRC.
- Establishment of an information dissemination centre at JRC.
- Definition and execution of benchmark tests for free distribution of raw data.
- Establishment of a Network of thematic areas where the state of the art can be discussed and the future need can be formulated.
- Establishment of a certification unit that allows to standardise methods and approaches.
- Creation of a panel that works out recommendations for codes and standards.
- Creation of an End Users Forum that will receive the relevant information free of charge and that is able to participate in the ongoing processes.
- Organisation of summer academies that provide information on the highest standard available in Europe for training and education of owners, End Users, engineers and academics.
- Set up and strengthen international collaborations, mainly through JRC Ispra, with relevant research centres and universities to compare the advance of competencies in various countries and in particular to compare design guidelines and standards in order to promote their realisation.

3.4 Organisation

The organisation knows a co-ordinator (1), a steering committee, thematic groups (5) organised with a thematic co-ordinator each, principal contractors (20) and members (130 and growing), an End Users forum, the data base, the standardisation activities and the certification. It is described in a chart on [www.samco.org](http://www.samco.org). Initially there are 20 principal contractors which have been selected on basis of their qualification. Each of them will take care of a number of participants which will be selected from the existing roaster or invited based on their qualification. This should enable the Network to concentrate the best forces on that subject. The progress of each of the groups shall be
monitored and approved by the steering committee. To enable a common exchange of a new ideas an annual workshop and a final conference shall be organised.

3.5 International Collaboration

The Network aims also to set up relevant international collaborations with Japan, the United States and other leading organisations in countries that have a program of assistance with the European Union. Contacts to the huge number of organisations involved in monitoring assessment and control shall be made and the related conferences, workshops and symposia shall be co-ordinated. The collected information shall be displayed in the data base at JRe. A feedback on the European approach is expected improving it’s position.

3.6 Innovation

New technologies require many years to become accepted by the general public. The innovative character of the proposed Network is that the basis for such an acceptance shall be provided. At present the parties involved in technology development (such as IASC, on the academic level) organise their meetings and conferences completely independent from those who should apply the technologies (such as IAB SE, on the end users level). The overwhelming response to the offer of participation to such a Network from both scientific and the practical side shows that there is a large gap to be bridged. To make sure that it is the End Users requirement that counts, the Network is organised by a private End User and foresees in a key position the End Users forum, which shall generate the requirements on the technology.

Another main innovation is the establishment of a data base, that contains real raw data, which can be used by everybody for research and development activities. The scientific community is demanding such data since many years without success. In the response of participants particular the possibility to receive real data has been appreciated. The 3rd major innovation is that it will be tried to establish a certification of methods, which enables the participants to calibrate their systems and gather a certificate that enables them to sell their products under a recognised standard. The 4th major innovation planned is the drafting of codes and standards to enable a proper tender procedure of the relevant works. The lack of standards provided a major obstacle in the distribution of the methods.

The 5th major innovation is the combination of bridge management, a risk and cost related activity, with the monitoring and assessment capabilities. The procedure shall be consistent from the basic technology to societal needs.

3.7 Partnership and Participation

The participation list shows the huge interest from various groups to the subject. From the biggest European construction company (Bouygues) to a major re-insurance company (SwissRe), from Bridge-owners (Autostrade) to Bridge-managers (TRL), from the biggest railway consultant
world wide (SSTRA) to state run road authorities (Finish roads and Greek roads), from leading European equipment suppliers (LMS) to the technology leader in bridge monitoring (VCE), from famous European Research Institutions (BAM and EL\IPA) to the European Joint Research Centre (JRC) from housing authorities (Hong Kong) to bridge owners (Denmark), from city governments (City of Vienna) to regional governments (Norway) from University to Research institutes world wide. There are participants representing 33 countries out of 5 continents. Some of the objectives and means in this proposal have been directly received from this huge group of End Users.

The large testing facilities in Europe provide unique features which are rarely used by practising engineers. Some of the features of the Network are targeted to make better use of this facilities and to stimulate the use of the results. The role of JRC shall be improved considerably and the lack of knowledge on testing possibilities shall be removed.

3.8 Relevant other Subjects

Within the Network the following ideas shall be realised:

- Organisation of a summer school on the subject
- Elaboration of scripts of the main topics for reading lessons at universities
- Provision of supporting material for promotion of the technologies
- Demonstration tests at the main large facilities to enable attendance in key projects
- An idea competition on testing with awards
- A co-ordination with other Networks approved (eventual clustering)
- To enable references to a larger group of European SME's
- Enhancement of the use of large testing facilities

To ensure a smooth flow of data, know how transfer and dissemination a hotline will be offered. A communication manager shall be appointed, who sorts the incoming questions and forwards them to the relevant appointed experts. Through this procedure a trouble shooting can be achieved.

Similar activities have been realised in the United States, where the NEES project (network on earthquake engineering simulations) has been launched. A separate paper is devoted to this research project which has a magnitude of 81 Million US$. It is intended that the European initiatives and the US initiatives will start to establish links to make the best use of such major R&D investments.

4. Examples of usefull Tools

The trend towards knowledge based engineering requires the establishment of databases, where the knowledge can be stored, sorted and withdrawn. A typical example shall be provided in the following.
4.1 The SAMCO data base

There is open source data base management system developed by the Joint Research Centre within the frame of the EWSE (European Wide Service Exchange) and GELOS G7 prototype projects and began as a collaborative effort between the Institute for Space Applications and the Institute for Systems, Informatics and Safety (ISIS), both of which are based at the Joint Research Centre's Ispra, in Italy.

4.2 Monitoring Frankfurts Skyscrapers

One of the major topics at the natural disaster side is the increased number of storms experienced in Europe. The fall of a glass panel from the 29th storey of a skyscraper in Frankfurt has triggered interest in these phenomena. 3 of the major towers, including Europe's highest building the Commerzbank (302 m), has been instrumented to record the consequences of wind action on the city.

This includes shading of the buildings and group effects. The equipment is in operation since January 2002 and delivers reliable data to the database. A correlation between wind measurement and acceleration within the building is provided. The experiment will last for at least 2 year and the results are consequently updated in the database. The relevant link is http://www.samco.org/

![Figure 1: Spectrum Commerzbank](image)

4.3 Cable Assessment

Cable stayed bridges have become a fashion since the 1960th. Nowadays we deal with a number of structures of age 30 to 40 years. This is the point where the first problems are experienced. To show that not only the old bridges are concerned an example is provided of a new bridge, where subsequent monitoring was able to show the development of cable forces over a couple of years with results that made an intervention necessary.
Table 1. Deviation of cable frequency

<table>
<thead>
<tr>
<th>Cable number</th>
<th>Cable type</th>
<th>1. mode assessment 2001</th>
<th>Dead load g</th>
<th>Diameter of cable</th>
<th>1. mode assessment 1998-98-01</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>55-150</td>
<td>0.57</td>
<td>104.24</td>
<td>0.22</td>
<td>0.50</td>
</tr>
<tr>
<td>2</td>
<td>37-150</td>
<td>0.61</td>
<td>79.53</td>
<td>0.22</td>
<td>0.60</td>
</tr>
<tr>
<td>3</td>
<td>55-150</td>
<td>0.55</td>
<td>104.24</td>
<td>0.22</td>
<td>0.68</td>
</tr>
<tr>
<td>4</td>
<td>37-150</td>
<td>0.77</td>
<td>79.53</td>
<td>0.22</td>
<td>0.75</td>
</tr>
<tr>
<td>5</td>
<td>37-150</td>
<td>0.95</td>
<td>79.53</td>
<td>0.22</td>
<td>0.85</td>
</tr>
<tr>
<td>6</td>
<td>37-150</td>
<td>0.91</td>
<td>79.53</td>
<td>0.22</td>
<td>0.96</td>
</tr>
<tr>
<td>7</td>
<td>37-150</td>
<td>0.79</td>
<td>79.53</td>
<td>0.22</td>
<td>0.74</td>
</tr>
<tr>
<td>8</td>
<td>55-150</td>
<td>0.70</td>
<td>104.24</td>
<td>0.22</td>
<td>0.76</td>
</tr>
<tr>
<td>9</td>
<td>37-150</td>
<td>0.57</td>
<td>79.53</td>
<td>0.22</td>
<td>0.60</td>
</tr>
<tr>
<td>10</td>
<td>37-150</td>
<td>0.54</td>
<td>79.53</td>
<td>0.22</td>
<td>0.55</td>
</tr>
</tbody>
</table>

4.4 Damage Identification

Construction joints of concrete bridges built in the 1960’ies represent notorious week points. The Großram - Bridge is a typical continuous structure built by the advancing shoring method. During earlier inspections a defect construction joint was identified. The joint opened under heavy loads exposing the prestressing tendons to moisture and salt attack. The joint was repaired and strengthened by clued fibre plates. The task for the monitoring team was to assess the quality of the strengthening work and the function of this important structure. It was demonstrated that the
damage was repaired successfully and the capacity of the structure was restored. The assessment was carried out using Modal Assurance Criteria (MAC) which provide figures for modal fitting between calculations and test data. Figure 3 shows the superimposed modes in theory and practice.

![Figure 3: Comparison between calculated and measured data](image)

4.5 Environmental Influences

Dynamic monitoring is particular depending on environmental influences. In order to make the assessment of the environmental influence easy, it would be nice to have long time data on for example temperature of a structure. For this purpose several monitoring systems have been installed that provide this information. If it is related to ambient temperatures rules valid for a
certain location can be developed, that allow an easy approximate assessment. If all this measurements could be linked in a European network the dissemination of the methods could be improved and the work of innovative engineers could be supported.

5. References


6. Links

www.samco.org SAMCO - Thematic Network: Structural Assessment Monitoring and Control
www.nees.org NEES - Network for Engineering Earthquake Simulation
www.vce.at VCE - Vienna Consulting Engineers
Sonic & Ultrasonic NDT Methods in the Assessment of Concrete Structures

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Abstract
This paper reviews certain sonic and ultrasonic non-destructive testing (NDT) methods as applied to concrete structures. The basic principles of the NDT methods are described with particular reference to the five major factors that influence the success of a survey: depth of penetration, vertical and lateral resolution, contrast in physical properties, signal to noise ratio and existing information about the structure. The main NDT methods used are discussed and illustrated with brief case histories. The integration of NDT surveys into the investigation of structures is described. The underlying reasons why NDT methods are perceived as ‘not working’ by the structural engineer are identified as: a lack of understanding of variability of concrete materials used and NDT methods themselves.

KEYWORDS: NDT, concrete, sonics, ultrasonics, bridges, buildings.

1. Introduction
Non-destructive testing methods clearly have a role in the evaluation and testing of civil engineering structures but all too often they are used to provide discrete information on specific problems rather than as an integral part of the overall survey programme. Hence, in this paper the fundamental principles of non-destructive testing methods are considered in some detail with a view to establishing a definitive role for them in the structural evaluation programme. Part of this role must be the setting of agreed standards and guidelines both for the execution of each surveying method in the field and the interpretation of the physical data obtained as an integral part of the investigation programme. The involvement of the NDT specialist as an essential part of the investigation team has been long delayed on the majority of major civil engineering projects worldwide.

There is a wide range of NDT methods, which are used in the civil engineering industry, and an example of some of these techniques appropriate to bridges is summarised in Table 1 [Forde & McCann, 1997].
Table 1: Sonic & ultrasonic NDT Tests for Bridges

<table>
<thead>
<tr>
<th>Inspection Method</th>
<th>Parameter Measured</th>
<th>Advantage</th>
<th>Disadvantage</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Visual</td>
<td>Surface condition</td>
<td>Quick; modest skills required</td>
<td>Superficial</td>
<td>Low</td>
</tr>
<tr>
<td>Proof Load Test</td>
<td>Load Carrying Capacity</td>
<td>Definitive</td>
<td>Very Slow &amp; possibly dangerous</td>
<td>Very High</td>
</tr>
<tr>
<td>Coring</td>
<td>Specific internal dimensions</td>
<td>Definitive dimensions</td>
<td>Measurement only at test point; scars the bridge</td>
<td>Moderately High</td>
</tr>
<tr>
<td>Vibration Testing</td>
<td>Mode shapes and/or signature</td>
<td>Gives some indirect measure of current condition</td>
<td>Difficult to quantify data; heavily damped masonry bridges give yield little response</td>
<td>High</td>
</tr>
<tr>
<td>Impact Testing</td>
<td>Mode shapes and/or signature</td>
<td>Gives some indirect measure of current condition</td>
<td>Difficult to quantify data; heavily damped masonry bridges give yield little response</td>
<td>Moderate</td>
</tr>
<tr>
<td>Ultrasonic NDT</td>
<td>Wave velocities through structure</td>
<td>Relatively quick</td>
<td>Only works on individual masonry blocks due to signal attenuation; no information on major elements</td>
<td>Moderate</td>
</tr>
<tr>
<td>Sonics</td>
<td>Wave velocity; tomographic cross-sections</td>
<td>moderately slow; Gives useful information on major elements</td>
<td>requires skill to interpret data</td>
<td>Moderately high</td>
</tr>
</tbody>
</table>

The application of non-destructive testing techniques to the solution of civil engineering problems has sometimes been disappointing. This has arisen from either using a method, which lacked the precision required in a particular structural investigation or by specifying a method that is inappropriate to the problem under consideration. In some cases these problems could have been avoided by taking expert advice before initiating the survey. It is also emphasised that in other cases the physical condition of the structure was far more complex than anticipated at the planning stage of the NDT survey and hence interpretation of the data did not yield the information expected by the engineer.

It is often advisable to undertake a feasibility study on the structure to assess the suitability of the proposed NDT techniques for the investigation of the structural problem. Such a procedure might be referred to as a “desk study” [Das, Hardy, McCann & Forde, 2000].
However the key issue is to undertake a systematic and holistic investigation – an example for a bridge is given below:

- **Phase 1**: Visual inspection
- **Phase 2**: Analysis of load carrying capacity
- **Phase 3**: Review need for further investigation - if none, then revert to routine visual inspection schedule. If further investigation required, then proceed to Phase 4.
- **Phase 4**: “Desk study” - before undertaking any more detailed field study, a research needs to be undertaken of the origins of the bridge, who designed and built it and the possible style of construction such as soil backfill or cellular construction. See Colla, 1997 [Colla, Das, McCann & Forde, 1997] for further data on historical records of bridges.
- **Phase 5**: Cost effectively choose the most suitable strategy for further investigation - see Table 1. An NDT method would only be chosen when a direct physical measurement strategy was inadequate or too expensive.
- **Phase 6**: Implement the investigation technique.

In this paper some of the major NDT methods used within the civil engineering industry are summarised together with their advantages and limitations. It is essential to distinguish between methods that are considered to be state of the practice as opposed to those that are under development and are described as state of the art. The latter methods are likely to be specific to one organisation where the original research and development was carried out and should be used with caution until sufficient experience of their application in the civil engineering industry has been achieved.

### 2. Basic Principles OF Non-Destructive Testing Methods

There are many non-destructive testing techniques, each based on different theoretical principles, and producing as a result different sets of information regarding the physical properties of the structure. These properties, such as compressional and shear wave velocities, have to be interpreted in terms of the fabric of the structure and its engineering properties. Inevitably, this interpretation involves some degree of assumption about the structure, and the use of calibration measurements is an essential feature of most non-destructive surveys. Furthermore, many structural problems will be best studied by a particular non-destructive testing method, depending upon which physical properties of the concrete offer the best chance of being reliably determined.

There are five major factors, which need to be considered in the design of a non-destructive testing survey, as follows:

1. the required depth of penetration into the structure.
2. the vertical and lateral resolution required for the anticipated targets.
3. the contrast in physical properties between the target and its surroundings.
4. signal to noise ratio for the physical property measured at the structure under investigation.
5. historical information concerning the methods used in the construction of the structure.
Careful application of all the above factors to the design of a non-destructive testing survey should result in a specification which either achieves the desired objectives or, more importantly, recommends an alternative approach if no NDT surveying method is deemed appropriate to the solution of the problem specified. Some examples of the importance of these factors are presented below; the principles of the different methods that can be used are described later in the text.

The most common problem that an NDT specialist faces in dealing with his client during the investigation of a structure is the integration of the fundamental information derived from the construction records with the results from the NDT survey. The construction record plus any additional engineering assessment represents the most accurate information that can be obtained on the structure in the area of the investigation. The actual resolution that can be achieved with all NDT methods will be inferior to the precise measurements obtained from the original plans of the structure. For example, vertical resolution is defined as the smallest vertical dimension $Z_{\min}$ that can be detected, and this is normally expressed as:

$$Z_{\min} = \frac{\lambda}{4}$$

where $\lambda$ is the dominant wavelength of the NDT data being analysed.

Although outwith the scope of this project, it is instructive to consider the case of an impulse radar survey, the resolution achieved is a function of the frequency of the incident electromagnetic energy and its velocity of propagation. The differences that the NDT interpreter is faced with can be illustrated by the following calculations incorporated into Table 2 below:

Table 2: GPR Propagation through concrete and masonry ($\varepsilon_r = $ dielectric constant (real))

<table>
<thead>
<tr>
<th>Material</th>
<th>$\varepsilon_r$</th>
<th>Frequency MHz</th>
<th>Velocity cm/ns</th>
<th>Wavelength cm</th>
<th>Resolution cm</th>
<th>$Z_{\min}$ cm</th>
<th>Penetration cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stone Parapet</td>
<td>5.69</td>
<td>900</td>
<td>12.55</td>
<td>13.9</td>
<td>7</td>
<td>4.6</td>
<td>low</td>
</tr>
<tr>
<td>Concrete</td>
<td>9</td>
<td>900</td>
<td>10</td>
<td>11.1</td>
<td>5.6</td>
<td>3.7</td>
<td>Low</td>
</tr>
<tr>
<td>Stone Parapet</td>
<td>5.69</td>
<td>500</td>
<td>12.55</td>
<td>25.1</td>
<td>12.6</td>
<td>8.4</td>
<td>Medium</td>
</tr>
<tr>
<td>Concrete</td>
<td>9</td>
<td>500</td>
<td>10</td>
<td>20</td>
<td>10</td>
<td>6.7</td>
<td>Medium</td>
</tr>
<tr>
<td>Stone Parapet</td>
<td>5.69</td>
<td>100</td>
<td>12.55</td>
<td>125.5</td>
<td>62.8</td>
<td>41.8</td>
<td>High</td>
</tr>
<tr>
<td>Concrete</td>
<td>9</td>
<td>100</td>
<td>10</td>
<td>100</td>
<td>50</td>
<td>33.3</td>
<td>High</td>
</tr>
</tbody>
</table>

Clearly from the above calculations it is important to select the optimum frequency to achieve the maximum penetration into a structure coupled with the required resolution of the likely targets. Practical use of impulse radar indicates that the shallowest target that can identified below the surface of a structure is $\lambda/3$ [Forde, 1999] and within a structure the minimum size of target is a value of $\lambda/2$ [Martin, Hardy, Usmani & Forde, 1998].
It is also essential that there is a contrast in the physical properties of the materials within the structure - since there will be no resolution of any significant changes in the engineering properties unless these give rise to contrasts in physical properties, such as sonic velocity, dielectric properties and so on. Different physical properties can also be a problem; for instance, there is very little difference in sonic velocity between a material saturated with fresh water and one saturated with a saline solution. The same materials would be significantly different as far as electromagnetic properties are concerned since the material saturated with a saline solution will have a much higher attenuation coefficient than the one saturated with freshwater.

3. Non-Destructive Methods and their Application

3.1 Sonic/Ultrasonic Methods

Non-destructive sonic and ultrasonic testing methods are non-invasive and have been used for the past thirty years in the assessment of civil engineering structures and materials. The sonic method refers to the transmission and reflection of mechanical stress waves through a medium at sonic and ultrasonic frequencies.

The five most commonly used sonic methods are:

- Sonic transmission method
- Ultrasonic/Sonic/seismic tomography
- Sonic/seismic reflection method
- Ultrasonic reflection method
- Sonic resonance method
- Impulse response method

(a) Sonic transmission method

Direct transmission involves the passing of a compressional wave at frequencies between 500 Hz and 10 kHz through the thickness of the wall (or the structure) under investigation. Transmission of the wave is initiated on one side of the structure by the impact of the force hammer, and reception on the opposite side is performed by an accelerometer positioned directly opposite the force hammer. The resulting wave velocity is an average of the local velocity along the path and it is not possible to establish the position and the extent of any possible inhomogeneity. The velocity magnitudes may be plotted in a contour map format, with grid points as X and Y co-ordinates and the pulse velocity as the Z co-ordinate. This format allows a simple evaluation of the relative condition of the concrete in a structure or an evaluation of the internal fabric of a structure, such as a concrete bridge.
It has generally been recognised that the direct transmission arrangement is a simple technique to apply in the non-destructive testing of structures since it provides a defined path length through the structure. Furthermore, since the arrival time of the first wave is of primary concern, no attempt to distinguish complex wave frequencies and reflections is required for the analysis. This method has been successfully used to evaluate material uniformity, detect the presence of voids, estimate the depth of surface crack, and calculate an average compressive strength for the structure or the material. The detection of flaws is possible due to the fact that sonic waves cannot transmit across an air gap, which could be due to a crack, void or delamination at the interface between brick or stone and mortar. A propagating wave must find a path around the void, resulting in attenuation and an increase in the transit time of the signal.

(b) Ultrasonic/Sonic/seismic tomography

Sonic tomography represents an improvement in the sonic transmission test method because tests are performed not only in the direct mode but also along paths which are not perpendicular to the wall surfaces. The wall of the structure or the masonry section is thus crossed by a dense net of raypaths, each of which relates to a specific travel time between the sonic source and receiver through the structure. These values of travel time can be used to compute a three-dimensional reconstruction of the velocity distribution across the structure or selected cross-section so that local variations in velocity can be identified and correlated with zones of weakness or flaws in the internal fabric of the structure.

It is usual to assume a linear structural response in the application of the tomographic method. This is because the response is measured with transducers which are normally mounted well away from the location of the impact where non-linear behaviour may arise. Any variation from the expected travel time is therefore attributed to in-homogeneity in the structure or damage occurred. In order to obtain good statistical accuracy it is necessary to maximise the amount of experimental data included in any calculation used by ensuring that all areas of the proposed tomographic section
have adequate raypath coverage. A number of inversion algorithms are available commercially for tomographic reconstruction.

![Tomographic ray paths](image1)

**Figure 2** Tomographic ray paths

![Ray path coverage for different transducer arrangements](image2)

**Figure 3** Ray path coverage for different transducer arrangements

Data acquired usually exhibits a good deal of velocity scatter, resulting from variations in the strength and nature of the hammer hit generating the input signal, the interpretation of acquired waveforms by the operator and coupling of the receiving transducer to the masonry or concrete surface. Data scatter has the effect of increasing the residual of tomographic velocity reconstruction and may lead to identification of false anomalies. The accuracy of the velocity reconstruction can be improved by a better understanding of the input signals, by a carefully planned choice of position and number of the reading stations and by simple data smoothing prior to analysis.

An example of a post-tensioned concrete beam is given in Fig. 4 below:
Sonic and Ultrasonic NDT methods in the assessment of concrete structures

13. October 2004, ETH Zurich, Switzerland, ed. B. Elsener

Note: The voids are formed by an air gap or a polystyrene box-out

Figure 4 Stanger Science & Environment model with included defects shown

Figure 5 Stanger Science & Environment beam Tomographic Survey. Position 0.4m from front end

Figure 5 is the 2-D tomographic interpretation of the ultrasonic tests on this beam (Martin et al, 2001)
(c) **Sonic/ seismic reflection method**

In the sonic reflection method both the initiation and reception of the sonic wave are performed on the same face of the masonry as in the case of indirect transmission, but the stress wave recorded is the direct stress wave reflected from any internal flaw or the rear face of the structure investigated. The value of velocity calculated from the rear wall or face of a structure is a measure of the local velocities along the path.

The problems that reflection methods may be used for in the investigation of retaining walls/wing walls/spandrel walls are:

- Internal dimensions and shape.
- Type and properties of fill.
- Voiding.
- Cracks and voids within the internal fabric of the structure.

(d) **The Impact Echo System**

The most recent development of the ultrasonic reflection and frequency domain method is known as the Impact Echo Test Method, which was originally developed to measure concrete thickness and integrity from one surface. The method is performed on a point-by-point basis by using a small instrumented impulse hammer to hit the surface of a structure at a given location and recording the reflected energy with an accelerometer mounted adjacent to the impact location – Figure 6.

![Figure 6: Set-up for Impact-Echo Test](image-url)
Since reflected signals are more easily identified in the frequency domain the received energy recorded in the time domain is passed to a signal analyser for frequency domain analysis – using a fourier transform algorithm such as Fast Fourier Transform (FFT). A transfer or frequency response function (FRF) is then calculated for the impulse hammer/accelerometer system and reflections or echoes of the compressional wave energy are indicated by pronounced resonant frequency peaks in the transfer function or frequency spectrum record – Figure 7.

![Figure 7. Frequency spectrum obtained after impact with hammer on test wall.](image)

These peaks correspond to the thickness or flaw depth resonant frequencies and knowing the compressional wave velocity in concrete or any other construction material the depth to the corresponding flaw can be calculated. The depth of the reflector will correspond to the slab or wall thickness if the concrete used in construction is sound. The original concept of FRF testing of civil engineering structures dates back to the testing of concrete piles [Davis & Dunn, 1974], whilst the modern adaptation was undertaken at NIST and Cornell University [Carino et al, 1986; Cheng & Sansalone, 1993; Sansalone & Street, 1997].

Impact echo testing of bridges has largely been focused upon identifying voids in ducts in post-tensioned concrete bridges. Finite element analyses of a laboratory experiment at the University of Edinburgh showed that defects can be identified provided a sufficiently high frequency is used: $> \lambda/2$ with respect to the defect.

In practice it is often not that straightforward. Results from a laboratory scale investigation at the University of Edinburgh, indicated that the experimental results were not as unambiguous as the FE work. This ambiguity is due to a number of potential reasons:

- three-dimensional dispersion of the impact echo wave through the concrete due to the presence of aggregate and other inhomogeneities.
- possible reduction in frequency of the impact echo signal due to crumbling of the concrete surface resulting in longer contact time and thus lower frequency [Martin & Forde, 1995].
- possible lack of sensitivity of the ultrasonic transducer.
Seismic waves, which are also generated by an impact source, are commonly referred to in non-destructive testing applications and propagate at frequencies in the range 100Hz to 1kHz. However, the terms sonic and seismic are often interchanged in practice, since both refer to the propagation of compressional waves in a medium. Seismic reflection techniques may be employed from the road surface, arch barrel or spandrel walls of a masonry arch bridge, the front of a retaining wall, or a harbour dock wall. However, it is not a method currently recommended since the resolution achievable with the low frequency energy is poor and it is often difficult to distinguish reflections from surface waves and refracted arrivals.

(e) Ultrasonic reflection method

Ultrasonic waves, which are generated by a piezoelectric transducer at frequencies above 20kHz propagate with a wavelength around 50 to 100 mm in concrete. This form of testing is used successfully at ultrasonic frequencies for the detection of flaws in metal castings and is the first non-destructive technique that was developed for the testing of concrete. However, it is much less practical in concrete, which has much higher attenuation characteristics and hence lower frequency signals are required to obtain a reasonable penetration. It has been successfully used for identifying and locating specific flaws in concrete.

However, at present the method is not commonly used for these purposes due to a number of technical difficulties. In the case of ultrasonic signals the main factors to overcome are the need for good coupling of the transducer to the surface, which is often rough, and the scattering of the wave due to material heterogeneity. The need for effective coupling requires the use of a coupling agent, such as grease or petroleum jelly, to temporarily adhere the transmitter and receiver to the surface. This makes the process of moving the points of measurement quite slow and it is often difficult to achieve adequate coupling on some uneven surfaces. Scattering of the signal limits the propagation through the material and also leads to a complicated series of return signals. This makes it difficult to identify defects amongst the noise. In addition surface waves, which travel more slowly than the compression waves, may arrive at the receiver within the same time interval and confuse interpretation. Further developments of the ultrasonic technique, for example improvements in signal generation, detection and data processing are underway and may lead to a practical tool if the problems mentioned above are overcome.

(f) Sonic resonance method or “coin tap” test

A simple variation of the Impulse Echo Method described above has been carried out for many years in the UK to detect defects or cavities behind the linings of tunnels or areas of delamination in a concrete wall or soffit of a bridge. In this case the wall or lining is tapped with a lightweight hammer and the ringing or echo associated with a hidden cavity or defect produces a significant change in frequency as the impulse hammer is operated in the defective area. The method is rapid to use since the human ear is extremely sensitive to the change in the resonant frequency. An instrumented version of this test was developed to identify debonding of metal plates - glued to concrete – was used successfully on a bridge in Scotland [Forde & Mackie, 1996].
(g) Impulse Response Test

This test measures the mobility of a surface by calculating the dynamic stiffness of the surface impacted by an instrumented hammer (McCavitt, Yates & Forde, 1992; Davis, Lim & Petersen, 2003; Ottosen, Ristinmaa & Davis, 2004). See for example data from Ottosen et al above: Figure 8

\[\text{mobility, } \frac{m}{sN}\]

\[\times 10^{-6}\]

\[0\quad 100\quad 200\quad 300\quad 400\quad 500\quad 600\quad 700\quad 800\quad f [\text{Hz}]\]

Fig 8 Typical mobility plots for sound and honeycomb concrete

4. Discussion

Geotechnical and civil engineers are accustomed to referring to British Standards Institution and National Measurement Accreditation Service (NAMAS) standard testing procedures when requesting material testing and site investigations. These standard procedures assist them and the associated contractor in carrying out an investigation to a given set of instructions - it therefore appears to be reasonably straightforward for an engineer to specify, supervise or check the work carried out by the subcontractor.

However, engineers are surprised that when wishing to investigate structures using NDT methods they can find no appropriate British Standard, Code of Practice or NAMAS standard on which to base a tender specification and prepare a bill of quantities. The engineer can search in vain for a standard procedure for carrying out most NDT methods, the one exception being the recently published assessment of the use of impulse radar in the testing of concrete structures [Concrete Society, 1997].

With all this information available it is difficult to understand why there is still the frequently expressed opinion of engineers that ‘NDT methods do not work’ and the complementary retort from the geophysicist that ‘all the engineer wants is the cheapest job’.

It would appear that the problem is a joint one:
1. Engineers have difficulty in understanding the variability both in the construction materials and geometry of the structure and what the NDT specialist can and cannot quantify.

2. NDT specialists have been slow to relate their measurements and interpretations to geotechnical and engineering parameters. NDT specialists, in general, have little or no training in civil engineering design and practice.

In summary, there are two independent groups of investigators with the same goal but with no agreed route to that goal. In a discussion on the application of geophysical methods to the site investigation process Annan [1992] attempted to explain the difficulties: “Geophysics should be to the engineer what medical imaging is to the doctor. However, the geophysical problem is far more difficult than the medical problem. Normally the human parts are in the same locations.” The same comment is equally applicable to non-destructive testing of structures since in most cases there is very little positive information available on the internal composition of the structure under test.

It must be borne in mind when an NDT survey is being considered that there must be distinct differences in the physical properties of the subsurface target, and its surrounding material for the survey to provide the possibility of a success or as the engineer would say ‘make it work’. However usually the detailed information required to specify an NDT survey is not available until after the survey has been completed and reported. How does the engineer proceed?

The engineer must become familiar with NDT methods since an NDT investigation is not a simple routine procedure but it is a piece of research in which experience and instrumentation are deployed to carry out an investigation. The message must surely be if you are specifying or requesting a service one should be aware of the background to the techniques requested and their limitations.

The question of lowest contract price is a familiar discussion point and of concern to all, but it is a fact of life within and without the earth sciences profession that contract prices have decreased continually since the late 1980s. The NDT specialist will have to come to terms with the world of low and perhaps decreasing prices in the future and can only survive by being better trained, equipped and more efficient. Certainly the advent of NDT equipment incorporating computers and data loggers plus the savings in time and money of PC-based data display systems will assist.

While it is important to produce detailed NDT standards for structural investigations there appears to be ample technical literature either currently available or in the pipeline to ensure the availability of sufficient guides, guidelines and specifications for the correct specification of an NDT structural survey. What is missing at the moment is the close collaboration between the civil engineer and the NDT specialist that is essential for the successful outcome of the NDT survey at an economic cost.
5. The Future

The future of NDT lies in the areas of:
• Better understanding of material properties
• Better understanding of complex structures
• Improved tomographic imaging
• Forward modelling
• A more realistic approach to the accuracy and achievable results from NDT

Considerable progress has been made within the international community in order to identify the accuracy of various non-destructive testing techniques for the analysis of bridge reliability. However, the challenge from the professional community is to adopt these understandings when undertaking ultrasonic, impact echo and radar surveys of structures. It is now clear that the null hypothesis approach using impact echo testing that “if a defect is not seen then it does not exist” is too simplistic.

6. Conclusions

• A wide range of NDT techniques applied to bridges and buildings has been reviewed.
• The shallowest detectable defect using impact-echo is at a depth of \( \lambda/2 \).
• The best practical resolution using impact-echo is \( \lambda/2 \).
• The null hypothesis approach using impact echo testing that if a defect is not seen then it does not exist is too simplistic – thus impact echo testing may be oversold in certain circumstances.

7. Acknowledgments

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Electrical Isolation as Enhanced Protection for Posttensioning Tendons in Concrete Structures (PL 3)

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Abstract

Electrically isolated tendons with plastic ducts for internal grouted post-tensioning were developed about 15 years ago. This new generation of tendons offers enhanced corrosion protection of the steel strands and the possibility to monitor the corrosion protection by simple non-destructive measurements (electrical impedance). This is the reason that the fib draft "Durability specifics for prestressed concrete structures" proposes this technology for the highest protection level (PL3) for post-tensioning tendons. The measurement principle and the development of the monitoring technique are discussed.

Keywords: Electrically isolated tendons, monitoring, durability

1. Introduction

Prestressing created new possibilities for the structural use of concrete. This success was made possible by advanced technologies producing steels with high strength, but more susceptible brittle fractures caused by corrosion and hydrogen embrittlement.

Figure 1: Corrosion hazard of traditional tendons with metallic ducts

Figure 2: Protection against corrosion by the new electrically isolated tendons
A summary of possible corrosion hazards and mechanisms is given in the FIP recommendation “Corrosion protection of prestressed steel” published in 1996 [1]. This corrosion of prestressing steel may cause much more serious problems than that of normal reinforcement as prestressing tendons normally have a small cross-sectional area under very high stress.

Environmental loads such as chloride attack due to de-icing salts or seawater have increased in the last decades. Research and investigations on a great number of existing structures [2]~ some of them on structures after demolition at the end of service life~ have shown that corrosion of the prestressing steel occurs at points where water and chloride ions can penetrate. The metallic ducts usually used can not be considered to be water-tight and thus do not present a barrier against corrosion. All these facts together with an increased awareness of the aspect of durability has lead to demands for improved performance of the PT systems in corrosion protection and the possibility to monitor the tendons.

Figure 3: Corrosion hazard scenarios on a bridge deck [2]
2. History and Standards

2.1. Milestones to Electrically Isolated Tendons (EIT)

Various recommendations and technical reports give indications to special corrosion protection measures during shipping~ storage and other transient states of PT steel and PT components. Also reports and recommendations of faulty construction practice~ e.g. inadequate concrete cover~ poor concrete quality or inadequate grouting techniques show proposals for improvement. In the 1960s~ corrugated plastic PT tendon sheathing was introduced to have a more cost efficient solution compared with traditional steel ducts. The petrol crisis of the 1970s, was responsible that plastics as material for PT tendon sheathing disappeared again. When investigating fretting fatigue and fretting corrosion of tendons with metallic ducts [2], polymeric materials were introduced again for tendon ducts because of their improved fatigue behaviour. This was the starting point of a fully isolated tendon.

A second step, towards Electrically Isolated Tendons (EIT) was the already mentioned increasing concern about corrosion of tendon mostly due to penetration of chloride ions. The request for a tight encapsulation of the PT tendon was introduced.

The break through for Electrically Isolated Tendons was finally given by the requirement for the 945 permanent ground anchors for the railway station Stadelhofen, Zürich in 1985. Since the station is located in the middle of the city, close to one of the city's most powerful rectifiers of the DC powered trams it was necessary to provide the entire ground anchors with suitable protection against the potential risk of stray current corrosion. Details of the electrical isolation of PT tendons on the corresponding measurement are explained later on in this paper. However, it can be summarised that investigation to avoid fretting fatigue and fretting corrosion of tendons and the search for a water and stray currents tight envelope, brought out the EIT, Electrically Isolated Tendons for Post-Tensioning.

2.2 Reference to Standards

Several reports such as the already mentioned FIP recommendation, "Corrosion protection of prestressing steels" 1996 [1] and the EOTA "Guideline for European Technical Approval of Post-Tensioning Systems" [3] 2001, are giving guidance for Electrically Isolated Tendons. The most complete document regarding specification of EIT, measuring instructions and interpretation of results on structures with EIT, is the Guideline for "Measures to ensure the durability of post-tensioning tendons in RC concrete structures [4]", published jointly by the Swiss Federal Highway Agency and the Swiss Railways in 2001. The principle has been adapted in the recent fib-draft "Durability specifics for prestressed concrete structures".

The guideline [4] and the fib draft propose three categories for Post-Tensioning tendons in terms of corrosion protection. Tendons of protection level I (PL1) are in a traditional corrugated steel duct. Tendons of protection level 2 (PL2) specifies corrugated plastic ducts for enhanced...
Electrical isolation as enhanced corrosion protection of post-tensioning tendons (PL3)

Protection. Tendons of protection level 3 (PL3) shall be electrically isolated. The main difference from tendons of PL3 to PL2 is the complete electrical isolation at the anchor heads to avoid stray currents and to allow to monitor the integrity of the tendon encapsulation by measuring the electrical resistance between the tendon and structure.

![Diagram of electrically isolated tendons (PL3)](image)

Figure 4: Schematic representation of the electrically isolated tendons (PL3) [4]

The guideline [4] and the jib-draft indicate criteria for the choice of the categories, detailing mainly tendon category band c and gives specific information for the measuring equipment, the performance of the measurements and the interpretation of the results as described below.

![Diagram of structural protection layers vs. aggressivity of exposure](image)

Figure 5: Structural protection layers versus aggressivity of the exposure defines the necessary protection layer
3. Measuring the impedance of the duct

The measurements of the electrical isolation between the strand and the rebars require a sound electrical connection to each individual tendon (mostly mounted to one of the end anchorages) and another connection to the rebars. All the connection wires (area 1 mm² apart from the case of AC railway bridges where it is 6 mm²) are concentrated in a box easily accessible. Monitoring of the electrically isolated tendons shall be performed with AC impedance measurements at a frequency of 1 kHz with portable LCR meters. The instrument measures the impedance Z that includes (over the tendon length) the grout in the duct, the duct (with couplers, vents, pores and defects) and the concrete surrounding the duct. Grout and concrete are pure resistances, the intact polymer duct is a pure capacitance and "defects" are represented by ohmic resistances in parallel. The instrument calculates and displays the ohmic resistance R, the capacitance C and the loss factor D.

![Grouted plastic duct with steel strands](image1)

![LCR meter](image2)

**Figure 6: Measurement principle**

![Electrical impedance of a tendon](image3)

**Figure 7: Electrical impedance of a tendon**

The capacitance C is constant for a specific tendon length, diameter and material, it increases proportionally to the length of the tendon. The ohmic resistance R for a given tendon decreases with its length. For a good electrical isolation the specific value $\rho$ (gm) should be as high as possible, limiting values are given in [2]. $\rho$ increases with time due to hydration and drying out.

Laboratory measurements of the impedance on 1 m long segments of PT Plus ducts (0.59 mm) with defects of different size are shown in figure 8. The line with the -1 slop in the bode plot represents the capacitive impedance, the resistance decreases with increasing defect size. It can further be noted that a grout vent corresponds to an equivalent very small defect.

The measurements of R, C and D allow to determine the degree of electrical isolation (and thus of the tightness of the duct on its whole length) at any time after grouting. This can be used both for quality control and for long-term monitoring of the corrosion protection of posttensioned tendons as will be shown in the following papers.
4. Conclusions

Electrically isolated tendons with plastic ducts offers enhanced corrosion protection of the steel strands and the possibility to monitor the corrosion protection by simple non-destructive measurements (electrical impedance).

The electrical impedance ("electrical resistance") of an internal grouted tendon can be represented by a parallel circuit of a capacitance (of the duct) with a resistance (related to the defects in the duct).

The measured capacitance depends on the material, duct geometry ( thickness, diameter) and increases with the length of the duct. The measured resistance decreases with the length of the duct.

Figure 8: Impedance measurements on 1 m long PT plus ducts (ø 59 mm) with different defects [5].
References


Protection against corrosion and monitoring of posttensioning tendons in prestressed concrete railway bridges in Italy

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Abstract

The paper describes the state of the art and future trends in Italy with regard to durability and monitoring of post-tensioning tendons in prestressed concrete bridges and viaducts for high-speed and ordinary railways. Two major tasks have been considered in the planning and design of these structures: durability and possibility of monitoring. In order to reach these goals in the frame of post-tensioning systems for prestressed concrete bridge decks, the thick-walled corrugated plastic ducts for bonded internal tendons have been applied and electrically isolated anchorages have been adopted, as first applications of these systems in Italy. The first data on quality control are presented.

Keywords: Post-tensioning; Durability; Plastic ducts; Electrically isolated tendons; Monitoring of post-tensioning systems

1. Introduction

Post-tensioning tendons contribute decisively to the serviceability, safety and durability of prestressed concrete structures, especially of road and railway bridges. Optimum corrosion protection of post-tensioning tendons has been a priority since the beginning of this technology. The critical aspects of durability of post-tensioning tendons are summarized in a recent state-of-the-art report [1]: main problems are corrosion due to the ingress of chloride containing water and the lack of an established non-destructive technique that is able to obtain reliable information on the corrosion protection of the prestressing steel. Several national guidelines approved in the last years try to improve the situation. In Switzerland the SIA codes state that depending on the environmental conditions more stringent requirements may apply to achieve durability of post-tensioning tendons. The guideline "Measures to ensure the durability of post-tensioning tendons in bridges" published recently in English [2] thus explicitly requires the use of electrically isolated tendons with grouted plastic ducts for longitudinal tendons with low structural protection (concrete cover), high risk of chloride ingress and/or particular requirements for long term monitoring. In Italy the new high-speed train lines have been planned and designed according to two major goals that are durability and possibility of monitoring [3]. Industry developed,
manufactured and improved in the last years thick wall plastic duct systems including electrically isolated anchorage and couplers where the prestressing steel is protected on the whole tendon length. The requirements for these plastic ducts are defined in a fib technical report [7]. The electrical isolation protects against stray currents and, more important, allows to assess the degree of the corrosion protection of the prestressing steel both during construction (quality control) and during the service life of the structures (long term monitoring) by measuring the electrical impedance between the prestressing steel in the duct and the normal rebars in the structure [2,4,5]. All the results are further evaluated and compared to specific (independent of the length of the tendon) values of the capacitance, the electrical resistance and the loss factor from laboratory tests of grouted plastic ducts [4]. In addition, possible solutions for practical execution problems encountered are proposed.

2. Electrically isolated tendons

The design of an electrically isolated system for internal bonded post-tensioning differs from the normal one in order to a) guarantee the maximum degree of protection of cables against corrosion, and b) to allow at any time during the service life of the structure checking of the integrity of the protection with a rapid non-destructive method. The plastic duct fully encapsulates and isolates the strand bundle on the whole tendon length. Detailing is most important near the anchorages in order to guarantee complete encapsulation and electrical isolation [5]. Also the connection to the plastic trumpet, vents and drains need great care.

Another advantage of plastic ducts is the reduction of friction losses. The friction coefficient J.1 is around 0.14 instead of 0.3 for steel ducts. The phenomenon of fretting fatigue is greatly reduced.

2.1 Anchorage

Between the steel anchor head (a in fig. 1) with the wedges that block the strands and the cast iron bearing plate (b), a mechanically resistant insulation plate (c) is placed in order to electrically isolate the tendon from the normal rebar network. Inside the anchorages a plastic trumpet (d) tightly connected to the duct (c) isolates the strands from the cast-iron bearing anchorage.

Figure 1: Electrically isolated tendon
According to Swiss guideline [2], the anchor heads are protected by plastic caps (f) and fully grouted. Italian Railways asked for transparent caps in order to check visually the grouting execution of the caps. The guideline leaves to the designers the choice to cover the anchorage caps with concrete or not. The Italian Railways choice was to ask to all designers for head anchorages completely surrounded by non-shrinking reinforced concrete (g) as a mean of further protection.

2.2 Duct

Ducts are usually made of high-density polyethylene or polypropylene: the fib Technical Report [7] specifies mechanical resistance and chemical requirements for the acceptance of a plastic duct. In particular, any new prestressing system with plastic ducts has to pass a System Approval Testing [7]. The Italian Railways have recently started to apply System Approval Testing to every new system proposed by the construction companies. So far only one electrically isolated system has been approved and used on site.

![Figure 2: Core from System Approval Testing (Modena viaduct, Italian high speed railway)](image)

The assembling of the ducts may be solved with plastic couplers produced by the prestressing companies, by thermal welding of the head faces with a special machine, before assembling, or by plastic collar and outer thermal shrinking sheath. Also the connection to the plastic trumpet of the anchorages must be carefully executed, with a watertight gasket and thermic shrinking sheath. The end of the grout inlets and vents should be carefully closed with a tight cap, otherwise an electrolytic pathway with preferential ingress of chloride containing water is formed. Leak test of the whole duct before grouting is important.

2.3 Measuring the impedance of the duct

The measurements of the electrical isolation between the strand and the rebars require asound electrical connection (h in fig. 1) to each individual tendon (mostly mounted to one of the end
anchorages) and another connection to the rebars. All the connection wires (area 1 mm² apart from the case of AC railway bridges where it is 6 mm²) are concentrated in a box easily accessible. Monitoring of the electrically isolated tendons was performed with AC impedance measurements at a frequency of 1 kHz with portable LCR meters. The instrument measures the impedance Z that includes (over the tendon length) the grout in the duct, the duct (with couplers, vents, pores and defects) and the concrete surrounding the duct. Grout and concrete are pure resistances, the intact polymer duct is a pure capacitance and "defects" are represented by ohmic resistances in parallel. The instrument calculates and displays the ohmic resistance R, the capacitance C and the loss factor D.

The capacitance C is constant for a specific tendon length, diameter and material, it increases proportionally to the length of the tendon. The ohmic resistance R for a given tendon decreases with its length. For a good electrical isolation the specific value p (Ωm) should be as high as possible, limiting values are given in [2]. P increases with time due to hydration and drying out.

The measurements of R, C and D allow to determine the degree of electrical isolation (and thus of the tightness of the duct on its whole length) at any time after grouting. This can be used both for quality control and for long-term monitoring of the corrosion protection of posttensioned tendons.

Figure 3: Precast deck of Piacenza viaduct, Italian high speed railway
3. Quality control

28 days after grouting the impedance measurements shall be performed for the first time [2]. The results can be used as a first quality control of the degree of electrical isolation. This quality control may be particularly useful when a great number of identical post-tensioned decks are produced in a pre-casting plant as it is the case for many railway bridges in Italy.

The first application of this technology is the Piacenza Viaduct (figure 3 and 4), on the Milano-Bologna high speed line: 151 simply supported pre-cast prestressed concrete decks composed by a monolithic box girder with two cells, each spanning 33.1 m and weighing about 1000 tons. Other continuous prestressed concrete slab decks are under construction and the results of tests will soon be available.

So far, data have been collected from the first 71 decks of the Piacenza viaduct, each containing 9 cables with 12 strands, duct Ø 76 mm (in the lower slab) and 15 cables with 19 strands, duct Ø 100 mm (in the webs), both typologies of about 32.1 m length (figure 3).

A first control on the execution quality allows the values of the capacitance C: the mean value of the capacitance increases with the diameter of the duct, the specific capacitance (per meter length) is well below the limiting value specified in [2]. Thus the wall thickness of the duct is higher than required for electrical isolation and the execution is sufficiently well done.

Table 1: Mean values and standard deviation of the capacitance values of the cables tested

<table>
<thead>
<tr>
<th>Type</th>
<th>C mean [µF]</th>
<th>Std dev [µF]</th>
<th>C spec [µF/m]</th>
<th>C lim [µF/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ø 76 mm</td>
<td>70.3</td>
<td>2.3</td>
<td>2.2</td>
<td>&lt; 3.05</td>
</tr>
<tr>
<td>Ø 100 mm</td>
<td>73.5</td>
<td>2.2</td>
<td>2.3</td>
<td>&lt; 3.35</td>
</tr>
</tbody>
</table>
The statistical analysis of the measured resistance $R$ is more complicated because the values do not show a gaussian distribution. The analysis is thus performed with the cumulative probability plot (figure 5). The plot shows that overall less than 1% of all the values are below 10 Ohm, thus cables with a short circuit (electrical contact) between the strand and the rebars. The limiting value $R$ (300 $\Omega$ m / 32.1 m) is not reached by about 8% of the cables with the duct diameter of 100 mm. Individual cables show even better performance (e.g., cable 11, 17). From figure 5 it can also be noted that for every cable nr. one or more segments were produced with a perfect isolation (reaching the theoretical value of a completely tight plastic duct).

The increasing number of segments that will be produced in the future will improve the statistics. This will allow to review the acceptance criteria defined in the Swiss guideline [2] and to get a more statistically founded approach in defining them.
4. Conclusions

The experience with the new electrically isolated post-tensioning system with plastic ducts in Italy is very encouraging.

Measuring the electrical resistance has been shown to be an efficient way for quality control especially when a number of cables has to be checked in a large amount of simply supported spans.

The analysis of the measured capacitance values provides additional information on the quality and reproducability of the execution.

The high number of identical cables in the segments will allow to review the limiting value of the electrical resistance proposed in the Swiss guideline and put it on a more statistical basis.

The large scale application of this technology in some high speed railway viaduct in Italy will allow the Italian Railways to reach enough confidence with this system and to apply it in the future when durability of post-tensioning tendons has to be improved.

Acknowledgments

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References


Experience with electrically isolated tendons in Switzerland

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Abstract

Electrically isolated tendons with plastic ducts for internal grouted post-tensioning were developed about 15 years ago. This new generation of tendons offers enhanced corrosion protection of the steel strands and the possibility to monitor the corrosion protection by simple non-destructive measurements (electrical impedance). In this paper the Swiss experience will be presented and discussed with respect to long term monitoring, the most frequent failures and the value of the limiting values of the specific resistance given in the Guideline.

Keywords: Electrically Isolated Tendons, long term monitoring, smart structures

1. Introduction

Tendons contribute decisively to the serviceability, safety and durability of prestressed concrete bridges. Optimum corrosion protection of post-tensioning tendons has been a priority since the beginning of this technology [1]. The UK temporary ban of grouted post-tensioning tendons, from 1992 to 1996, has initiated a review of all aspects related to durability of posttensioning tendons [2]. Traditionally, ducts for bonded post-tensioning tendons have been made from steel stripes with a special corrugation. A long experience and generally good "track record" with these tendons is available from many different applications. Their overwhelming use has allowed the creation of national and international standards. Despite a generally good long-term behaviour, some corrosion problems are documented that rose concern about the durability of post-tensioned tendons. An overview on corrosion damages of prestressing steel in Germany has been published [3], the situation in North America, Europe and Japan is summarized in papers presented at the first fib/IABSE workshop "Durability of post-tensioning tendons" in 2001 [4]. Recently, the situation in Switzerland was analyzed [5, 6]. From 143 structures approximately half showed small to significant corrosion damage of the prestressing steel. From 27 sufficiently documented bridge case studies, 12 bridges were dismantled of which 9 for traffic reasons and only 3 due to lack of serviceability, structural safety and durability. In this group of dismantled bridges (where a much better tendon inspection is possible) significant damage with corrosion of the prestressing steel was found in 2 bridges. The reason was chloride-containing water penetrating at "weak points" such as expansion joints, drainage systems etc. to the steel duct [5, 6]. Plastic ducts have
been used for many years in prestressing technology for such applications as mono strands, ground anchors, stay cables and external tendons, mostly in the form of smooth pipes. An early example is the Schillersteg in Stuttgart completed in 1961; after 13 years of exposure to the environment no particular changes in the properties of the polyethylene have been observed [7]. Corrugated plastic ducts have been used for ground anchors in the bond length. Between 1968 and 1974 about 300'000 m of thin wall corrugated black polyethylene ducts have been installed in Switzerland for bonded post-tensioning (simply because the material at that time was cheaper than steel ducts). Nowadays some of these bridges were demolished, the analysis of these thin-walled polyethylene ducts after up to 30 years in use showed no deterioration and the steel strands did not present crevice corrosion. A renewed interest for plastic ducts occurred after 1980 when the phenomenon of fretting fatigue in bonded tendons was discovered and investigated [8, 9].

Only recently thick-walled plastic ducts for post-tensioning tendons have become popular for use with curved tendons. They have been mainly developed and introduced for three reasons:

- reduced friction losses during stressing of the tendon
- increased fretting fatigue resistance of the tendon [10]
- improved corrosion protection, especially in the case of stray currents, which is a vital aspect for railway bridges
- feasibility for electrical monitoring of the tendon

The research work and development of thick-walled plastic ducts for post-tensioning and the performance in practice is well documented in literature [11, 12], a recent fib report [13] summarizes the actual knowledge and presents a recommended specification for corrugated plastic ducts for bonded post-tensioning. The use of plastic ducts allows in principle the electrical isolation of the prestressing steel of the tendons from the rebars, thus providing for the first time the possibility of monitoring these important load-bearing parts of a structure.

2. Monitoring of electrically isolated tendons

The use of electrically isolated tendons allows to control the electrical isolation and the integrity of the duct after construction and to monitor the corrosion protection of the steel strands during service life with simple AC impedance measurements (usually called ‘electrical resistance measurements’). Since 1993 about 12 bridges or fly-overs have been constructed with electrically isolated tendons (EIT) in Switzerland, on some of them electrical impedance measurements were performed. In the following, results from two different flyovers, the P.S. du Milieu and Pres du Mariage, both over the National Highway AI in the Cantons of Vaud and Fribourg are presented.
2.1 Electrical connections and measuring procedure

The measurements require a sound electrical connection to each individual tendon (mostly mounted to one of the end anchorages) and another connection to the rebars. All the connection wires usually are concentrated in a box easily accessible (Fig. 1). Monitoring of the electrically isolated tendons was performed with AC impedance measurements at frequency of 120 Hz and 1 kHz. A small portable and battery driven instrument (LCR meter ESCORT 131) measures the real and imaginary part of the impedance of the tendon under test. The instrument calculates and displays the ohmic resistance \( R \), the capacitance \( C \) and the loss factor \( D \) for the measuring frequency chosen.

After opening of the box with the sockets and connection cables, a visual check of the sockets is performed, they should be dry and clean. The connection from the instrument to the socket "rebar" and "tendon number." is established using short laboratory cables with a diameter of 1 mm. The measurement is initialized by pressing the "measure" button and the results are written down in a form. Each tendon has been measured twice. Temperature and weather condition are documented.

Fig. 1: Connection box for the measuring cables
3. Field Results - control the integrity of the duct

The flyover "P.S. du Milieu" near Avenches (Fig. 2) is about 100 m long and consists of six spans with five columns. The anchorage zone is constructed with robust plastic ducts and plastic sleeve and electrically isolated anchorages. Six electrically isolated tendons (length 100 m, PT PLUS duct diameter 59 mm) were measured in order to control the integrity of the duct after construction. The temperature during measurements was 11 degree, there was no rainfall several days before the measurements. Table 1 presents the results of the impedance measurements at 1 kHz.

![Figure 2: Flyover "Pont du Milieu" with six electrically isolated tendons](image)

Table 1: Experimental and calculated specific values of the ohmic resistance $R$, the capacitance $C$ and the loss factor $D$ from the flyover "P.S. du Milieu" (length 100m)

<table>
<thead>
<tr>
<th>tendon Nr</th>
<th>experimental values</th>
<th>specific values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$R$ (kΩ)</td>
<td>$C$ (nF)</td>
</tr>
<tr>
<td>1</td>
<td>7.234</td>
<td>234.0</td>
</tr>
<tr>
<td>2</td>
<td>13.70</td>
<td>233.0</td>
</tr>
<tr>
<td>3</td>
<td>20.87</td>
<td>235.0</td>
</tr>
<tr>
<td>4</td>
<td>17.81</td>
<td>237.2</td>
</tr>
<tr>
<td>5</td>
<td>28.25</td>
<td>234.7</td>
</tr>
<tr>
<td>6</td>
<td>0.006</td>
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</tr>
</tbody>
</table>
As can be observed, the ohmic resistance $R$ is high or very high for five of the six tendons (nr. 1 - 5), tendon nr. 6 show a very low value, indicating a short circuit between steel strand in the duct and rebars. The capacitance values $C$ are nearly constant (234.8 ± 2 nF). The loss factor $D$ varies between 0.023 and 0.093. From the experimental values of the 100 m long duct the specific values (related to 1 m duct) were calculated by multiplying $R$ with the length and dividing the capacity $C$ by the length. The loss factor $D$ is independent of length. Note that for short-circuited tendons no further information can be obtained.

In order to control the integrity of the ducts after construction, the specific values of $R$, $C$ and $D$ calculated on the basis of the experimental values (Table 1) have to be compared with electrically isolated plastic ducts without any defects. These reference values were obtained from a laboratory study [5]. The capacitance $C$ of a plastic duct can be calculated knowing the geometry (length, diameter, wall thickness) and material properties (specific resistance $\rho$, dielectrical constant $\varepsilon$) of the duct. The capacitance $C$ increases proportionally with the length of the duct. In the laboratory study a specific capacitance of 2.34 ± 0.03 nF/m was obtained for PT-PLUS diameter 59 mm [5], the field results (table 1) are in perfect agreement. The specific ohmic resistance $R$ of a defect free grouted plastic duct in concrete was found to be ca. 4 M\(\Omega\)*m, in presence of a duct coupler ca. 2.8 M\(\Omega\)*m was measured in the laboratory study [5]. The specific ohmic resistance of the ducts in a real structure (table 1) are equal or lower than the laboratory value. This is due to system related imperfections in the 100m long ducts (e.g. couplers, not perfectly closed grout vents etc.) and possible defects in the duct, forming an electrolytic connection (low resistance path) between steel strand and rebar network. From the laboratory study [5], a grout vent showed an ohmic resistance of 0.5 M\(\Omega\), a hole of 2 mm in the duct ca. 0.1 M\(\Omega\). The Swiss Guideline [4] defines a limiting value of 500 k\(\Omega\)*m as acceptance criteria for the integrity of the duct (59 mm). Thus all the tendons of the flyover P.S. du Milieu (table 1) show a good to perfect electrical isolation with no defects present (otherwise the electrical resistance should be much lower). An additional confirmation can be obtained from the (length independent) loss factor $D$ that was found to be always < 0.1 (table 1) corresponding to the value of a grout vent $D = 0.098$, whereas a small defect of 2 mm showed a loss factor of 0.67 [5].

4. Field Results - long term monitoring

The flyover "Pres du Mariage" (Fig. 3) is a simple, short construction with only one column in the middle. In each bridge girder three tendons were installed. The electrical resistance (AC impedance) of each of the six individual tendons (length 49.3 m, PT-PLUS duct, diameter 76 mm) was measured at frequent intervals since the time of grouting. Temperature during measurements varied between 6 and 22 degree. The evolution of the electrical resistance $R$ with time is shown in Fig. 4. As can be noted, the six individual tendons show a certain scatter, but the overall trend is an increase of the resistance with time; in the log $R$ vs log $t$ plot (Fig. 5) a straight line with a slope of about 0.5 can be observed (on the linear scale a square root law). This indicates that the increase of the resistance is very rapid at the beginning but slows down after some months becoming asymptotic after several years.
This continuous increase of the electrical resistance can be explained by the hydration of the grout and the surrounding concrete [6] and the subsequent drying out; this trend is expected to continue in the future service life of the structure. This allows to detect the ingress of water in a very early stage: if (chloride containing) water reaches a defect in the duct, the concrete and the grout get wet and the electrical resistance of this tendon will not increase any more but decrease steadily. Thus the measurement of the electrical impedance at the normal inspection intervals represents a simple but very effective early warning system to detect a water (and chloride) ingress and thus a future corrosion risk situation.

Figure 3: Flyover Pré du Mariage with 6 electrically isolated tendons ø 76 mm

Figure 4: Evolution of the electrical resistance of the six tendons with time
Since about 1993 more than 80 structures (mostly bridges) with tendon length from less than 20 m to more than 600 m were built in Switzerland with electrically isolated tendons. Overall the experience with the new technology is very positive. The possibility to measure the electrical resistance of the tendon and thus the overall quality of the work is new for civil engineering - to be confronted with a defined limiting value that has to be reached (electrical resistance 28 days after grouting) leads frequently to (not very fruitful) discussions.

From the data available it can be estimated that about 60% of all tendons fulfil the criteria of the electrical resistance (the Swiss guideline [16] would require 90%). In some of the structures the reasons for this relatively poor behaviour was analysed. It was found that about half of the tendons showed a short circuit (resistance < 10 Ω), the other half showed resistance values below the limiting value.

5. Discussion

The fact that a simple measurement can control the quality of the electrically isolated tendons is nearly a revolution in civil engineering. For structures where some of the tendons did not fulfill the criteria, discussions among owner, contractor, engineer and subcontractor are common. As long as only a "guilty" has to be found this makes not much sense.

The quality of the "product" electrically isolated tendon is determined by many individual steps and persons. The idea should thus be to try on all levels and together to reach the required quality. Some important points will be biefly discussed here.

- The role of the owner: the owner of a structure has to define clearly from the beginning that electrically isolated tendons (protection level 3) will be used in the construction. This has to be specified in all tender documents.

- The role of the design engineer: the engineer has to be aware of the special requirements necessary in structures with electrically isolated tendons. The most important point is a smooth curvature of the tendon. Then enough space for the tendons between the normal rebars, should be left and the accessibility of anchorages should be assured. In detail supporting half-shells below and above contact points with normal rebars (if they cannot be avoided) have to be specified already in the design.

- The role of the contractor: the contractor (and especially all the personal working on site) has to be aware of the differences between a metallic and a plastic duct (material properties, care that has to be taken during installation). Any abrupt change in curvature has to be avoided, persons should not walk on the ducts, no soldering of rebars close to plastic ducts, etc.

- The role of the subcontractor: the subcontractor (usually the company delivering the electrically isolated tendons, responsible for tensioning and grouting) should take all measures not to damage the plastic ducts when inserting the steel strands (protection cap on the sharp ends of the strands). Before inserting the strands the duct should be controlled for tightness (light in the night, development of artificial fume,...).
The most important point is that all persons involved recognize that the production process of an electrically isolated tendon is like a chain: everyone involved has his specific responsibility for the success - this has much to do with communication and teamwork.

The limiting values specified in the guideline (editon 2001) [16] are - as is explicitly stated - preliminary and might be subject to revision (based on experience) in the edition 2006. This is due to the fact that these values are based on laboratory work [14, 15] and on a small number of structures with long tendons (50 to 100 m). The percentage of the anchorage zone to the total length is thus small - on short tendons possibly different criteria have to be defined to take into account this fact. Further the limiting values were defined on a small number of identical tendons, thus the aspect of statistical distribution of the resistance values of a "good" tendon could not be taken into account. Progress on this point can be expected from the data becoming available from the segmental viaducts of the Italian Railways [17, 18].

6. Conclusions

Electrically isolated tendons with plastic ducts offers enhanced corrosion protection of the steel strands and the possibility to monitor the corrosion protection by simple non-destructive measurements (electrical impedance).

Tendons that fulfil the required acceptance criteria (electrical resistance) have been produced in great number on all type of structures. Increasing dissemination of knowledge for owners, design engineers, contractors and the persons working on site is necessary to avoid tendon failures.

Electrically isolated tendons (even when the acceptance criteria is not reached) can be monitored over time. The electrical resistance measurements are - for the first time - a simple early warning system that can identify tendons with a future corrosion risk.

References


Swiss Experience — long term monitoring

[18] M. Della Vedova, L. Evangelista, this conference