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Session D3

Monitoring of Structures
Surveillance des structures
Überwachung von Bauwerken

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In-Service Monitoring of Structures

Surveillance des constructions en service

Ueberwachung bestehender Bauwerke

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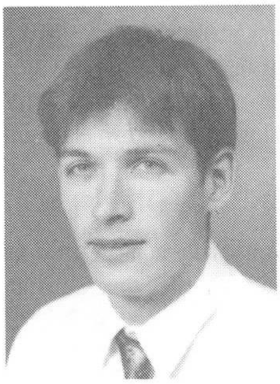

Richard Moss, born 1963, obtained his first degree at Sussex University and his PhD from Imperial College, London University. During a period of 10 years at BRE his research has concentrated on load testing and structural monitoring, although he has also carried out general appraisal work.

SUMMARY

This paper addresses the subject of monitoring of structures on a philosophical basis. Issues addressed include reasons for installing a monitoring scheme, problems associated with monitoring schemes, methods of monitoring and instrumentation. The paper briefly describes methodologies for effective instrumentation deployment based on concepts of robustness and vulnerability. Different monitoring techniques and in particular the advantages and disadvantages of visual and manual methods vis-a-vis autonomous systems are also discussed.

RÉSUMÉ

L'article traite de la surveillance des structures du point du vue philosophique. Parmi les sujets abordés, il y a lieu de citer les raisons qui poussent à installer un système de surveillance, les problèmes associés aux-dits systèmes, les méthodes de surveillance et l'instrumentation. Il donne en outre un bref descriptif des méthodologies utilisées pour un déploiement efficace de l'instrumentation en se basant sur les concepts de robustesse et de vulnérabilité. Y sont également traitées les différentes techniques de surveillance, en mettant en particulier l'accent sur les avantages et les inconvénients que présentent les méthodes visuelles et manuelles vis-à-vis des systèmes autonomes.

ZUSAMMENFASSUNG

Dieses Referat behandelt das Thema Überwachung von Bauwerken auf philosophischer Basis. Unter anderem werden folgende Fragen in Erwägung gezogen: Gründe für die Installation eines Ueberwachungsplans, mit Ueberwachungsplänen verbundene Probleme, Methoden zur Überwachung und Instrumentarium. Das Referat beschreibt in kurzen Zügen die Methoden des wirksamen Instrumenteneinsatzes auf der Basis der Konzepte der Robustheit und der Verletzbarkeit. Verschiedene Ueberwachungsverfahren und insbesondere die Vorteile und Nachteile von visuellen und manuellen Systemen im Vergleich zu autonomen Systemen werden ebenfalls besprochen.

1. INTRODUCTION

1.1 There are an increasing number of buildings that require informed decisions to be made about their continued safety and serviceability. The approach adopted to the design of structures has traditionally been the performance of calculations to justify the adequacy of the proposed construction in accordance with codes of practice. Assumptions are made as to the loads to which structures are subjected and the structure's behaviour.

1.2 The structure is considered at different hierarchical levels - typically the performance of individual elements and frame assembles - with assumptions made concerning the boundary conditions applicable, coupled with consideration of overall behaviour (eg ensuring structural stability). Such an approach has been necessitated by the fact that most structures are 'one-offs' and it is not feasible to codify for individual structures. However with technological developments in the fields of monitoring instrumentation, data logging and software, consideration of the actual as-built performance of individual structures and classes of construction is becoming a realistic possibility.

2. REASONS FOR INSTALLING A MONITORING SCHEME

2.1 There may be many reasons for employing a structural monitoring system and these will generally dictate the length of time over which the monitoring system is required to function. Depending on the reasons for installing the monitoring system it may be installed during or subsequent to construction. For monitoring equipment installed subsequent to construction there may be clearer and more direct objectives which can be set for the monitoring scheme. This is because in such cases the monitoring system will usually be installed to address specific problems identified with the structure.

2.2 It is obviously crucially important that the equipment used to monitor the response of the structure is suitable. Different types of equipment are likely to be needed for structures whose response is predominantly dynamic rather than static in nature.

2.3 Supplementary to the basic concept of monitoring the structure, there is the idea of testing the structure at regular intervals to check for changes in the response. It is advocated that such testing can give early indication of structural degradation.

2.4 Since measurements of load are difficult to make directly, static load testing potentially has a useful rôle to play in the initial calibration of the monitoring system, thereby providing a benchmark against which the results of subsequent monitoring of the structure in service can be assessed.

3. PROBLEMS ASSOCIATED WITH MONITORING SCHEMES

3.1 In devising any monitoring scheme the first step is naturally to define the objectives and purpose of the monitoring scheme. Considerable thought needs to be given to the information required and the appropriate instrumentation. Presentation and storage of the information is also important, and consideration needs to be given to what the data collected will be used for.

3.2 For economic reasons it is not feasible to instrument the whole of a structure and a rational basis is required for selection of positions to monitor which represents the worst case within a structure. Approaches to this problem are considered further in Section 6.

3.3 For long-term monitoring (ie over many years) the reliability and stability of the instrumentation used is of crucial importance. It is necessary to identify the parameters which it is desirable to measure and the most appropriate instrumentation to use for measuring these parameters. Types of instrumentation which may be suitable are described further in Section 5.

4. ADVANTAGES AND DISADVANTAGES OF VISUAL AND MANUAL METHODS

4.1 Manual methods have relatively low initial set-up costs compared to automatic or autonomous systems, but incur relatively high repetitive costs each time a set of measurements is taken. Accordingly manual methods are likely to be employed when either the programme of monitoring is expected to be short and involve only a limited amount of instrumentation, or where monitoring of a limited number of instruments will take place over a longer time-scale with repeat measurements taken infrequently.

4.2 The economic balance between automatic / autonomous and manual data collection methods is constantly changing. Technology continues to enhance the sophistication of instrumentation and data logging equipment, whilst reducing their cost in real terms. Many automatic systems can be controlled remotely via a modem and this can reduce the need for periodic visits to site.

5. INSTRUMENTATION

5.1 The instrumentation considered here is in general restricted to that which is capable of being incorporated within data logging systems so that measurements can be taken automatically and remotely.

5.2 Instrumentation for monitoring of structural behaviour can be categorised broadly into instrumentation which measures actions on the structure (eg imposed loads, wind loads and temperature changes) and that which measures the response of the structure to those actions (eg deflections, vibrations, rotations, strains and resultant stresses).

5.3 The need for dynamic capability of the instrumentation is governed by the anticipated behaviour of the structure (ie whether predominantly static or dynamic in nature), and is principally related to the type of structure being considered. Dynamic actions and responses will need to be measured for bridges and offshore structures but not for most buildings. Other structures and structural elements likely to exhibit a dynamic response include long-span floors, masts and chimneys.

5.4 Direct measurement of loads is usually achieved using load cells. Because of the high dead load component in structures, small changes in load require to be resolved against a background of high standing load. This calls for a high degree of resolution and hence stability of the instrumentation. Of the different types of load cell available, vibrating wire load cells have better long-term stability than load cells with electrical resistance strain gauge elements.

5.5 Installation of pressure transducers on the ends of pressure tappings can allow direct measurements of wind pressures on the elevations of buildings. This has been done extensively for small-scale models tested in wind tunnels and more recently at full scale [1]. Pressure transducers measure pressure difference, not absolute pressures. For this reason a static probe is needed to allow a reference pressure to be established against which other pressure measurements can be compared.

5.6 Pressure transducers are normally used in conjunction with an anemometer to measure wind velocity. The anemometer can be used to control the logging process by triggering logging of the transducers only when a minimum threshold velocity is reached.

5.7 Of the different types of device available for measuring temperature, thermocouples are generally considered to be the most suitable. Although not as accurate as some other types of temperature measurement device, they are likely to be sufficiently accurate for most structural monitoring purposes.

5.8 The type of instrumentation most suitable to monitor long-term deflections of structures will depend on a number of factors such as the accuracy with which measurements are required to be made, the accessibility of the positions where movements are required to be measured, and the ease of providing a fixed frame of reference.

5.9 Where a frame of reference can be established reasonably easily for individual measurement points, the simplest method of measuring displacements is to use displacement transducers. These are usually either of resistive or inductive type design, being termed potentiometers or linear variable differential transformers (LVDT's).

5.10 Where a frame of reference cannot easily be established, other techniques such as use of hydraulic levelling systems and lasers could be adopted. The choice between these two is likely to depend on factors such as the visual access of the positions at which measurements are required to be made and the distances involved.

5.11 Where the installation of a reference frame is not possible or would be obtrusive, an alternative method of deriving deflections is to measure slope and integrate to calculate deflection.

5.12 Perhaps the most promising device for measuring rotation for the purpose of long-term monitoring is the electrolevel. Electrolevels can be made waterproof, and this makes them particularly suitable for use in hostile environments. However electrolevels are not appropriate for taking dynamic measurements and can be adversely affected by variations in temperature.

5.13 The vibration measurement device most suited to long-term monitoring applications is generally considered to be the geophone. This is a relatively cheap and robust device and does not need separate amplification and signal conditioning equipment. However geophones are generally not as accurate as accelerometers.

5.14 The two types of device commonly used for measuring strain are vibrating wire strain gauges and electrical resistance strain gauges. Electrical resistance gauges do not have as good long-term stability as vibrating wire gauges.

5.15 The instrumentation considered above is suitable for incorporating within a data logging system. Such a system becomes essential when the instrumentation needs to be read remotely and a large number of instruments require to be read within a relatively short time span. It is also necessary to have an autonomous system where alarms are to be activated automatically when threshold values are exceeded.

5.16 Distributed data acquisition systems have advantages over more conventional data logging systems for use in structural monitoring, particularly for large structures, because of the savings in cabling requirements. Distributed systems also allow the logging device to be located in the proximity of the instrumentation and this can be important to preserve signal quality.

6. METHODOLOGIES FOR EFFECTIVE INSTRUMENTATION DEPLOYMENT

6.1 BRE has been considering methods for deciding which parts of structures should be monitored, since it will not be possible to instrument a whole structure and a rational basis is required for deciding which parts of a structure to instrument. In many cases this will be determined by logistical constraints, in terms of which parts of the structure are accessible and where installation of instrumentation is acceptable aesthetically. However it is still useful to have a theoretical basis for deciding optimum instrumentation deployment.

6.2 With this in mind, BRE has let a contract to look at the possible application of vulnerability theory to this problem.

6.3 The emphasis of structural vulnerability analysis is not the usual one of analysing a structure under some given loading condition. Rather it is to examine the vulnerability of a structure to any possible loading action. This is accomplished by examining the quality of well-formedness of what are termed structural rings at various levels of definition within a structure, and those rings which are the most vulnerable or critical together with the actions which might cause failure.

6.4 A structural ring is a load path which is capable of resisting an arbitrary set of applied forces. A structure can then be represented at various hierarchical levels of definition in terms of clusters of interconnected structural rings.

6.5 The well-formedness of a structural ring is a measure of its ability to resist loading from any arbitrary direction. The well-formedness depends on the orientation and stiffness of the members within the ring and the stiffness of the joints connecting the members in the ring.

6.6 Vulnerability theory thus considers the geometric layout of the structure under consideration and homes in on the most critical elements within the structure.

6.7 It is intended to apply the methods described above to predict where instrumentation should be installed in structures which are to be monitored in the field.

7. MONITORING OF STRUCTURES IN THE FIELD

7.1 The Section in which the author works has been involved in the monitoring of a number of structures in the field. However the structure with which the author has had particular involvement is known as the Swaminarayan Hindu Mission Complex and is in Neasden, NW London.

7.2 The aims behind monitoring this particular structure, and hopefully others in the future, are principally to gain a better understanding of the behaviour of structures in service and the loads to which they are subjected.

7.3 In deciding which parts of the structure to monitor, design drawings were considered detailing the various forms of construction and materials used. The main part of the structure comprises an assembly hall comprising an outer steelwork frame and a series of long-span trusses giving clear spans of 45 m. Adjacent to the assembly hall are areas constructed in reinforced concrete.

7.4 Initially it was decided that it would be interesting to carry out the following :

1. Instrument the main steelwork frames - measure loads in columns, deflections of beams and loads / strain in bracing members.
2. Measure loads in reinforced concrete columns.
3. Measure deformations of both ribbed and solid reinforced concrete slabs.
4. Instrument the steelwork trusses - measuring loads / strains in the top and bottom chords near the centre, and diagonal members near the ends of the trusses. If feasible measure the overall deflections of the trusses.

7.5 In the event financial and other constraints limited the amount of instrumentation that could be installed and concentration was placed on monitoring the steelwork trusses.

7.6 Two out of a total of seven main trusses were instrumented, the decision on which trusses to instrument being governed by the loading likely to be seen by these particular trusses in service. As the trusses were symmetrical it was decided to measure the outer fibre strains in the most heavily stressed members in one half of each truss only. This was achieved using vibrating wire (VW) strain gauges (total 8 gauges per truss).

7.7 It was not practicable to measure the deflections of the trusses directly and in an attempt to measure the deflections in service a series of electrolevels was installed on the bottom chord of each truss. These devices measure rotation and in deriving deflections the slope measurements have been integrated assuming local / secondary bending effects between node positions can be ignored.

7.8 Wiring from all the instrumentation on each truss is fed back to a single data collection point and the intention is to install a data logger with a modem link so that data from all the instrumentation can be accessed remotely via a telephone line.

8. CONCLUSIONS

1. There are an increasing number of buildings that require informed decisions to be made about their continued safety and serviceability. With technological developments in the field of monitoring instrumentation, data logging and software, consideration of the actual as-built performance of individual structures and classes of construction is becoming a realistic possibility.

2. There may be many reasons for employing a structural monitoring system and these will generally dictate the length of time over which the monitoring system is required to function.

3. For monitoring equipment installed subsequent to construction there may be clearer and more direct objectives which can be set for the monitoring scheme. This is because in such cases the monitoring system will usually be installed to address specific problems identified with the structure.

4. For economic reasons it is not feasible to instrument the whole of a structure, and a rational basis is required for selection of positions to monitor which represents the worst case within a structure. Approaches are described which may enable such positions to be determined.

5. Manual methods of monitoring are likely to be employed when only a limited amount of instrumentation is involved and a limited number of measurements is to be taken. An autonomous system is necessary when the instrumentation is required to be read remotely or alarms are to be activated automatically when threshold values are exceeded.

6. Distributed data acquisition systems have advantages over more conventional data logging systems for use in structural monitoring, particularly for large structures.

7. Monitoring appears to have an important rôle to play in the management of structures and potentially extending their useful life. This is particularly so given developments in data handling and interpretation methods which are occurring.

9. REFERENCE

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Issues in Instrumented Bridge Health Monitoring

Surveillance de l'état de santé de ponts par capteurs

Brückenzustandsüberwachung mit Instrumenten

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Emin Aktan received his Ph.D. in engineering from the University of Illinois at Urbana-Champaign in 1973. His research into the structural identification of constructed facilities is motivated to contribute to the state-of-the-art in analytical modelling and condition assessment of existing construction.

SUMMARY

A market survey and laboratory evaluation identified the most promising sensors and data acquisition systems for infrastructure application. A pilot monitor was implemented on a steel-stringer bridge in Cincinnati for traffic and environmental monitoring. This research improved understanding of the actual loading environment and the corresponding res-ponses of a highway bridge. Instrumented monitoring is expected to complement inspec-tion methods, provide an objective measure of the state-of-health, and alert bridge officials to deterioration or failure.

RÉSUMÉ

Une étude de marché et une évaluation en laboratoire ont évalué les capteurs et les systèmes d'acquisition de données les plus prometteurs en matière de surveillance d'infrastructure de ponts. Un appareil prototype de surveillance a été installé sur un pont métallique à Cincinnati pour mesurer le trafic et les conditions d'environnement. Ce projet a permis d'améliorer la connaissance de la charge réelle et du comportement correspondant du pont. La surveillance avec ce type d'instrumentation devrait compléter les méthodes d'inspection, permettre une mesure objective de l'état de santé du pont et alarmer les responsables du pont en cas de détérioration ou de panne.

ZUSAMMENFASSUNG

Eine Marktstudie und Laborauswertungen identifizierten die vielversprechendsten Sensor- und Datenerfassungssysteme für Überwachung der Brückeninfrastruktur. Ein Bildschirm wurde in eine Stahlbrücke bei Cincinnati eingebaut um Verkehrs- und Umwelt-einflüsse zu beobachten. Diese Untersuchung ergab ein tieferes Verständnis für die Belastungen und das damit verbundene Verhalten der Brücke. Diese Art der Überwachung soll ergänzend zu anderen Möglichkeiten gesehen werden. Sie soll eine objektive Messung des Bauwerkzustandes gestatten und so die Brückenverantwortlichen bei Verschlechterungen oder Versagen alarmieren.

1. PROBLEM STATEMENT

There are two significant reasons why we should be interested in instrumented bridge monitoring. The first reason is to improve our understanding of the actual loading environment and the corresponding bridge responses. With this knowledge, better design, construction and maintenance practices can be initiated. The second reason is to explore whether the information from monitoring may properly complement the current practice of bridge inspections, in order to provide an objective measurement of the state of-health or reliability of the bridge.

This research project aimed at exploring in two phases the multi-disciplinary issues and advancement of the state-of-knowledge in instrumented monitoring of highway bridges. The first phase necessitated a rigorous investigation of commercially available hardware for the bridge monitor in terms of cost, laboratory verification, field accuracy, and useful lifetime. In the second phase, the initial design of the monitor system optimized the installation of the scaffolding and wireway framework; the number, location, and positioning of selected high-speed and long-term strain gages on steel girders; and, the installation of a weather station on a typical three-span steel-stringer bridge in Cincinnati (HAM-42-0992). An integrated multi-disciplinary team of electrical and civil engineers was fully utilized to tackle the challenging research, optimal design, and analytical interpretation of the necessary components of the bridge monitor. For example, a civil engineer used finite-element modeling and cross-sectional analysis to define the optimal sensor types and positions for the bridge monitor, while an electrical engineer designed an accurate data acquisition system for these sensors based upon application-specific signal conditioning and hardware selection.

2. PHASE 1: MARKET SURVEY AND LAB VERIFICATION

The first phase of this research necessitated a rigorous investigation of commercially available hardware for the bridge monitor in terms of cost, laboratory verification, field accuracy, and useful lifetime (Fig. 1). One has to consider reliability in terms of the measured quantity and the interpretation of that quantity. A response that is measured by a sensor and data acquisition system will be composed of five components: (a) Transducer assembly errors (e.g., self-response of the transducer and the attachment assembly as a mini-structure); (b) Instrument/data acquisition variance errors (e.g., spurious readings or noise due to electromagnetic interference); (c) Instrument/data acquisition bias errors (e.g., drift or changes in calibration due to an impact or temperature effects); (d) Apparent structural response (e.g., unrestrained temperature strains and rigid-body displacements as well as rotations caused by settlements, temperature, creep, or shrinkage); and, (e) Structural response associated with stress and force. The logistics for longterm continuous monitoring include the careful decomposition of a sensor reading into the above components, and the resulting reliability of each.

The sources of errors and uncertainties in the strain measurement system may have their origins in the gage itself, the measurement circuit or in other portions of the instrument, such as power supply, amplifier, analog-to-digital converter, etc. To make certain that static characteristics of instruments such as repeatability, linearity, accuracy, hysteresis, sensitivity as well as instrument gage factor, fall within the published tolerances which the manufacturer provides, the sensors are individually calibrated by measurement-system scaling techniques in the laboratory. This requires the input of a series of known displacements by utilizing the micrometer. The calibrator used reads directly in ten-thousandths of an inch over one inch range. Calibration was performed with three cycles of extensions and retractions, over the useable and application ranges for each instrument. The researchers tried to simulate the field test system with respect to data acquisition settings, cable length, and connections, that will be used in actual bridge monitoring and testing. This allowed the error of entire system to be measured.

For weldable foil and vibrating wire strain gages, a simply supported structural steel rolled W6x20 beam was loaded to induce constant moment along the span and used as a calibration structure. The beam was tested in the elastic range, by applying two concentrated loads 34.5" apart, creating stresses up to 140 MPa at the flanges at the midspan. To obtain redundancy, more than one sample of the same kind of gages were placed at the spots that would yield the same reading. A finite element analysis of the beam was calibrated based on the measured mid-point deflection of the beam. Longterm and shortterm monitoring were performed on this calibration beam and strain readings of various gage clusters were compared with FE analysis results.

Long-term reliability of sensor clusters and data acquisition hardware were evaluated with respect to temperature, humidity, and ultraviolet radiation. The QUV Accelerated Weathering Tester is a laboratory simulation of the damaging forces of weather, for the purpose of predicting the durability of certain transducers exposed to the outdoor environment. Cyclic environmental tests of sensors mounted on free moving steel plates were conducted with controlled heat, humidity, and UV radiation. For simulation of freezing winter conditions, a simply supported structural steel profile beam was instrumented and placed in a freezer, to serve as a calibration structure. These weathering tests were monitored over long-term (weeks) in order to estimate gage reliability. Static and dynamic temperature tests were conducted to measure the apparent effect on sensor readings. There were many problems discovered with these tests and, in some cases, resulted in the modification of vendor literature or even gage design.

After the tests, it was concluded that the smaller Geokon Vibrating Wire models (VK 4100 and 4150) were not adequate due to their extreme sensitivity to installation and to hysteresis observed during load testing. Although Hitec Products foil gages performed well under load testing, they showed erratic reading when bathed with moisture in the Weathering Tester. Also contrary to vendor specifications, the full bridge strain gages showed considerable response to temperature. During the calibration process and the beam tests, it was observed that the accuracy and resolution of the Tokyo Sokki Kenkyujo clip gages were not suitable for the bridge monitoring under service loads. Further, the vendor's specification of gage factor was found to be in error by a magnitude of ten.

The Optim MEGADAC was chosen for data acquisition due to its proven performance and accuracy in past research experiments. Lab testing and analysis related to electronic acquisition of data included, electrical I/O calibration, cable shielding and losses, surge protection, proper grounding, filter and gain selection, minimum sample frequency, and software optimization. The selected Tokyo Sokki Kenkyujo AWC-8B is a weldable strain gage especially designed for use in harsh environments and longterm measurement. The gage is constructed of a strain element enclosed in a stainless steel tube, a metal carrier base for spot-welding, and an integral shielded 3-wire system. The selected vibrating wire Geokon VSM-4001 strain gage with thermistor revealed a rugged, dependable gage which exhibited unparalleled thermal compensation in our laboratory tests. The advantage of vibrating wire technology over the traditional resistance strain gage is in the use of a frequency, rather than a voltage, output from the sensor. This factor, coupled with rugged steel design and hermetically sealed construction, results in excellent long-term zero stability.

For all kinds of gages, installation plays an important role in accurate and reliable data collection. During the lab studies, it was seen that improper installation can cause errors resulting in no data reading at all as well as misreading that can lead the researcher to erroneous conclusions. Several different installation techniques ranging from industrial magnets to various epoxy applications were evaluated for outdoor testing. The ones selected for the next phase of the project were spot welding for foil strain gages and 3M Scotch-Weld epoxy for the VSM-4001 gages.

3. PHASE 2: IMPLEMENTATION OF THE MONITORING SYSTEM

In order to be able to develop a reliable bridge monitoring technique, one should first select a bridge type which would represent large populations such as continuous steel-stringer bridges. The Cross-County Highway Bridge over Reading Road was selected as a typical sample as it has been extensively instrumented, monitored and tested by a multitude of global and local nondestructive techniques in the past. The nondestructive tests conducted for structural identification included modal tests by impact as well as vertical and lateral forced excitation, followed by truck load tests for measuring global and local bridge responses under static loading patterns. The results of these experiments helped to improve an understanding of bridge behavior at the service limit states. A complete and accurate characterization of the as-is structural condition has been related to the structural capacities and structural reliability. Only with this level of understanding can it be possible to determine exactly how many and what types of instruments are needed as a minimum, and where exactly they should be located for the specific bridge type, so that we may confidently interpret the measured responses and have sufficient advance warning for critical changes in the state of an instrumented bridge. Following this type of research, a low-cost instrumented monitoring strategy may be developed and implemented in a large number of bridges.

The two main characteristics of the bridge that need to be captured are the longterm behavior of the bridge under environmental changes and the shortterm behavior of it under traffic loading. In order to capture these behaviors, locations for the baseline sensor set were determined by careful investigation. A finite element analysis was utilized in order to get the optimum sensor locations on the bridge. The general-purpose, FE analysis program SAP90 was employed to analyze the model for sensitivity studies. To find the maximum responses, a moving unit load is used to obtain influence lines of the two middle girders which are considered as critical. The influence lines for several locations are compared. The same kind of analysis is also conducted for the other half of the bridge and the results are found to be compatible with the previous ones. It is concluded that instrumenting only half of the bridge would be sufficient due to symmetrical bridge design with a slight skew. This simplifies installation and reduces cost. The optimum sensor location sections were selected as the abutment, midpoint of the quarterspan, over the pier and midpoint of the midspan. Because the truck traffic is expected most on girder 2, the instrumentation is concentrated on that girder. The other girders are instrumented for control measurements. A finalized first round instrumentation plan can be seen in Fig. 2. The possible deterioration and damage locations were also taken into consideration during design. Because of the limitations of the data acquisition system, the sensor set was limited to 64 gages. The selections of these gages were based on the lab studies done in UCII and former bridge tests done by UCII researchers.

The baseline set is composed of two different gage types: the Geokon VSM-4001 vibrating wire strain gages with thermistor, and the Tokyo Sokki Kenkyujo AWC-8 weldable foil strain gages. The vibrating wire gages are used to capture any strain accumulations due to overall temperature changes, soil pressure changes at the abutments, and differential temperature changes across the width of the bridge. Ten locations have been determined for the placement of the vibrating wires, each having two gages, one at the top flange and one at the bottom flange. They are placed on the outermost girders at the abutments where the maximum responses are expected, as well as on girder two and girder three. The weldable foil strain gages are used to monitor the real-time behavior of the bridge under traffic loads. The data sampling rate of these gages in conjunction with the data acquisition system is sufficient to do real-time monitoring (roughly one kHz). Nine locations with twenty six gages were determined for foil gages. A weather station was also implemented including wind speed, wind direction, temperature, and humidity sensors. It is used to correlate ambient or climatic conditions with readings of the other various gages. Other gages such as tilt meters and DC-LVDTs are planned for future installation.

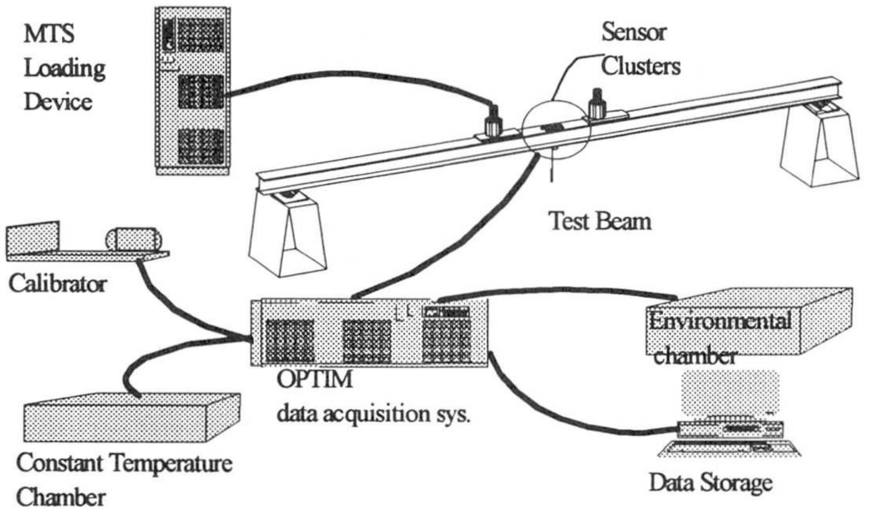

Fig. 1 Schematics of the measurement system scaling for developing a bridge monitoring system.

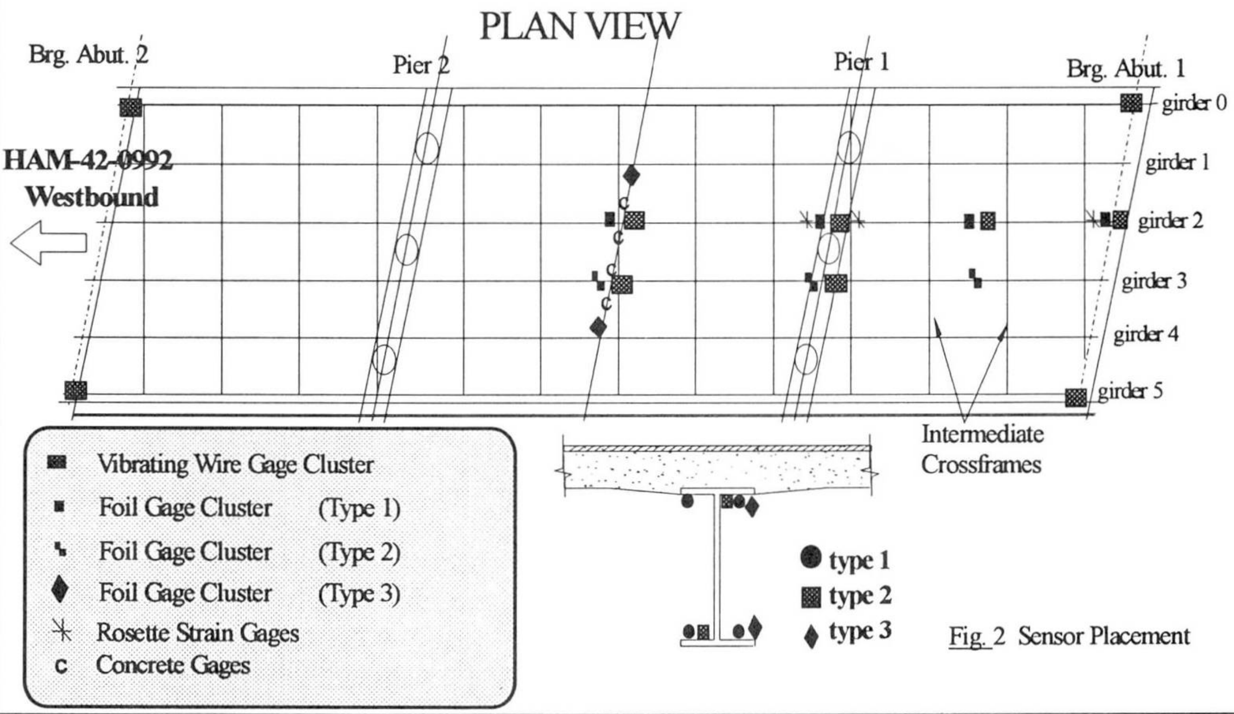

Fig. 2 Sensor Placement

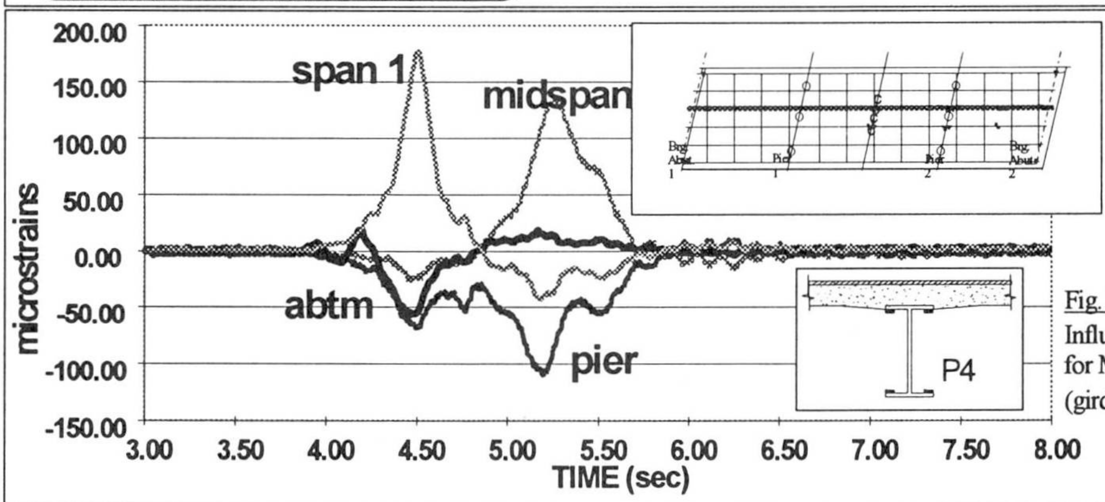

Fig. 3 Real-Time Influence Lines for Moving Truck (girder 2 @ P4)

One may consider the following scenarios for instrumenting a bridge for measuring global and/or local responses: *(a) Instrumenting an existing bridge for intermittently measuring any changes which may occur in the global geometry for health monitoring; (b) Instrumenting an existing bridge for measuring incremental global and/or local responses under static or dynamic loads over a shortterm (in the order of seconds to hours),* such as in weigh-in-motion or in diagnostic testing for bridge rating; *(c) Instrumenting an existing bridge for measuring incremental global and/or local responses under static or dynamic loads over longterm (in the order of months to years); and (d) Instrumenting a bridge through fabrication and construction for measuring the absolute local and global responses over a longterm (months to years).* The first three kinds of testing are being performed on Cross County Highway Bridge over Reading Road (HAM-42-0992).

A "Static Loading Test" is used for measuring responses due to controlled loading and for calibration of the finite element and section analysis models. The load distribution in both the longitudinal and transverse directions are immediately apparent from the field results. The location of the neutral axis is monitored during the entire loading sequence and is useful for determining the degree of composite action that the structure exhibits. The measured strains indicated a nearly fully-composite action, even though the bridge was designed as a non-composite bridge, since the top flange of the girder was embedded into the deck. Since the 3-D FE model of the bridge had been calibrated to closely simulate all the critical response mechanisms, it was used for analytical comparison with the experimental strains of the girders. "High-Speed Dynamic Test" is used for measuring bridge responses due to moving traffic and conducting an on-line bridge rating. A section capacity analysis was performed considering different limit states. Transform equations were written to calculate the rating factors based on the strain readings obtained from the monitor. These calculated factors can allow an instant appreciation of the bridge's health. A "Long-Term Environmental Test" is used for longterm monitoring of thermal and environmental effects on the bridge. Strain and temperature readings at all girder locations are monitored continuously at one-second/gage intervals for capturing their distributions across the bridge with ambient temperature changes. Successful monitoring over three months (Nov. 94-Jan 95) has been achieved. The strains captured by the monitor indicate significant stress cumulation (~14 MPa.) close to abutment due to a ambient temperature change of 20^0C.

The fourth test *Instrumenting a bridge through fabrication and construction for measuring the responses over a longterm* will be pursued in the third phase of the research. This would be the greatest challenge in instrumented monitoring, as there has not been any previous effort to instrument and monitor the responses of a stringer bridge from construction through service and with the objective to synthesize the absolute state-of-stress.

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Monitoring of Bridge Conditions for Durability Evaluation in Poland
Inventaire de l'état des ponts en Pologne et estimation de leur durabilité
Überwachung des Brückenzustandes zur Bewertung der Dauerhaftigkeit

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SUMMARY

This paper describes how the durability of bridges is taken into account in the Polish Bridge Management System on the basis of data acquired by systematically monitoring the technical condition of over 18'000 bridge structures through annual inspections . The construction of an optimum model for the Polish conditions of degradation will allow to determine the real life for each of the bridges in service. This is also important for long-term planning, because it creates the basis for optimising the allocation of funds for the maintenance and rehabilitation of these bridges.

RÉSUMÉ

L'article décrit comment la durabilité des ponts est prise en compte dans les normes polo-naises sur la base de données acquises chaque année par surveillance et évaluation systématiques de l'état des structures de plus de 18'000 ponts en Pologne. La construc-tion d'un modèle optimal pour les conditions de dégradation en Pologne permettra de déterminer la vie réelle de chacun des ponts en service. Cela a également de l'impor-tance pour la planification à long terme, car il créera une base pour l'optimisation des fonds alloués à la maintenance et à la réhabilitations des ponts.

ZUSAMMENFASSUNG

Dieser Vortrag beschreibt wie im polnischen Brückenwirtschaftssystem auf der Grundlage von Daten aus systematischer Überwachung ihres technischen Zustandes bei den jähr-lichen Revisionen von über 18'000 Konstruktionen die Dauerhaftigkeit berücksichtigt wird. Der Aufbau eines für polnische Verhältnisse optimalen Degradationsmodells wird die Be-stimmung der tatsächlichen Lebensdauer jeder der betriebenen Brücken erlauben. Dies ist auch wesentlich angesichts der langfristigen Prognostizierung ihres technischen Zu-standes, da es die Grundlage zur Optimierung der für die Erhaltung und Modernisierung dieser Brücken im Haushalt vorgesehenen Mittelaufteilung darstellt.

1. INTRODUCTION

An extensive knowledge on the technical and service condition of the bridges, bearing on the estimation of their real life, is of essential importance for the process of the management of bridges. The term real life commonly refers to the serviceability of a bridge structure which agrees with the requirements of the design. The knowledge on this subject forms the basis for the optimization of bridge work which in Poland is closely connected with a computer system that aids the management of bridges, further referred to as the Bridge Management System - BMS.

This system is made up of the main Inventory module (EGM) and some programs which by collaborating with one another carry out the selected reporting and service options of the planning function. The Polish BMS has some procedures which make the system of bridge inspection more efficient. By means of the functioning KPP (Basic Inspection Data Form Editor) program all damage to bridge components, including its frequency, can be described and their technical condition can be assessed [1, 2].

At the moment, the Polish BMS monitors the following parameters which are taken into consideration when the durability of bridges is estimated: the technical condition of the bridges, the amount and the structure of the traffic, the frequency and the quality of the maintenance work, and the effect of the environment. In this paper, the authors would like to focus on the estimation of the technical condition of bridges combined with the recording of damage, which will constitute the basis for the evaluation of durability in the Polish BMS.

2. DIVISION OF BRIDGES INTO COMPONENTS FOR MONITORING THEIR TECHNICAL CONDITION IN POLISH BMS

The developed and implemented version of the KPP program, which describes in computer terms the basic inspection, divides each of the monitored structures of the bridge into eleven basic components that occur in each case (see Fig. 1) that are complemented, depending on the need, by eight additional components (piers and their foundation, river-bed, foreign equipment, mechanisms of span movement, articulated joints, retaining walls, pylons, external tension members).

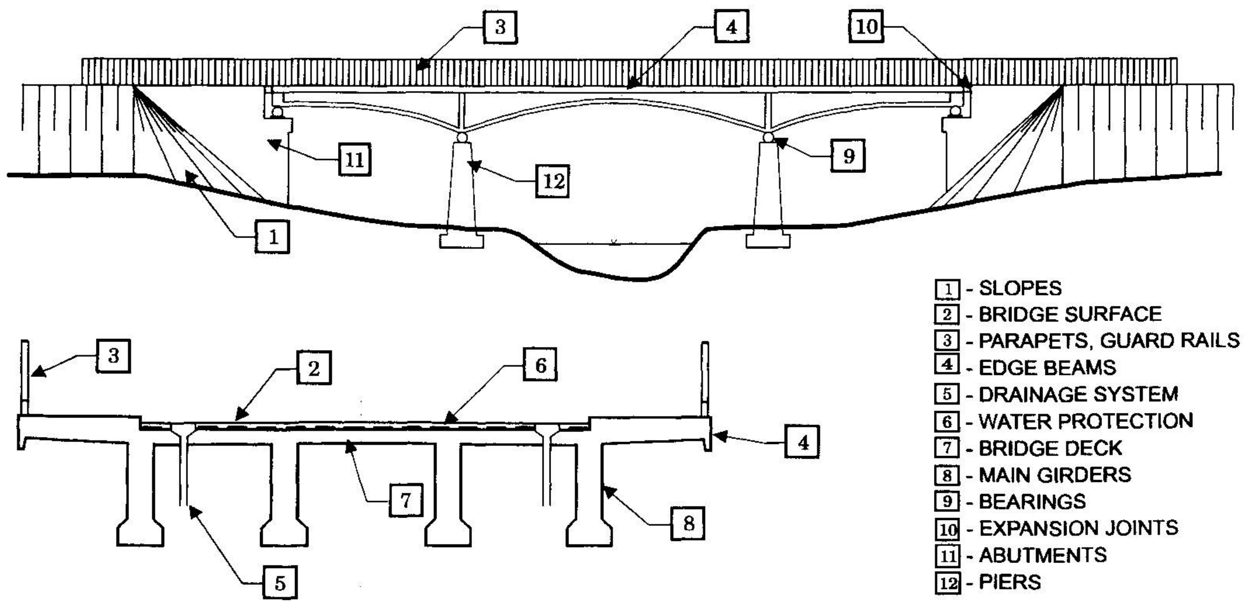

Fig. 1 The bridge components for which damage was monitored

Considering the size and the typical character of the bridge structures in Poland, one can say that in most cases, the technical condition is monitored for 12 bridge components (11 basic ones plus piers) which perform specifically defined functions when the bridge is in service.

In the years 1990-1994, the authors of this paper together with their team carried out inspection, within the framework of the work on the CEDOM (Central Inventory of Great Bridges) program, of over 300 bridges situated on bigger rivers in Poland, in which, among other things, the damage to the particular components of the bridges was monitored. In our considerations, we have limited ourselves to the analysis of the results for 100 selected types of bridge structures situated on the Warta, the Pilica and the San and on the southern Polish border.

No.	OBSTACLE (river)	NUMBER of BRIDGES			TOTAL
		CONCRETE	STEEL	STEEL WITH TIMBER DECK	
1	THE SAN	4	13	3	20
2	THE WARTA	34	17	4	55
3	THE PILICA	16	2	1	19
4	S. BORDER	3	3	0	6
	TOTAL	57	35	8	100

Table 1 The number of bridges for which the rates register was kept (bridges divided according to the span structure material)

A sketch of the typical bridge structure with the superimposed division into the 12 components that the KPP program calls for, for which the damage and the technical condition was monitored in accordance with the principles of conducting the basic inspection being in force in Poland, is shown in Fig. 1.

One hundred selected types of the bridge structures registered in the CEDOM program, for which rates were recorded have been compiled in Table 1.

3. MONITORING OF CONDITION OF BRIDGES IN POLISH BMS

The monitoring of the technical condition of bridges is one of the primary tasks of the basic inspection. According to the regulations that apply, basic inspections are conducted once a year by trained bridge inspectors in all the 171 Road Management Units on the whole territory of Poland. Over 18 thousand of bridge structures of different types and with different static schemes are inspected regularly. During the inspection all the damage to the structure sustained in service is monitored and the current technical condition of the particular bridge components are evaluated. The data are entered directly from the field into the KPP program installed on the bridge inspector's notebook.

Fig. 2 The condition rating scale in Polish BMS

To standardize the description of different kinds of damage a catalog of damage of bridge components [3] was worked out for the further processing of the information. Each of the detected damage is described by a code consisting of two letters which stand for the kind of damage and the material. Each of the evaluated components is rated using a six-grade scale from 0 to 5. The adopted rating scale is presented graphically in Fig. 2.

The results obtained from basic inspections are stored in BMS in the base of rates which is serviced by the KPP program. The overall rate for each bridge is computed as the lowest from the mean rates of all the components, the girders, the deck and the supports [4].

The mean rates of the technical condition of the particular bride components for the monitored group of 100 bridges have been compiled in Fig. 3. A comparative analysis shows that expansion joints are rated decidedly lowes but these elements traditionally degrade fastest.

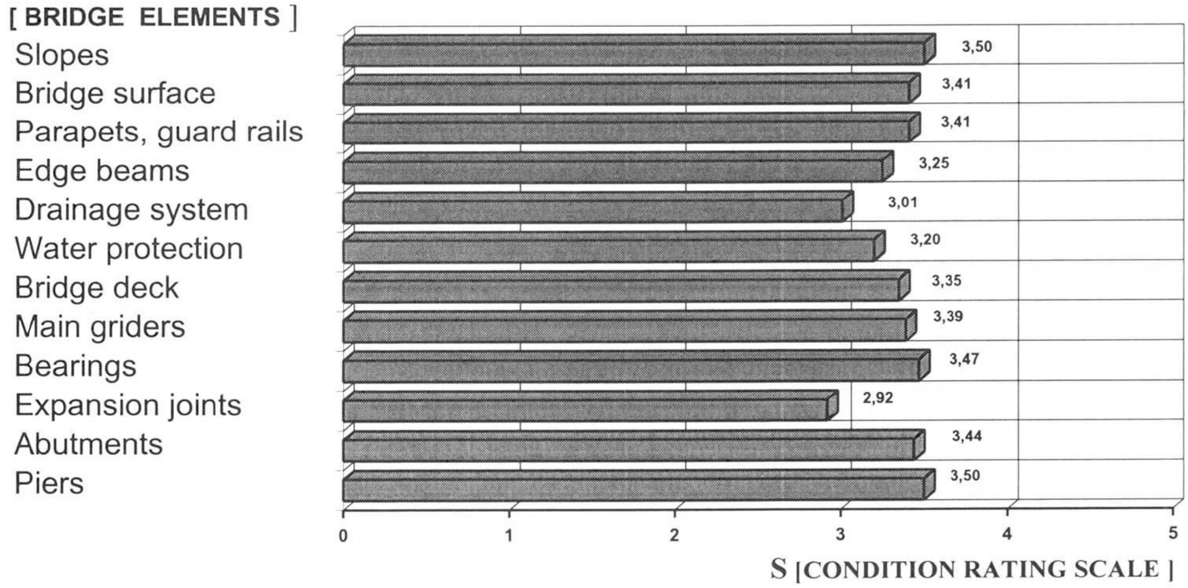

Fig. 3 Mean rates of the technical condition for the particular bridge components

4. HOW POLISH BMS EVALUATES DURABILITY

During the work on the Polish BMS it was assumed that the real life of bridges was affected by factors associated with the technical condition of the bridge structure itself, its service and the immediate environment. The total effect of all these factors on the durability of bridges can be estimated through their continual monitoring which provides the basis for the development of an appropriate model of degradation. It has also been taken into consideration that the future model of degradation should be maximally effective in the Polish bridge-work conditions.

This problem can be solved in two ways: through the theory (models of degradation) and through the practice (monitoring conducted for many years). At the moment the work aimed at this is at the conception stage.

4.1. Theoretical Model

Solutions based on the theory yield bridge structure degradation curves in the form of a function which, however, must be modified so that it would meet the practical requirements (empirical correction coefficients). In Poland, attempts are being made to derive theoretical relationships in order to construct degradation curves through computer analyses of different types of structural solutions taking into consideration the division of bridges into components. Bridge structures that include components that, for example, have not been repaired, been repaired, are damaged, were badly executed or have structural defects have been subjected to this analysis.

Various service parameters that characterize a particular bridge structure are associated with the real life of each of the analyzed components. The overall effect of these parameters resulting in a change of the technical condition of the bridge is represented by the level of its wear based on the relationship between the service life till now and the assumed durability of the bridge. The following coefficients that affect the level of wear of a bridge most strongly have been selected on the basis of the analyses: **a material coefficient**, **a fatigue coefficient**, **a load coefficient**, **a maintenance coefficient**, **an environment impact coefficient** and **a construction period coefficient**. [5].

The level of wear of a bridge can be described by this formula:

$$p = \frac{W x\, t_e}{M x\, T} \qquad (1)$$

where: W - structural-service coefficients, t_e - the service life till now,
T - the assumed service life of the bridge, M - a material coefficient.

At the moment the individual coefficients are being calibrated on the basis of the input data obtained from the inspections of the bridges and this work is in its final stage. Such calculations have been done for all types of bridges.

4. 2. Practical Model

One can approach the problem of a model of degradation in a practical way using as the basis the data obtained from monitoring for many years the technical condition of bridges. The input data for the plotting of degradation curves are obtained from systematically conducted basic inspections. Naturally, the way in which the bridges are evaluated affects the shape of the degradation curve, where the measure of degradation are the rates the bridges received during inspections. Attempts are being made to plot degradation curves for the different types of bridges on the basis of the collected data. Further research in this area will allow us to obtain curves similar in their shape to the theoretical ones [6].

COMPONENT	NUMBER OF BRIDGES WITHIN RANGE OF RATES S		MEAN RATE S_m
	S≥3.0	S<3.0	
MEAN FOR ALL COMPONENTS	79	21	3.33
BRIDGE DECK	90	10	3.34
MAIN GIRDERS	93	7	3.38
SUPPORTS	89	11	3.39
MEAN RATE OF BRIDGES			3.04

Table 2 Compiled results of the technical condition monitoring for the group of 100 bridges

The results of the monitoring of the technical condition for the selected group of 100 bridges registered in the CEDOM program have been compiled in Table 2. On this basis, sample degradation curves have been plotted by the linear regression method and the curvilinear regression method.

5. SAMPLE DEGRADATION CURVES PLOTTED FROM DATA GATHERED IN BASE OF RATES

Having at their disposal the data collected during the work on the computer data base for the CEDOM program and using the derived theoretical relationships, the authors of this paper made an attempt to plot sample degradation curves for the selected group of bridges. The obtained degradation curves should be treated with caution since they pertain to a peculiar group of bridges whose transportation importance may lead to a quite different mechanism of degradation than in the case of small or less used bridges.

A sample diagram describing changes in the technical condition of the selected group of bridges as the function of time, obtained by the linear regression method is presented in Fig. 4. The visible distortion of the diagram was caused by the relatively low rates which some advanced in their age bridges received.

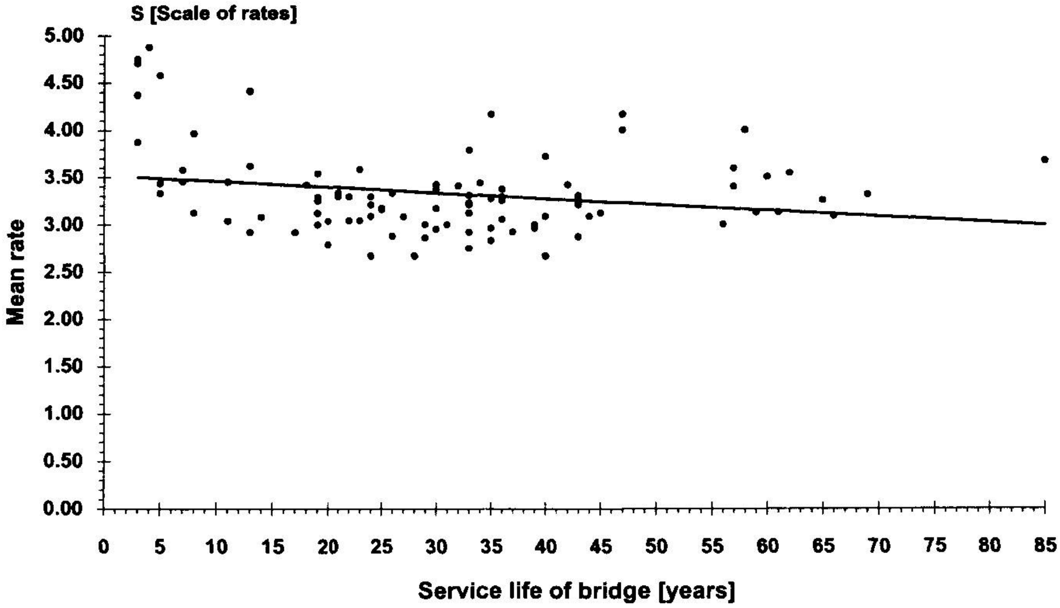

Fig. 4 A sample degradation curve for the selected group of bridges (including repaired bridge structures during their service)

6. RECAPITULATION

Models of the degradation of bridges allow one to plan long-term their technical condition which facilitates the optimization of the allocation of budget funds for their maintenance. This is why one finds so many solutions when systems that aid the management of bridges in different countries in the world are analyzed. In Poland, the foundation on which an optimum model of degradation is being built is the conducted since 1991 systematic monitoring of the condition of over 18 thousand of bridge structures of various types.

The methods of constructing degradation models presented in this paper should be treated as conceptual solutions since they are based on an analysis of technical condition rates for a selected group of 100 bridges. When these solutions are verified using the full data base, then one will be able to estimate more realistically the influence of the technical condition on the durability of structures of various types. These procedures will be included in the Polish BMS in the nearest future, which will make it possible to conduct rational policy in the domain of bridges maintenance and will result in the extension of the life of each of the bridges in service.

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Development of a Measuring System for Long-Term Structures' Displacements

Développement d'un système de mesure pour les déplacements structuraux à long terme

Entwicklung eines Mess-Systems für Langzeit-Bauwerksverschiebungen

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SUMMARY

Displacement measuring system utilising a laser beam for large scale concrete structures has been developed. To apply this system for long-term monitoring, several improvements have been made. Based on the measured displacements of existing concrete structures, the long term behaviour of these concrete structures can be explained.

RÉSUMÉ

Un système de mesure des déplacements utilisant les rayons laser pour les grandes structures de béton a été développé récemment. Plusieurs améliorations ont été réalisées pour une application de ce système en vue d'une observation à long terme. Sur cette base, le comportement à long terme des structures en béton peut être expliqué.

ZUSAMMENFASSUNG

Ein mit Laserstrahlen arbeitendes Mess-System für Verschiebungen grosser Betonkonstruktionen wurde neu entwickelt. Um dieses System auch für Langzeitbeobachtungen nutzen zu können, wurden viele Verbesserungen gemacht. Basierend auf den gemessenen Verschiebungen von bestehenden Betonkonstruktionen kann das Langzeitverhalten dieser Betonkonstruktionen erklärt werden.

1. INTRODUCTION

The essential technique to monitor the varying states of existing large scale structures is obviously to measure the displacement of various points of a structure under the consideration to a degree which may be sufficient enough to judge its condition by the comparison either with the other similar structures or with the analytically estimated values. However, it is also too obvious that the long term observation of the displacement of the points in large structures is quite expensive and out of budgeting. It is rarely carried out in the present engineering except for a special case such as the seriousness of structure is to such an extent that an authority permits to allocate budget for it.

Therefore, it is desirable to develop a system which enables to measure the displacement of large structures with feasible cost for man power and for equipment. The present paper reports the successful development of a measuring system for long term displacement of a structure by the utilization of the laser beam and the opt sensitive displacement sensor attached to a structure. In developing the system, the most critical difficulty was the accurate resetting of laser launcher with same laser direction at the fixed launching location and accurate resetting of the opt sensitive displacement sensor in the same location of a structure.

2. FUNDAMENTAL CONCEPT OF THE SYSTEM

The concise and less laborious measurement of the displacement of a large scale structure requires the system of non contact type measuring system. What we have adopted is the utilization of the laser beam and the opt sensitive sensor (PSD sensor) which can measure the movement of laser focus with the resolution accuracy of 0.05 mm in the x and the y directions of which picture is shown in Fig.1. By combining two of these sensors, we can detect the movement of the point displacement in the three dimensional space statically as well as dynamically. The laser should be launched to the fixed direction every time during the measurement period of a year or even of 10 years for this specific purpose. The laser launcher and opt sensitive sensor should be removed after each measuring because those precision apparatus cannot be kept outside for a long period. Therefore in resetting of laser launcher we need accurate direction identifying apparatus. For this purpose, we adopted the theodolite. Its angle resolution capacity is 0.65 second in horizontal angle, and 1.0 second in vertical angle which means that the laser can be launched in the same direction within the maximum error of 0.315 mm at the distance of 100 m horizontally and 0.485 mm vertically, respectively. It can be said as the world most accurate angle identifying theodolite at present of which picture is shown in Fig.2.

However, besides this error we have several other errors which should be minimized. The resetting error of the launchers and sensors are the biggest factors in this sense. To minimize the error of resetting the launcher, the launcher post to which the launcher can be reset with 1×10^{-4} mm error was invented and similarly to minimize the resetting error of PSD sensor, the PSD sensor table which can reincorporate the sensor in the same location with the accuracy of 1×10^{-4} mm was also invented.

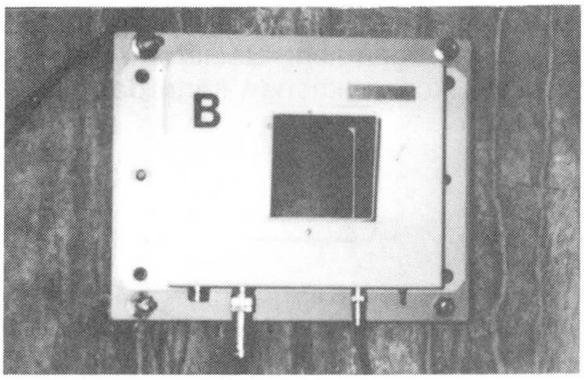


Fig.1 Opt sensitive sensor

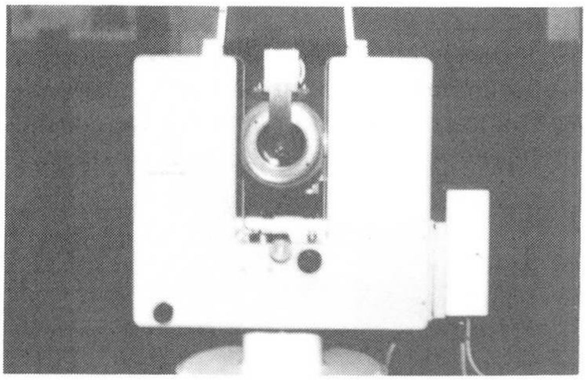


Fig.2 Precise theodolite

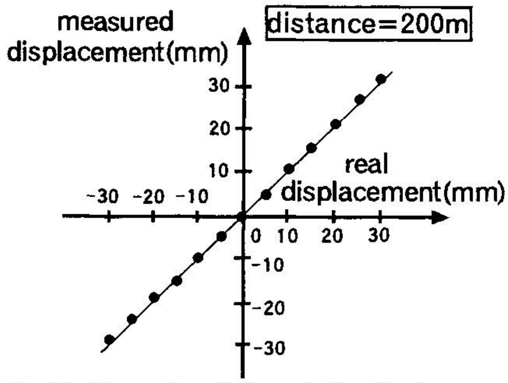


Fig.3 Linearity of the static displacement

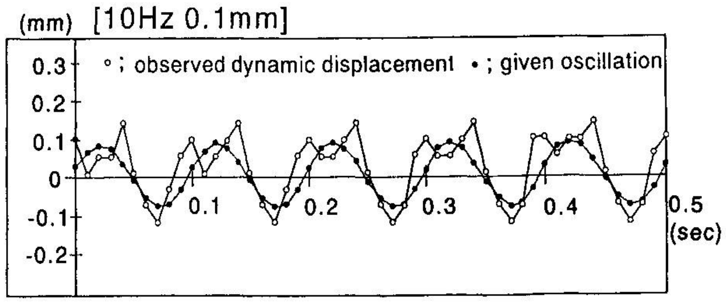


Fig.4 Linearity of the dynamic displacement

With all those precision apparatus, our strategy for the evaluation of the soundness of a structure is that during the life of a structure the measurement of the relative location in terms of the fixed three dimensional space which mark points are set with three unmovable points should be done. After certain elapse of time, the measurement has to be made. The time elapse depends on the maintenance circumstances. The measurement has to be continued as long as several years or more. Soundness judgment will be done with these observed data together with the analysis.

3. RELIABILITY OF THE DEVELOPED SYSTEM

Before discussing actual examples of application, the reliability of the system at present will be shown.

3.1 Linearity of the Static Displacement

The real displacement and measured displacement from the distance of 200 m are compared in Fig.3. It may be noted that good linearity is observed in both cases.

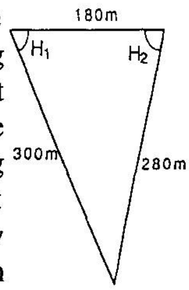
3.2 Linearity of the Dynamic Displacement

In Fig.4, the observed dynamic displacement of about 0.1 mm of about 10 Hz is shown in comparison with the given oscillation. Though there exists regularized difference, its accuracy is good enough for our purpose and more over it is satisfactory that the system has shown to be able to catch up the 10 Hz frequency displacement with this accuracy.

3.3 Reliability of resetting of the system

To identify the resetting accuracy, the three dimensional space coordinate identification was tried. After initial setting of the two launcher posts at about 180 m away, another fixed point is made about 300 m away from both the launcher posts making a triangle bench marks in the three dimensional space. Then, the horizontal angles of two corners were measured and apparatus were removed. After three weeks elapse, the resetting is done and the same two angles are measured.

In Table 1, the initial readings of the angles together with the following readings after resetting are shown. It may be noted that the agreement is quite satisfactory and the maximum resetting error is only 3 seconds which means that we can launch the laser with the accuracy at least of 1.5mm at the distance of 100 m even with the situation of resetting.



Standard value: H_1=64° 58′ 55″			Standard value: H_2=78° 48′ 27″		
Date	Measured Angle	Difference with H_1	Date	Measured Angle	Difference with H_2
94/10/7	64° 58′ 52″	-3″	94/10/6	78° 48′ 28″	+1″
〃	64° 58′ 56″	+1″	94/10/27	78° 48′ 29″	+2″
〃	64° 58′ 57″	+2″	〃	78° 48′ 28″	+1″
〃	64° 58′ 57″	+2″	〃	78° 48′ 30″	+3″
94/10/27	64° 58′ 55″	±0″	94/10/28	78° 48′ 30″	+3″

Table 1 Measured horizontal angles after resetting

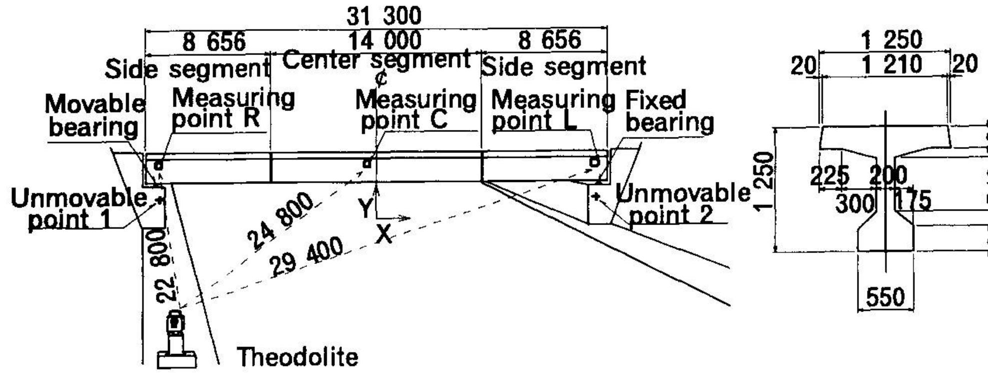


Fig. 5 General view of the bridge and the measuring points

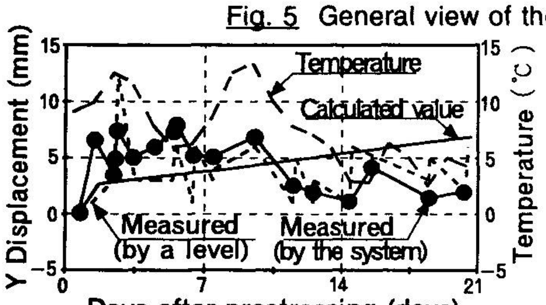


Fig.6 Measured and calculated vertical displacements of the point C

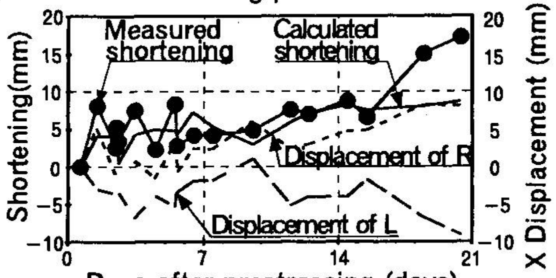


Fig.7 Shortening of the girder and measured X displacements of the points L and R

4. EXAMPLES OF APPLICATION

4.1 Prestressed Concrete Precast Segmental Girder Bridge

The displacement of existing prestressed concrete precast segmental girder bridge was measured. The bridge has seven main girders and each girder consists of three precast segments. The measurement was carried out for one of the girders from Nov. 10 when the prestress was introduced into the girder until Nov. 30, 1994. During the measurement, each girder is considred to deform independently. The general view of the bridge and the measuring points are shown in Fig.5.

The results of the measurements are shown in Figs.6 and 7. In these figures, the calculated values are obtained considering the influence of erection load, creep of concrete (due to prestressing force and dead load), shrinkage of concrete and loss of prestressing (due to creep, shrinkage and relaxation of prestressing steel). These influences are calculated numerically according to "Standard Specification for Design and Construction of Concrete Structures 1991" of JSCE. In addition, the calculated value in Fig.7 contains the deformation in the horizontal direction due to the temperature. In Figs.6 and 7, the indented shape of measured values is due to the daily temperature change.

In Fig.6, qualitatively similar behavior can be seen for the measured vertical displacement of the point C and for the calculated one. The displacement measured by a level indicates that the displacement measured by this system is reliable. It seems that the difference between measured and calculated values results from the loss of prestress in ungrouted tendons due to the sudden decrease in temperature and the difference in temperature between the upper flange and the web. In Fig.7, the measured shortening of the girder is presented. From Fig.7, the horizontal displacement of the girder can occur not only at movable bearing but also at fixed bearing when a rubber bearing is provided.

4.2 Prestressed Concrete Hollow Girder

The deflection due to daily temperature change, dead load and live load was measured for prestressed concrete hollow girder constructed in Narita Airport. The girder is 1.5 m high, 8.2 m wide and 35 m long. The cross section is shown in Fig.8. Fig.9 illustrates the measuring system in which one theodolite, two sensors and three unmovable points are used. The theodolite is installed about 90 m distant from the girder. The deflection of the bridge is obtained by the difference between the vertical displacement of the sensor A attached at the center and that of B at the support. Two unmovable points are allocated on abutments on both sides of the bridge and third one on another pier. Thermo-couples are embedded in a side surface of the girder at three levels as 5, 50, and 90 cm from the lower surface of the girder to measure the temperature gradient.

Fig.10 shows daily changes of vertical temperature distributions in the bridge from '94 September 2 22:00 as well as long-term one from '93 August 5 14:00 which gives the curvature of the bridge. Fig.11 indicates the change of deflection measured by the system compared with that obtained by the curvature. A positive value in the coordinate is warping and negative one is curling. Measured deflection shows rough agreement with calculated one up to September 3 14:00 and then these deflections show the difference. Although the temperature is measured only at a side surface of the girder and is not measured at upper portion as well as inner portion, the curvature of the bridge is considered to depend on the temperature distribution of whole cross section.

The change of deflection with time was also measured and that was compared with the result computed considering the effects of creep and drying shrinkage of concrete, as is indicated in Fig.12. Creep analysis was done based on the principle of superposition in which creep coefficient and drying shrinkage strain were determined according to CEB MODEL CODE 90. It is assumed that the girder was prestressed and loaded by dead weight at the age of 7 days, and then loaded by both deck pavement and handrail at 28 days. According to Fig.12, measured deflection agrees very well with calculated result, which means that the present system enables to measure long-term displacement with satisfactory accuracy. As is shown in Fig.12, the increase in deflection under sustained load is negligible and temperature change marked by black circles in Fig.10 influences strongly the change of deflection. The increase in creep and drying shrinkage during measurement for 4 months is very slight, because measurement was carried out at the age of nearly 2000 days after construction. To measure instantaneous deflection of the bridge, a transportation vehicle with full weight of 21.9 tonf was loaded on the span center in one lane of the bridge. Measured deflection was 2.2 mm and calculated one was 2.4 mm using the beam theory.

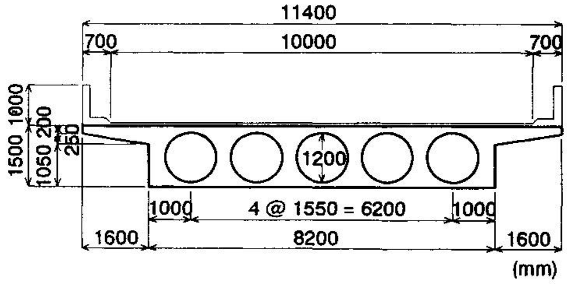


Fig.8 Cross section of the hollow girder

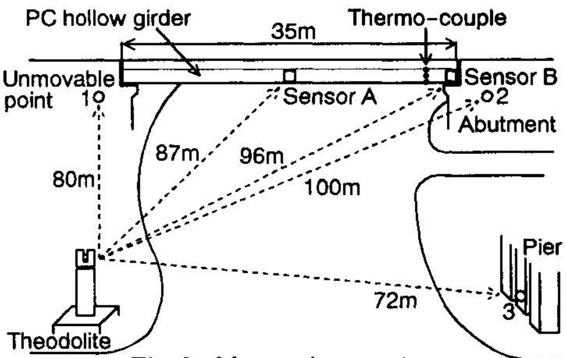


Fig.9 Measuring system

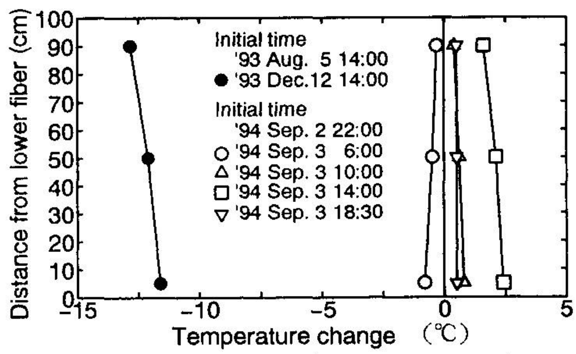


Fig.10 Distribution of temperature change

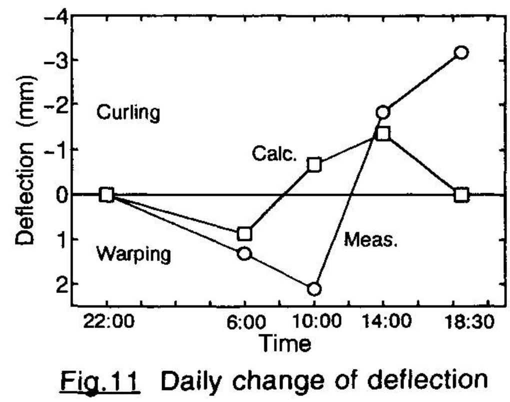


Fig.11 Daily change of deflection

4.3 Concrete Arch Dam

The 3-D displacements of concrete arch dam caused by water-level of the dam lake is measured by this system.

Two sets of theodolite with the launcher of laser beam are installed in the distance of around 150 m away from the dam and two sensors are attached on the dam. Using the output of a pair of sensors, the 3-D displacements of the structure can be calculated theoretically. The concrete arch dam to measure the displacement was built in 1994 which is 100 m high and has 300 m crest length. The measuring point is allocated on the downstream wall of the dam. The outline of measurement is shown in Fig.13.

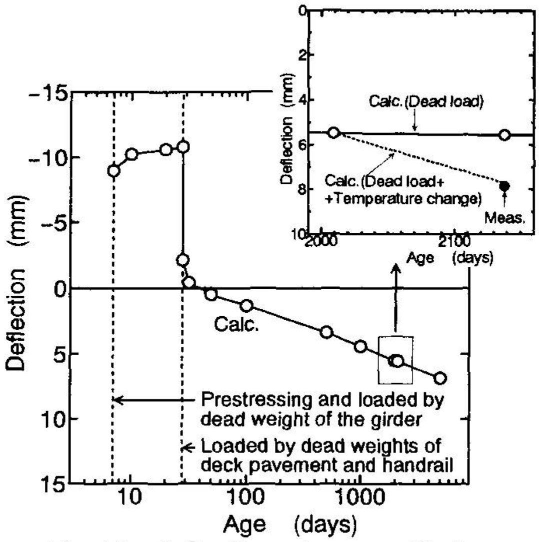


Fig.12 Deflection change with time

The measuring point is 20 m away from the center of the dam to the direction of the right river bank. The coordinate of 3-D displacements is also shown in Fig.13. The measurement was carried out in August and October 1994.

The displacement in the Z direction is shown in Fig.14. The displacement at the beginning of measurement at 9:00 A.M. in August 3 is assumed to be 0. The measured displacement in the Z direction at October 28 increased about 17 mm. This increase of the displacement in the Z direction is considered to depend on the water level of the dam lake, because the water level was around 90 m in August 3, and 75 m in October 28. According to another measuring system built-in the dam, 1 m change of the water level of the dam lake is corresponding to around 1 mm change in the Z displacement. The present measuring system can provide almost same value as the built-in measuring system. Using this system, the displacement of large size concrete structures like this arch dam can be measured accurately from the long distance. As for the measurement of long term behavior, the present system can be applied.

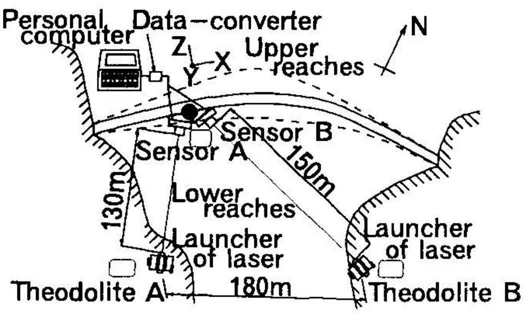


Fig.13 Outline of measurement

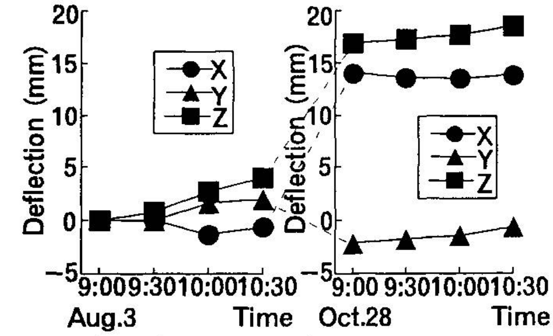


Fig.14 Results of measurement

5. CONCLUSIONS

The newly developed measuring system for long term displacement of large scale concrete structures is outlined. As presented in each application, the behavior of concrete structures can be explained by this system. Obtained information is especially useful for the maintenance of existing structures.

Acknowledgements

This research has been carried out by the co-operation of several universities and organization. The contributions of Prof. Higai of Yamanashi Univ., Dr. Niwa of Nagoya Univ., Dr. Ujike of Utsunomiya Univ., and Mr. Nishio and Mr. Imao of Abe Kogyo-sho are gratefully acknowledged.

Global Monitoring System on Lantau Fixed Crossing in Hong Kong

Système de surveillance intégrale pour la liaison de Lantau à Hong Kong

Integrales Ueberwachungssystem für die Lantau-Brückenverbindung in Hongkong

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Professor
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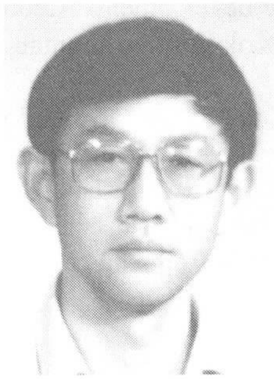

Xila Liu, born 1940, received his Ph.D. at Purdue University, USA. He is the vice Chairman of Academic Committee of China Civil Engineering Society, and Head of the Department of Civil Engineering at Tsinghua University, Beijing.

SUMMARY

The Lantau Fixed Crossing project includes three cable-supported bridges and an airport railway, expressway, viaducts and some road works. It will form the initial access to the proposed new Hong Kong port and new airport developments on Lantau. The paper introduces the preliminary design of the monitoring system on this project. The basic architecture consists of five parts: Sensor System, Data Transmission System, Traffic Control System, Health Monitoring System, and Decision Making System. Of the five the Traffic Control and the Health Monitoring Systems are introduced with some details.

RÉSUMÉ

Le projet de liaison de Lantau comporte trois ponts haubanés, un métro d'aéroport, une autoroute, des constructions en saut-de-mouton et diverses routes. Il doit former le premier accès aux nouveaux terrains projetés pour l'aménagement du port et de l'aéroport de Lantau à Hong-Kong. Cette communication présente l'organisation du système de surveillance comportant cinq parties: système de détection, système de transmission de données, système de contrôle du trafic, système de surveillance de l'état des structures et système de prise de décision. L'article décrit plus en détail ces deux derniers systèmes.

ZUSAMMENFASSUNG

Das Lantauprojekt umfasst drei Schrägseilbrücken mit einer Flughafen-Metro, Autobahn, Ueberwerfungsbauten und etlichen Strassen. Es wird den primären Zugang zum geplanten neuen Hafen- und Flughafengelände Hongkongs auf Lantau bilden. Der Beitrag führt in den Entwurf zum Ueberwachungssystems ein, das aus fünf Elementen besteht: Sensorsystem, Datenübertragung, Verkehrsleitsystem, Bauwerkzustandsüberwachung und Steuereinheit zur Entscheidungsfindung. Von diesen fünf werden das Verkehrsleitsystem und die Bauwerkzustandsüberwachung näher vorgestellt.

1. INTRODUCTION

Hong Kong is situated at the estuary of the Pearl River Delta of China and is centrally located at the Asia-Pacific Rim, one of the fastest growing regions in the world. In the recent years, there has been rapid development in Guangdong Province, especially around the Pearl River Delta area. As a result, Hong Kong will become one of the busiest ports in the world very soon. In order to enable Hong Kong to play more important role as a center of world trade, Hong Kong has embarked on a program of infrastructure development. A core project of this program is to build a new international airport at Chek Lap Kok, north of Lantau Island.

Placing the new airport at Chek Lap Kok requires to build a number of long-span bridges and high capacity roads to serve the airport. The transport links from the new airport on Lantau to the urban centers are in the form of a six to eight-lane expressway and a high-speed railway covering a distance of about 34 km to Hong Kong Island. Journey time for passengers using the airport railway will be less than 25 minutes from the central district of Hong Kong to the new airport.

The Lantau Fixed Crossing will link Tsing Yi and Lantau island via the island of Ma Wan. The route comprises two major bridges, namely the Tsing Ma suspension bridge with a central span of 1377 m main span (Fig.1) and the Kap Shui Mun cable-stayed bridge with a central span of 430 m (Fig.2). Both bridges have double decks. The Crossing will also link the border of Shenzhen with Tsing Yi by a triple tower cable-stayed bridge, called Ting Kau Bridge designed with spans of 475 m and 448 m respectively (Fig.3). It should be mentioned that when compared with similar existing bridges the Tsing Ma bridge would be the second longest span bridge in the world and will be the longest span bridge carrying both load and rail traffic on the same structure when completed. It is obvious the bridge is designed to withstand very heavy traffic and the most severe winds that could occur during its servicelife.

The present paper will briefly introduce the preliminary design on a global monitoring system on Lantau Fixed Crossing in general and on the Tsing Ma Bridge in particular. The basic architecture consists of five parts: Sensor System, Data Transmission System, Traffic Control System, Health Monitoring System, and Decision Making System. Of the five parts the Traffic Control System and the Health Monitoring System are introduced with some details.

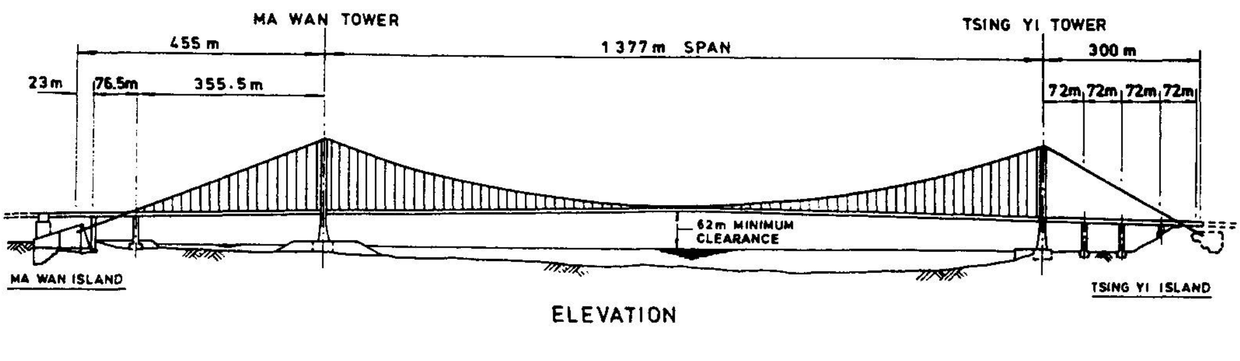

Fig.1 Tsing Ma Bridge

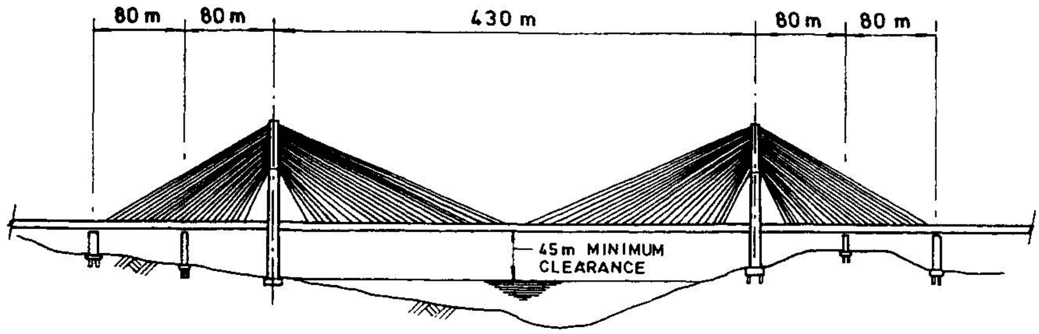

Fig.2 **Kap Shui Mun Bridge**

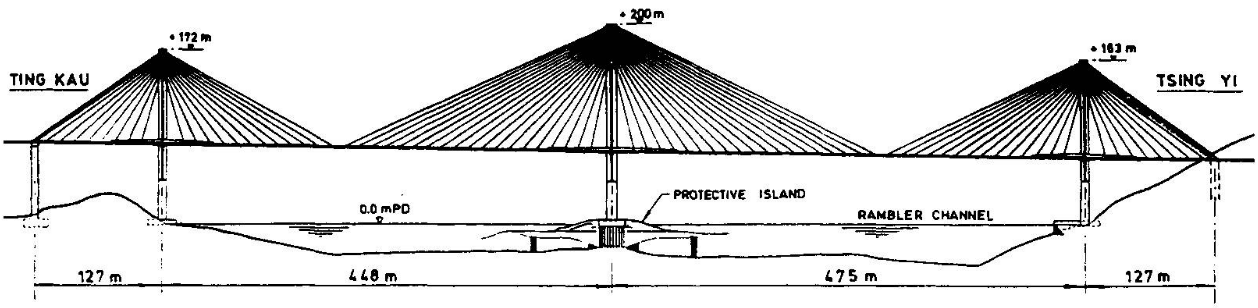

Fig.3 **Ting Kau Bridge**

2. PRELIMINARY ARCHITECTURAL DESIGN OF THE SYSTEM

Since the mentioned three bridges are the lifelines of the new airport, their service condition will ultimately affect Hong Kong economy. Thus, it is important to install a reliable monitoring system which can insure the long-term safety of these bridges. The proposed system will serve two major functions: (1) a warning system for partial or full traffic closure when the vibration of any of the three bridges reaches a critical state, and (2) a system continuously monitoring the structural health of the bridges for maintenance or repair work. Particular emphasis will be placed on (1) the dynamic responses of bridges under severe wind conditions or combination of wind and traffic loading, and (2) identification of bridge degradation and locations of deterioration for scheduling of maintenance or repair work.

The idea of installing a monitoring system to measure the dynamic responses of bridges is not new [1][2]. However, the proposed system is an integrated data management system. As shown in Fig.4, its important features are: (1) it is an automatic, remote data gathering system with high reliability, (2) it can provide continuous signal processing, (3) it utilizes a state-of-the-art knowledge-base for decision-making on traffic control and for damage assessment on maintenance work, (4) it is an integrated system in a way that the traffic control is synchronized for all three bridges as a unity.

Since the instrumentation part, which includes the Sensor System and the Data Transmission System, has already been planned by the Highways Department of Hong Kong through contractors, in the present paper, only the Traffic Control System and the Health Monitoring System are introduced as follows.

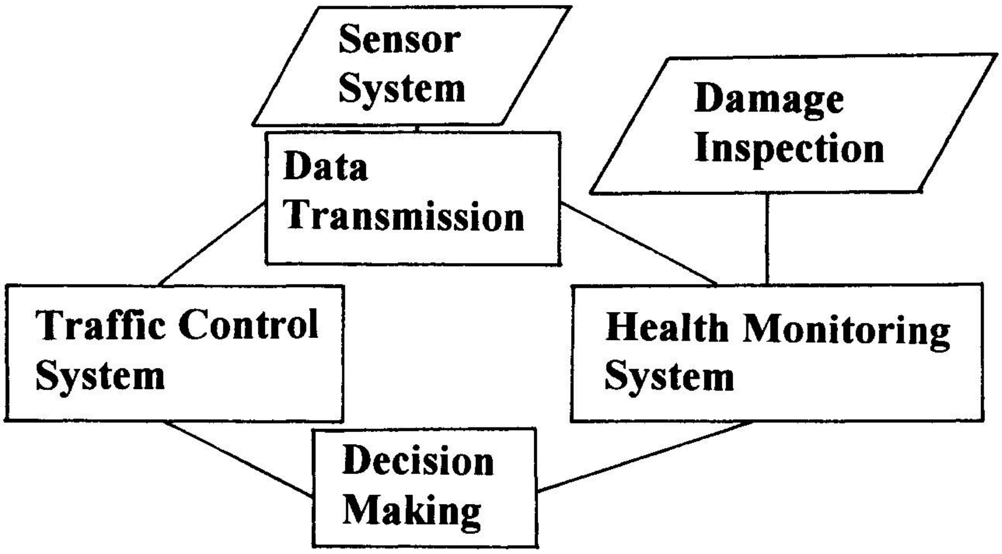

Fig.4 Preliminary Architecture of the Global Monitoring System

3. TRAFFIC CONTROL SYSTEM

For cable-supported bridges, most of the critical vibration modes can be quick determined qualitatively by the finite element method together with some simplifying assumptions. Once the standard vibration patterns of the three bridges have been constructed and these data are stored in a bank, or called finger-prints bank, a particular vibration response of the bridge may easily be identified at any instant by a pattern recognition technique[3]. The pattern bank is organized in a hierarchical structure in terms of bridge components, i.e. towers, decks and cables, and in terms of vibration types, e.g. vertical, lateral and torsion modes. To facilitate pattern recognition, a set of rules to be established to identify these modes. Moreover, some measures must be employed to match a specific mode or linear combinations of several modes. These measures can be one of several choices: distance, energy, inertia, and contract. The fuzzy set method may also be employed to assist our data processing in pattern recognition.

The control strategy in pattern recognition is considered as follows. For a given set of real-time dynamic signals, the corresponding data is classified as either one of the following three types. (1) **Type I Common vibration modes** - These are the natural vibration modes and those under ambient responses which are established under the task of pattern construction. It is anticipated that most of he real-time signals will fall into this category. (2) **Type II Common modes with abnormal features** - For a given dynamic situation caused by a traffic accident, it is quite possible that most part of the recorded signals falls into Type I except some special features appearing in localized area. These are treated as new patterns which will be fed into the pattern bank. (3) **Type III New vibration modes** -Under some unexpected environmental (wind) loading or traffic accidents, it is quite possible that a new set of vibration modes can be generated by real-time simulation of these events and quick dynamic analysis.

Once a particular dynamic pattern is identified, the system will check the possible resonance state of the bridge or its critical performance limits as to determine whether or not a decision (or recommendation) on traffic control should be issued. The proposed decision-making system is designed by way of en expert system which consists of four modules: user interface, rule base, inference machine, and a learning system.

The user interface is intended primarily for interrogation between the operator and the system. In many occasions, the operator may have to take actions via the interface, e.g. storing input or generated data, mitigating current events, checking intermediate decision status. In the rule base, specific rules are designed to control the traffic according to certain defined situations. For example, the rules in the expert system are expressed as IF-THEN statements. When the IF portion of a rule is satisfied by the facts, an action specified by THEN is subsequently issued. Rules may also be issued in a conditional form, e.g. IF A THEN B WITH C, where C is a kind of certainty factor. If necessary, a more efficient model can be constructed[4]. The inference machine is used for making decisions. In a factor graph, the follow-up operation is always based on the current operation. The learning system is for the operator to modify the rule base.

4. HEALTH MONITORING SYSTEM

For bridges, this is in fact sometimes called a damage assessment system. A salient difference between the health monitoring and traffic control is that the former does not require real-time response. However, continuous recording the vibration signals and comparison of these signals with the data stored in the pattern bank are also important for damage assessment. The present system consists of three modules: damage models, system identification, and damage evaluation. Furthermore, the system is supported by the measured data from sensors and assisted by manual damage inspections.

The damage models define the possible modes which may occur in typical cable-supported bridges. Among these, fatigue failure and corrosion, particularly at connections, are most common. In fact, several failure models for bridges are already in existence and some of the theories have been computerized for prediction of residual life or future risk ratios of structures[5]. In addition to the built-in damage models in the proposed expert system, some of the physical parameters of these models may have to be defined by data collected through (occasional) manual inspections. Based on the model predictions and a certain factor relation graphs, global damage assessment of the bridge can be made by some advanced inference method[4]. The system identification is equivalent to an inverse process in structural dynamics. The central idea here is by comparing the currently measured dynamic signatures of the bridges with the base-line signatures of undamaged bridges, the possible locations and even the extent of damages can be identified[6]. In general, the vibration signatures may include those parameters, e.g. fundamental frequencies, mode shapes, and damping ratios. In actual practice, bridge damage may not cause measurable changes in vibration signatures. To overcome this difficulty, a newly defined parameter, called model energy transfer, is far more sensitive to any structural changes than natural frequencies. The results obtained from both mentioned modules will be combined and then assessed by a knowledge-based evaluation module. It is noted that several systems for damage assessment of existing buildings have already been developed in China[5].

5. REMARKS

Following the preliminary design of the global monitoring system several relative projects have been started. Under the arrangement and support of the Lantau Fixed Crossing Project Management Office of Highways Department of Hong Kong a number of joint research groups have been organized between University of Hong Kong, Hong Kong University of Science & Technology, Hong Kong Polytechnic University, and Tsinghua University.

Acknowledgments

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Continuous Monitoring of Bridge Structures

Surveillance continue des structures de ponts

Kontinuierliche Überwachung von Brückentragwerken

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SUMMARY

This paper reports on continued efforts to develop a global monitoring approach applicable to bridges. A fully operational, remote vibrational system is used. Evaluation is based on using normal traffic loading to develop a signature which will change when significant changes occur in the structural integrity. The vibrational signature has changed in this bridge due to lack of movement in the support bearings. The goal of this long term research has been to develop a bridge monitoring system which can prevent catastrophic consequences.

RÉSUMÉ

L'article rapporte les développements d'une approche globale de surveillance applicable aux ponts. Un système totalement opérationnel de mesures d'oscillations à distance est utilisé. L'évaluation est basée sur les vibrations provoquées par les charges de trafic normal et qui se modifient lorsque des changements significatifs apparaissent dans la structure. Dans le cas du pont étudié, la modification du signal était due à l'absence de déplacement des appuis. Le but de cette recherche à long terme a été de développer un système de surveillance pouvant prévenir des conséquences catastrophiques.

ZUSAMMENFASSUNG

Der Beitrag berichtet von Fortschritten bei der Entwicklung eines globalen Ueberwachungskonzeptes für Brücken. Dazu wird ein voll funktionsfähiges System der Schwingungsmessung aus der Ferne verwendet. Die Auswertung beruht auf Schwingungen durch normale Verkehrslast, die sich bei bedeutenden Schäden an der Tragsubstanz ändern. Beispielsweise ändert sich die Schwingungscharakteristik infolge mangelnder Bewegungsmöglichkeit in Brückenlagern. Das Ziel dieses langfristigen Forschungsvorhabens ist ein Brückenüberwachungssystem, das katastrophale Folgen verhindern kann.

1. Introduction

Approximately 20% of the bridges in the United States are structurally deficient or inadequate for current loading conditions. Many of these bridges are nearing their life expectancy, and others were designed for lighter loads and/or lower traffic volume. Economics dictates that not all problem bridges can be renovated or replaced when they achieve their life expectancy. Thus, inspection becomes critical.

Present practice in the United States, using available manpower, is to inspect bridges visually on approximately a two year cycle. This is not always adequate to prevent catastrophic consequences. In 1983, a section of a bridge on the interstate in Connecticut collapsed. Routine inspection of the bridge 10 months earlier was not able to provide indication that one of the pins holding up the span had developed fatigue cracks due to corrosion. In 1988, another bridge in Rhode Island was closed after discovery of a long crack in one of the girders. This crack would have resulted in collapse, had a passing motorist not contacted authorities. The bridge had been inspected 5 months earlier. Scour of bridge piers resulted in a collapse in New York in 1988, which also had recently been inspected.

These collapses, or near collapses, indicate the desirability of developing equipment to supplement the visual inspections. The general report from the 1992 Conference on Nondestructive Evaluations of Civil Structures and Materials in Colorado [see Reference 4], states that 'the present practice of visual inspections at long intervals must be replaced by frequent, automated condition monitoring" and that this should "provide an early warning of distress, support aggressive maintenance programs and promote the timely remedy of emerging deterioration." What is needed is a continuous system which can send an alarm when there is a serious structural problem. The system can't rely on manpower alone or special inspection techniques, since these are not always available when needed. Further, the system must act as a global monitoring technique. It must be able to detect a variety structural problems, including those which are not expected. Monitoring systems should link sensors placed at different locations on a bridge and should utilize recent advances in computer techniques and hardware.

Many techniques have been proposed for use of different kinds of sensors to assist in the evaluation of bridges. Most are limited to localized evaluations. Examples include evaluation of the strains in specific bridge components, acoustic emission techniques to follow a fatigue crack and tilt meters to evaluate support movements. Some researchers have proposed use of vibrational information, which is capable of providing a global assessment of the bridges. Presently vibrations are used in many monitoring applications, including manufacturing plants, power plants and aircraft.

Samman and Biswas [1] recommended using pattern recognition with vibrational measurements as a means of evaluating bridge problems. Lauzon and DeWolf [2] have reported on testing, conducted by Lauzon, of a bridge near collapse. Vibrations were used to obtain bridge signatures as the crack developed. This test demonstrated that changes to the structural integrity are discernable from vibrational information. The Lauzon test was part of a long term effort in Connecticut to develop a field monitoring techniques, involving both strains and vibrations, for application to bridges.

This paper reports on the recent efforts to use a fully operational remote vibrational monitoring system on an older continuous bridge. The bridge's performance has been evaluated in both summer and winter. The vibrational signature has changed due to changes in the supports.

2. Vibrational Monitoring Approach

Based on a laboratory study with a bridge model subject to moving loads, Mazurek and DeWolf [3] concluded that the ambient vibrational approach could provide a feasible bridge monitoring technique. They found that major deterioration is detectable by comparisons of natural frequencies and mode shapes. This information was correlated to finite element analyses.

Lauzon [2] applied the results of the laboratory study to a portion of a bridge with three girders. A crack was introduced in the outer girder, and a moving truck was used as test vehicle. Extension of the crack into the web resulted in the development of additional frequencies and changes to existing ones. Acceleration levels changed dramatically, depending on the location of the sensor. The conclusion was that a statistical analysis of the frequencies and the relative acceleration levels could provide a global sign of loss of structural integrity.

3. Prototype Monitoring System

The vibrational monitoring system was built by Vibra-Metrics of Hamden, Connecticut, in conjunction with researchers at the University of Connecticut. The system has 16 accelerometers, two cluster boxes and a sentry unit. The accelerometer placement is based on a finite element analysis which identifies the appropriate mode shapes. The sentry unit contains a computer with software for processing the data from the accelerometers. It also provides for communication with a remote monitoring site.

The monitoring system was first placed on a newer single span bridge consisting of a composite steel plate girder deck [4]. The results demonstrated that traffic induced vibrational data is stable and provides adequate information to develop a signature. Modifications were made to the system before installation on the bridge reported in this study. These consisted of refinements to the system resolution and communication capability.

4. Bridge Description

The bridge studied has two equal length spans, with continuity at the center support. The superstructure consists of a composite concrete slab on seven non-prismatic welded steel plate girders. The cross-section is shown in Figure 1. The bridge was built in 1954 and the bituminous deck was replaced in 1976. There are plans to replace splice connections in the future.

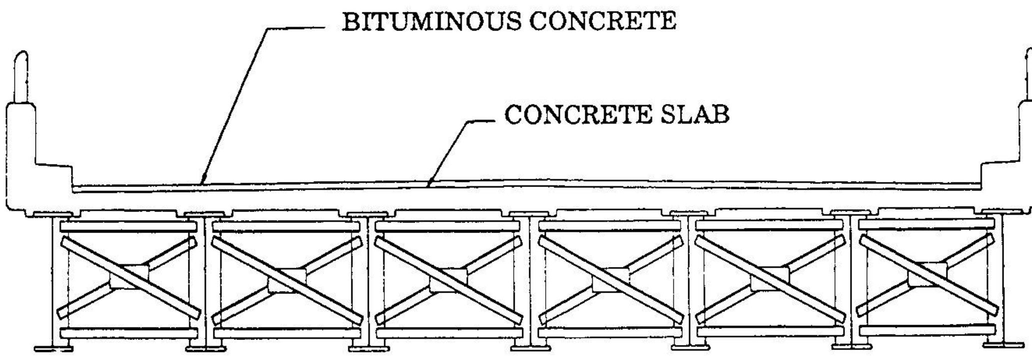

Figure 1: Bridge Cross-Section

The 16 accelerometers were located in both spans, distributed in both the longitudinal and transverse directions.

5. Field Results

Previous work at the University of Connecticut has defined the baseline signature of a bridge in terms of the natural frequencies and mode shapes. The acceleration patterns also form a part of this signature. The monitoring system was used to collect accelerations and process the data to develop the modal information. A typical frequency spectrum, taken during one collection period, is shown in Fig. 2.

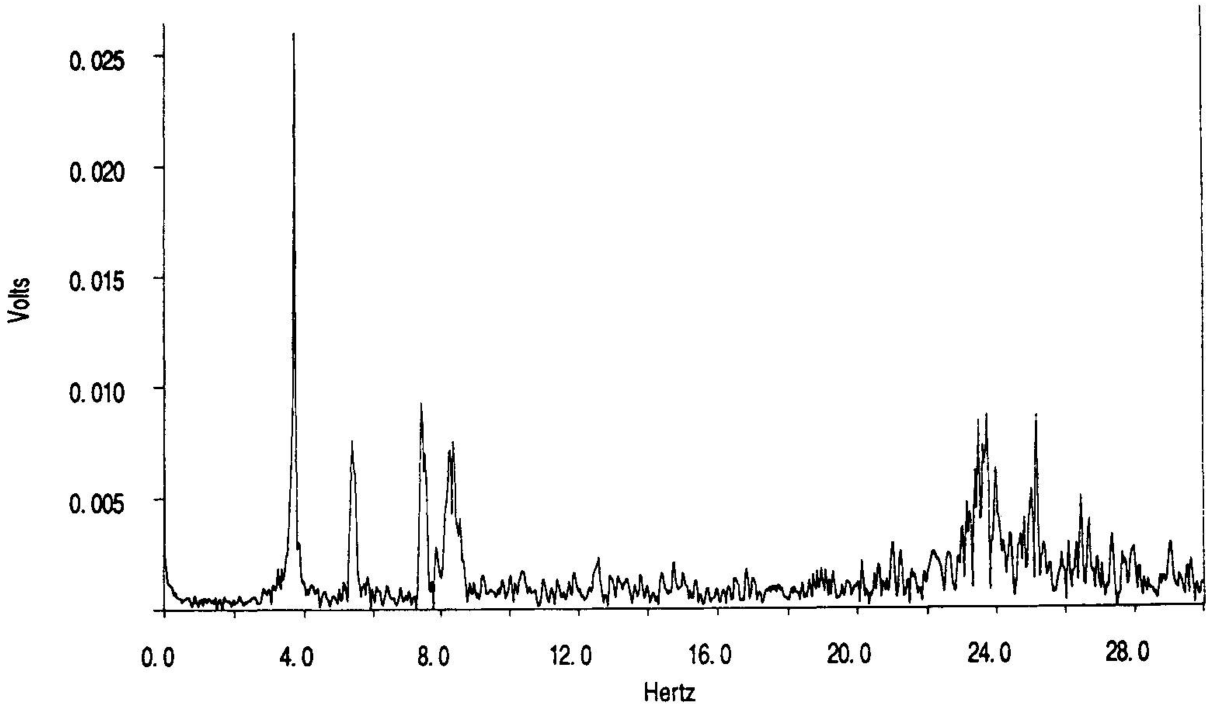

Figure 2 Typical Frequency Spectrum

This figure is a plot of volts (proportional to acceleration) versus frequency. Natural frequencies are associated with peaks in this diagram. However, not all peaks correspond to natural frequencies. It is necessary to compare spectra using modal analysis to determine which peaks correspond to natural frequencies.

The natural frequencies for the first bending (displacements vary in longitudinal direction), first torsional (displacements vary in the transverse and longitudinal directions), and second bending modes were identified using the monitoring system. In November prior to the onset of colder weather, these values were 3.6, 4.15, and 5.3 Hz respectively.

Small peaks were found at frequency values near 7.5 and 8.5 Hz on a regular basis. However, when an attempt was made to identify the mode shapes for these frequencies, the phase information was unrecognizable. In the range of 12.0 to 16.0 Hz, there are many peaks which were consistently excited. Two peaks usually appeared at 14.1 and 15.0 Hz, but again, the phase information was unrecognizable. While, analytical and finite element beam models indicate that there is another bending mode in this range, the large number of peaks within this range did not allow determination of the frequencies from the test data. Previous studies at by researchers at both the University of Connecticut and at other institutions have demonstrated that there is only sufficient energy to excite the lowest natural frequencies, and thus that it is not possible to decipher the higher modes.

The bridge's mode shapes are directly related to acceleration levels. To plot a mode shape, the phase angle of each channel is determined with respect to a given reference channel. The lowest bending and torsional modes shapes are shown in Figs. 3 and 4, respectively.

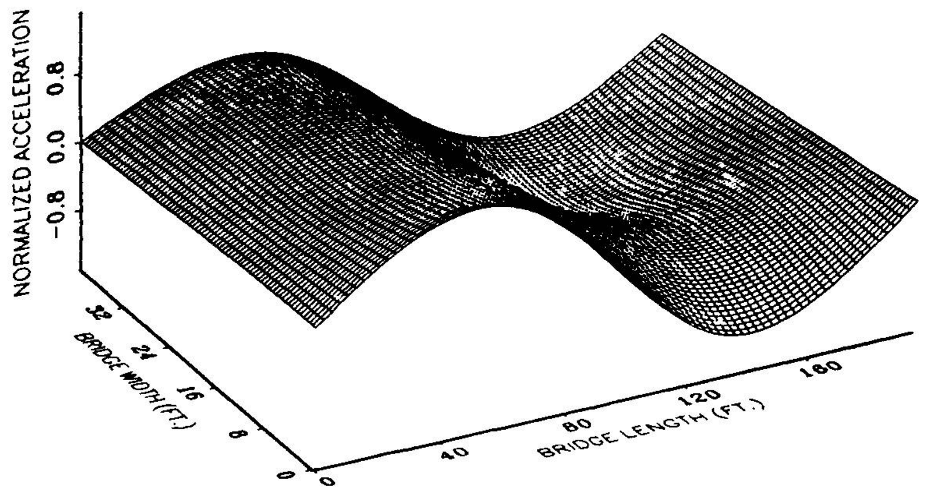

Figure 3 First Bending Mode Shape

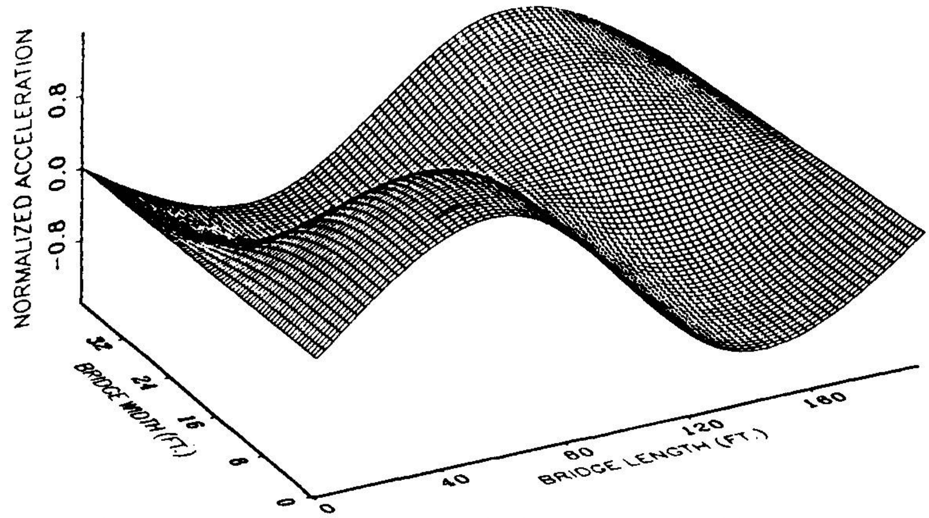

Figure 4 First Torsional Bending Mode Shape

The stability of the natural frequencies and mode shapes was determined by comparing data collected at different times. Lauzon and DeWolf have shown that structural damage affects relative acceleration levels, in addition to the frequencies and mode shapes. Thus other statistical efforts involved evaluation of accelerations. The results of these comparisons is reported in more detail by Conn and DeWolf [5,6].

At the onset of colder winter weather in December, the three lowest natural frequencies associated with the mode shapes began to shift (from 3.6 to 4.0 Hz, from 4.15 to 4.8 Hz, and from 5.2 to 6.0 Hz). A plot of temperature vs. frequency for the lowest natural frequency is shown in Figure 5. Further study indicated that the frequencies changed only for temperatures with values between 0°F to 60°F. There are no significant changes in natural frequencies for temperatures above 60°F.

Review of possible causes indicated that the bridge bearings were not completely free to translate. The bridge was thus not able to contract, and as a consequence axial tension forces were developed in the girders. Analytically, it has been shown that the lowest natural frequencies will change by approximately 5 percent with a 60°F temperature change, based on a prismatic single beam model [5,6]. Further study is now underway to evaluate and verify the changes in natural frequencies. The results establish that changes in the structural integrity do result in changes in the bridge vibrational signature.

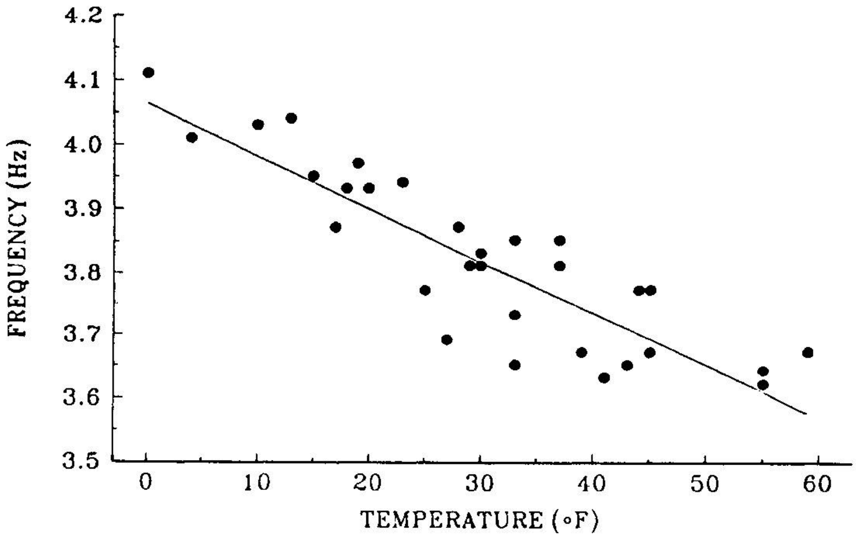

Figure 5 Frequency vs. Temperature for First Bending Mode

6. Conclusions

As shown in previous work with the vibrational monitoring system on a newer bridge, natural frequencies and mode shapes were relatively stable under varied traffic conditions. These form the bridge's baseline signature.

In this study, the monitoring system was placed on a continuous two span bridge. The data collected was relatively consistent during warmer weather. In the winter, the substantially lower temperatures caused the natural frequencies to increase. It was concluded that these are due to lack of full displacement at the bearings at the end of the bridge. These bearings should be free to rotate and allow longitudinal displacement. The changes to the natural frequencies demonstrate that the proposed approach, based on the vibrational monitoring, is able to detect changes in the structural performance.

This study has been a part of a continuing effort to establish that vibrational information can be used to evaluate the structural integrity of a bridge. Support from the State of Connecticut Department of Transportation and Vibra-Metrics, Hamden, Connecticut, a company which manufactures vibrational equipment, is gratefully acknowledged.

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Permanent Corrosion Monitoring for Reinforced and Prestressed Concrete

Contrôle permanent de la corrosion sur les constructions en béton structural

Kontinuierliche Korrosionskontrolle bei Stahl- und Spannbetonkonstruktionen

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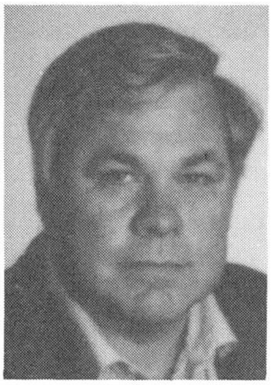

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SUMMARY

In the field of reinforced concrete structures with high chemical pollution, there is a need of quality control of the structures. This is especially true for prestressed steel structures, which are used most by bridges. In addition to the known half-cell monitoring systems which work only on the surface, there is a new monitoring system with an electrode available which is installed in the structure either when first built or when repaired. The new monitoring system, called corrosion monitoring system, is described in this paper.

RÉSUMÉ

Dans le domaine des constructions en béton structural, qui sont exposés à une grande pollution chimique, il est nécessaire d'avoir un très bon contrôle permanent, spécialement pour les ponts en béton précontraint. En plus des systèmes de mesure connus, travaillant seulement en surface, il y a maintenant une possibilité nouvelle de mesurer la corrosion avec une électrode, installée soit pendant la construction, soit ultérieurement au cours de la maintenance. Ce nouveau système de mesure est décrit dans cet article.

ZUSAMMENFASSUNG

Bei Stahlbeton- und Spannbetonkonstruktionen sowie bei Daueränkern, die einem unentwegten chemischen Angriff unterliegen, muss eine dauerhafte Kontrolle des Stahles gefordert werden. Besonders bei vorgespannten Stahlkonstruktionen, wie sie im Brücken-bau verwendet werden, ist eine solche Kontrolle unbedingt erforderlich. Zusätzlich zu den bereits bekannten Messverfahren, die die Oberfläche der Baukonstruktion mit Halbzellen abtasten, gibt es nun eine neue Möglichkeit, mittels einer eingebauten Elektrode, die ent-weder beim Neubau mitinstalliert oder nachträglich bei Erhaltungsmassnahmen ein-gebaut wird. Dieses neue Messsystem, genannt CMS (Corrosions-Mess-System) wird in diesem Artikel beschrieben.

1. FUNDAMENTALS

All over the world bridges and other comparable structures face great problems because large quantities of salt are applied for removing ice and snow on roadways in winter. This salt damages the steel parts of the various constructions and causes them to corrode. In several countries, such as Germany, the USA, Great Britain, Switzerland, Austria, Japan, and many others, requirements for concrete used in construction were toughened (low W/Z factor, definite content of air pores, etc.) in order to avoid this electro- chemical corrosion process. Moreover, the covering of steel parts with various materials was defined more precisely. It has to be pointed out, however, that all these monitoring have not yet lead to a satisfactory situation. Still corrosion of steel can not be absolutely prevented. For this reason a monitoring system is being searched for all over the world which definitively describes the condition of steel with regard to possible corrosion. Since such a system had not been found and thus a great gap between quality guarantees and results for prestressed bridges still existed, the Department of Transport of the United Kingdom decided to no longer permit any prestressed bridge to be built after September 25th, 1992. This was a blow for the entire prestressed concrete industry.

2. ACTUAL MONITORING SYSTEMS

Looking at the current literature on corrosion and corrosion monitorings in reinforced concrete structures you'll find, besides the traditional optical methods, only one system which can measure the electro-chemical process of corrosion in reinforced concrete without damage. The monitoring system is based on the fact, that when corrosion occurs an electro-chemical process takes place which can be directly monitored with an electrode and typical electical monitoring devices. Several systems are offered:

- a permanently installed monitoring electrode in all types of structures or
- a system for surface sounding.

The latter one has the advantage that all accessible steel parts near the surface can be checked fairly quickly and reliably.
In this case the surface of the concrete is surveyed by means of a half-cell and thus the electric potential between the half-cell and the reinforcement steel is monitored.

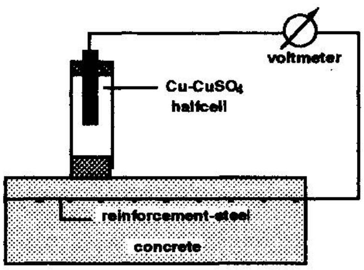

Fig.1 Potential monitoring by means of Cu - $CuSO_4$ half-cell.

Since it is difficult or impossible to investigate the entire area of a structure with only one monitoring cell, monitorings can also be carried out with a series of eight monitoring cells in one operation.

This is shown in the following picture:

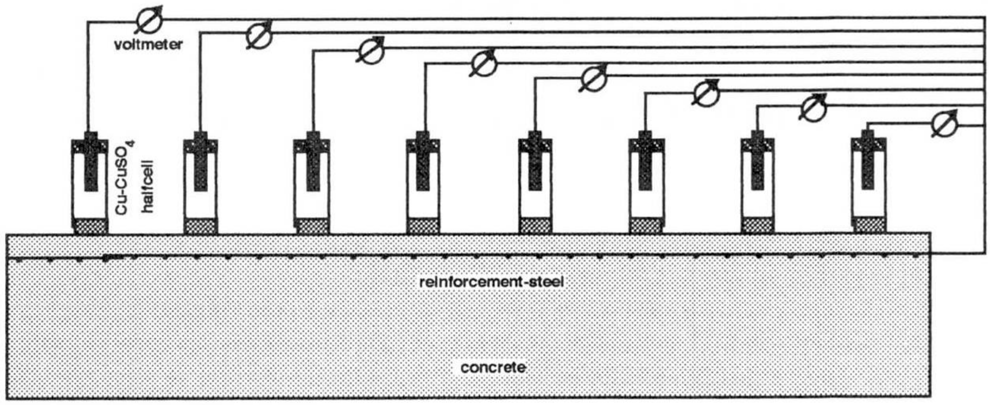

Fig.2 Monitoring system with 8 monitoring cells applied with a bridge construction

For the first time one can obtain a reasonably complete picture of the condition of surface reinforcement on a large area of reinforced concrete structures. The data can now also be evaluated graphically in a way that reveals the respective condition at all surface locations at once. In the following figure you can see an investigation made on the bottom side of a bridge where considerable corrosion areas were found on the projecting sides (overhang).

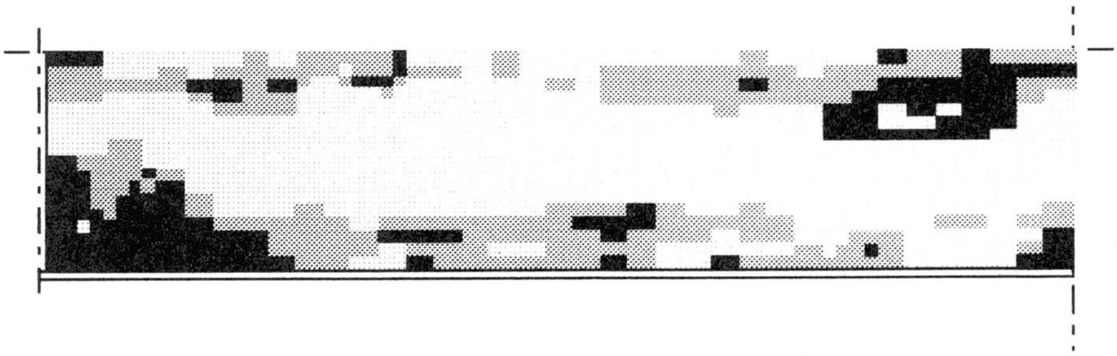

Fig.3 Evaluation of the potential monitoring method applied on the bottom side of a bridge.

Not only with bridges, which are exposed to chloride due to winter service, it is advisable to monitor the entire area of the reinforced concrete structure as well as other structures such as: buildings, water purification plants, sewage treatment plants, dams, etc.

For example, shown are the monitoring results of a sewage treatment plant with a reinforced concrete basin .

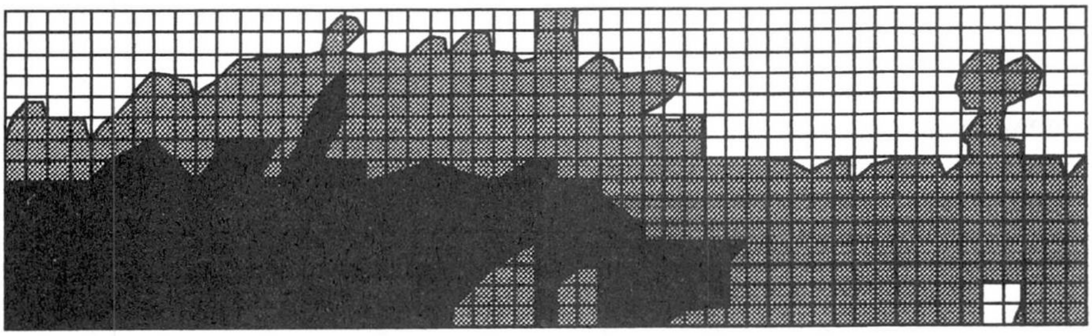

Fig.4 Evaluation of potential monitoring in the basin of a sewage treatment plant.

On first view it is amazing that in case of a basin of reinforced concrete such large corrosion areas occur where the quality of the concrete as well as the sheathing of the reinforcement steel corresponded to the normal standards and practices. Thus one would not expect corrosion to occure moreover, no especially agressive fluids are held in the basin. The pH-value is continously controlled and monitoring 7 which is within the neutral range. The only difference to normal water is that here an extremely high amount of organic parts exists and that these parts contain oxygen and thus activate the necessary biological processes in the basin. This biologically active fluid penetrates the concrete and in the process replaces the usual pore-water. Thus, the pH-value of the concrete can be reduced from its normal value of 13 all the way to 7 in extreme cases.

Under these conditions all the criteria for corrosion in reinforced concrete structures, such a

a) the pH-value goes down to less than 9
b) there is oxygen
c) there is water

exist. This means that the usual concrete covering alone does not guarantee a sufficient protection against corrosion of the reinforcement steel. This necessary protection can only be achieved by additional monitoring which permanently prevent one of the three prerequisites for corrosion. These monitoring are not the subject of this report, they will be described in detail in other papers.

On the whole the surfaces where potential monitoring was carried out show larger areas of corrosion than can be recognized by visual means alone. This makes sense if one remembers that the corrosion products, $Fe_2 O_3$ or Fe (OH) or other iron compounds, have to penetrate the entire layer of sheathing to the surface before they can be optically observed. Moreover, a certain degree of disintegration of the steel is necessary before the corrosion products reach the concrete surface. As a consequence, with the optically recognizable corrosion areas, a diminution of the cross-section of the reinforcement part has already taken place and it is certainly time for reconstruction, at which time a passive monitoring system for the reinforcement steel should be installed.

The greatest disadvantage of potential monitoring by means of these half-cells is that you must have direct access to the place of monitoring when applying this method. Steel parts which cannot be reached by the half-cell cannot be monitored. Therefore, most of the prestressed parts used in prestressed concrete structures cannot be monitored, because, in most cases they are not situated directly on the surface of the concrete but between other normal reinforcement steel elements.

Moreover, most of the prestressed reinforcements are shielded by what is called a protective tube or sheath of metal or plastic so that monitoring in the way described above is impossible. Again and again it can be observed that prestressed links corrode even though they were checked on installation and that subsequent cracking stress corrosion can occur. Since, at present, these sheathed prestressed links can not be monitored after installation and so their future remains a matter of unknown confidence, the United Kingdom made a rigid decision by prohibiting the further use of prestressed links for bridge construction. Experts in this field were thus challenged to find a monitoring method for monitoring single prestressed links for their state of corrosion. When such a system exists, the return to using prestressed concrete constructions in bridge structures in the United Kingdom can be expected.

3. NEW MONITORING SYSTEMS

Two systems for monitoring the usefulness of prestressed links have come on the market, both of them resulting from anchor construction. There too, there always has been an uncertainly about the corrosion of the steel prestressed parts. Therefore, the standards of all countries require a double corrosion protection which offers a certain security as far as corrosion is concerned; monitoring, however, is still not possible. In the following, the two monitoring systems are presented and evaluated according to their usefulness for the owner of the structure.

3.1. Electrical isolation encasement of the prestressed link (Swiss Method)

This method, at present, applied only with anchors in ground and rock, aims at encapsolating the entire steel stress link together with corrosion protecting anchor grout, in an electrically insulating capsle made of plastic and ceramics and at monitoring only the insulating property of the encasement. The measurement on the encasement is made by electrical resistance monitorings which are made during the installation for control purposes as well as after the stressing in order to check the installed anchor. This means that only the electric insolation of the anchor itself is being controlled. For the necessary resistance measurement, relatively high currents (500 volt) are used the impacts of which on the complete system are unknown and potentially damaging if everything is not perfect.

If the resistance monitorings reveal a hole in the encasement this is to be considered a negative result; it still remains uncertain, however, whether the stress link in the encasement will corrode or not. None of the three critical elements necessary for corrosion to occur are checked by this measurement. On the other hand, the prestressed steel can corrode with an intact encasement if the grout within the encasement was not compressed properly as bad experience has shown in many cases with bridge stress links.

Electric encasement serves as a monitoring method for the validity of the encasement. It is a necessary answer for the owner of the building, however, it is missing a key element and therefore it is almost useless. You can´t guess from the package whether or not the milk is sour - you have to examine the milk itself!

3.2. Direct corrosion monitoring with installed electrode (CMS)

A special electrode in the shape of a wire is put around the stress link which shall be examined and the potential between the electrode and the stress link is monitored as is done with the potential monitoring method by means of a half-cell. Similarly, as with the potential monitoring method with the half-cell, corrosion can be recognized by the potentials monitored. The electrode is a silver - silver chloride electrode as is used in electro - chemistry as a reference electrode. This kind of electrode has also proved successful for medical purposes, i.e. for millions of electro-cardiograms.

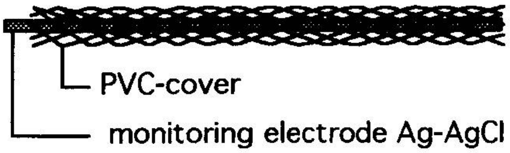

Fig.5 monitoring electrode with PVC-cover as protection against contact

The sensitivity of this system is so high that even with a length of 20 m, a corrosion area of 1 mm can still be recognized. Thus, depending on the arrangement of the monitoring electrodes an exactly defined part of the steel or of the prestressed part can be monitored.
You cannot, however, recognize in which part of the electrode, the corrosion occurs. Also, its extension and intensity cannot yet be defined. It is, however, irrelevant were corrosion occurs on a prestressed part only that it is occuring. When corrosion has been recognized on a prestressed part this part has to be relieved and a respective replacement has to be made. The necessary monitorings can be made either singularly with a normal voltmeter or, in case of larger buildings, by a computer with a fully automatically monitoring device at the site.
In case of a corrosion area, alarm is given automatically and the technical staff can make the necessary decisions for further surveilance. The individual data are, of course, always recorded and stored so that any change of potentials can be recognized subsequently.
With this monitoring method, which is also based on the fundamental principles of electrochemistry, any occuring corrosion can be monitored and the owner of the structure receives the exact information on the quality of the monitored reinforcements and which individual stress steel is involved.

4. CONCLUSIONS

In the field of reinforced concrete structures with high chemical pollution, there is a need of quality control of the structures. This is especially true for prestressed steel structures, which are used most by bridges.
In addition to the known half-cell monitoring systems which work only on the surface, there is a new monitoring system with an electrode available which is installed in the structure either when first built or when repaired. The new monitoring system, called CMS (corroision monitoring system), works in the following applications:

a) Sheated stressed steel members like anchors can now be continuosly monitored for the onset of corrosion.
b) All areas/regions of prestressed steel and reinforced concrete structures can now be monitored for the onset of corrosion, not just surface elements.
c) Thus, through the use of this system, the UK regulations monitoring on prestressed concrete structures can be lifted!

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Monitoring System for Bridges of the Livorno-Cecina Highway

Système de surveillance des ponts de l'autoroute Livorno-Cecina

Das Brückenüberwachungssystem der Autobahn Livorno-Cecina

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Marco Mezzi, born in 1952, received his civil engineering degree at the University of Rome. He was involved in research programs in seismic design. He was professor at the University of Perugia. He has worked as a consultant in design and software application.

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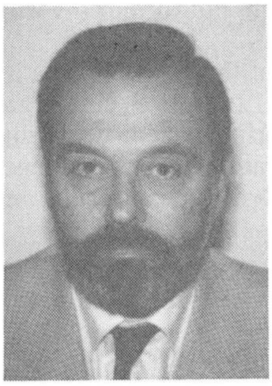

Alberto Parducci, born in 1934, received his civil engineering degree at the University of Rome. He has been involved in aseismic design of bridge structures. His research has involved seismic isolation and patented isolating devices.

SUMMARY

The monitoring project designed for the field control of the bridges of the new Livorno-Cecina highway is illustrated and the main targets are discussed. The project provides systems for automatic permanent measuring, periodic surveying, and seismic recording. Special softwares have been designed and developed for collecting and managing the recorded data. The corresponding procedures are discussed and typical examples are presented.

RÉSUMÉ

Le projet de surveillance et de contrôle des ponts de la nouvelle autoroute Livorno-Cecina est présenté et les principaux objectifs sont discutés. Le projet prévoit des systèmes de mesures permanentes automatiques, pour le contrôle périodique et pour l'enregistrement sismique. Un logiciel spécial a été développé pour la récolte et la gestion des données enregistrées. Les procédures correspondantes sont discutées et quelques exemples typiques sont présentés.

ZUSAMMENFASSUNG

Das für die ständige Bauwerkskontrolle der neuen Autobahn Livorno-Cecina entworfene, fest eingebaute Ueberwachungssystem wird vorgestellt und seine hauptsächlichen Zielsetzungen werden erläutert. Das System garantiert ständige Messungen, Überwachung in regelmäßigen Zeitabständen und Aufzeichnung der Erdbebendaten. Zur Aufnahme und Verarbeitung der aufgezeichneten Daten wurden spezielle Softwareprogramme ausgearbeitet. Die entsprechenden Abläufe werden besprochen und typische Beispiele vorgestellt.

1. INTRODUCTION

Technical evolution and industrialization leads to building complex and sophisticated structures exploiting the extreme resistance of materials. Moreover, in bridge constructions severe problems of durability occur [1,2], so that the frequency of maintenance works increases involving higher repair costs and uneasiness to the users. Thus, the need of planning and designing these works on the basis of organized instrumental control systems has also increased. In the past, little attention was paid to the knowledge of the actual behaviour of the structures during their life time. Only in the last few years the importance of the problem has been recognized and the design of control systems began to be performed together with the structural design. Nowadays, the experience of observers can be validated by instrumental measurements and through direct or indirect tests [3,4], whose reliability derives from the experience developed with their use. At present, the practice for the interpretation of the results is advanced enough, but further investigations must still be performed.

To improve the knowledge of these problems a monitoring project has been designed and applied to the main bridges of the new Livorno-Cecina highway, built in a seismic zone of Tuscany, in central Italy [5]. In 1994 the monitoring system was delivered to the tender SAT Company. Considering the point of view of the structural designers the general organization of the project, the acquisition and management software and some typical results are illustrated in the present paper.

2. THE MONITORED STRUCTURES

The bridges consist of two separate structures m 12.25 wide, one for each carriage-way. The decks were built as continuous multi-span segments of prestressed concrete structures. Different structural configurations and constructive systems were used [6]. All the bridges were provided with seismic isolation systems based on the elasto-plastic behaviour of horizontal restraints [7]. Also special connecting devices consistent with the requirements of the continuity of structures were applied.

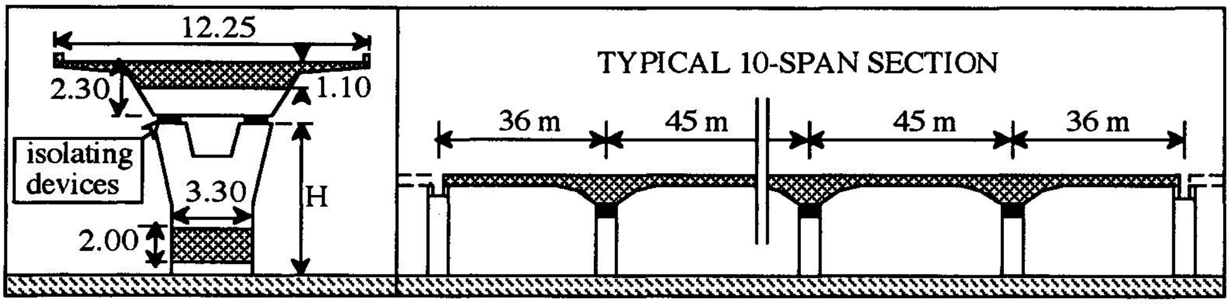

Fig. 1 - General lay-out of the "Coltano" bridge

The "Coltano" bridge, m 9860 long, is the most important and complex. The typical segment, spanning over 10 bays, is 432 meters long (Fig. 1). It was built using a cast-in-situ advancing procedure. Its particular configuration consists in a gross section of variable thickness (m 2.30 over the supports to only m 1.10 in mid-span) with large lateral cantilevers. The decks of the other bridges are all made as boxed structures. The "Gonnellino" bridge has a single continuous segment m 660 long; each span was built connecting two longitudinal precast half-elements. The "Savalano" bridge was built using this same procedure; its total length of m 1860 is divided into three continuous sections. The total length of the "Morra" bridge (m 856) is divided into two continuous segments; the boxed deck was built using an advancing prestressed procedure. The "Poggio Iberna" bridge is m 2518 long and is divided into five continuous segments; it was built by launching entire precast spans connected in situ.

3. THE MONITORING PROJECT

Different purposes were considered designing the monitoring system, including the recording of the effects of environmental conditions, the actual in-situ evolution of the characteristics of the structural materials, the control of deformations and displacements of structural elements (bearing and joint displacements) and the response of the structures to seismic attacks. Therefore the monitoring design, based on a permanent measuring system and periodic surveying operations, was directed to control the following factors:

- environmental data (temperature and humidity);
- evolution of the mechanical characteristics of the concrete;
- displacement of significant elements of the structures (bearing and joints) due to the evolution of the mechanical characteristics of the concrete and to environmental factors;
- local deformation in significant points of the structures;
- structural displacements due to soil settlements;
- response of the structures to seismic attacks.

Three recording stages have been foreseen. The first one was carried out during the final inspection of the works in 1993. In the second stage, during the following year, the data acquisition and management "ADaMo" software was developed and the general procedures were tested. At present, the normal monitoring survey is being carried out by SAT.

The permanent measuring systems consist of strain-gauges (±2500 μ/m), put into the concrete during the construction which record local deformations, thermometers (-50+150°C) and hygrometers (0÷100%HR) which record the near-by external data and those in the boxed structures (Fig. 10). Starting the "Indaco" automatic procedure the instrumental data are digitalized and recorded in peripheral acquisition systems. Moreover, the bearings displacements and the relative deformations of the expansion joints between continuous sections can be measured through digital devices.

The periodic surveying system consists in the following activities:

- topographic levelling of the vertical configurations of decks and foundations;
- dynamic and static load tests of typical spans;
- direct and indirect measurements of the resistance of the concrete.

The concrete survey is performed using cubic and cylindrical crushing tests, together with ultrasonic, rebound, pull-out and penetration tests. To avoid taking out samples from the bridges, sets of standard cubic concrete samples and large concrete blocks were fitted up during the construction works and laid down near the structure, so that cylindrical samples can be driven from the blocks.

The seismic surveying system consists of strong-motion recorders of the three response components (±2g, 0.02÷2000 Hz, post and pre-trigger of 10 s) in three points of typical spans: foundation, top of the column, deck structure. They start to run when the horizontal acceleration of a foundation device exceeds 0.02g. A special procedure avoids that loud peals of thunder start up the system.

Particular tests have been also performed during the construction works. Significant is the result achieved from a dynamic test carried out before the construction of the decks on a single pier of the "Coltano" bridge. In spite of the rigidity of the foundations, 80% of the longitudinal flexibility derived from the foundation's deformations and only 20% from the column's flexibility (Fig.2). This result emphasizes the importance of the foundation flexibility on the estimation of the natural periods.

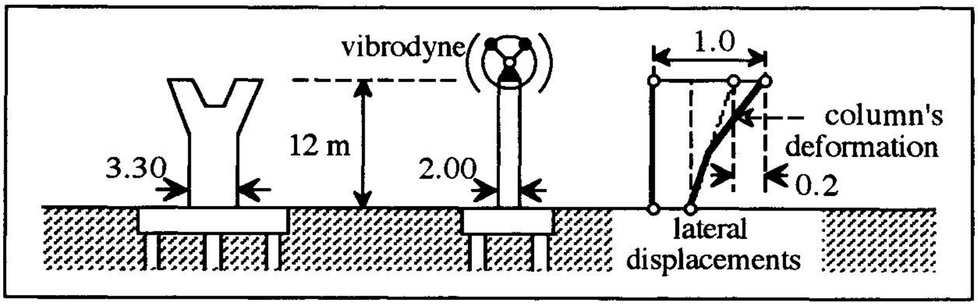

Fig. 2 - Dynamic test of a pier before the construction of the deck structure

4. THE DATA-BASE "ADaMo"

The field measurements are collected by the data-base "ADaMo" (Archiviazione Dati di Monitoraggio) which allows their management, showing their evolution and possible correlations. In order to accept different measurement procedures, the storage system uses uni-dimensional data containers ("instruments") collecting data with respect to time and location: the "time instruments" contain the evolution of data in time (example: the automatically recorded data); the "location instruments" contain the data recorded by different devices at the same time (example: the bearing displacements). The sets of data are then considered as bi-dimensional time-location matrixes.

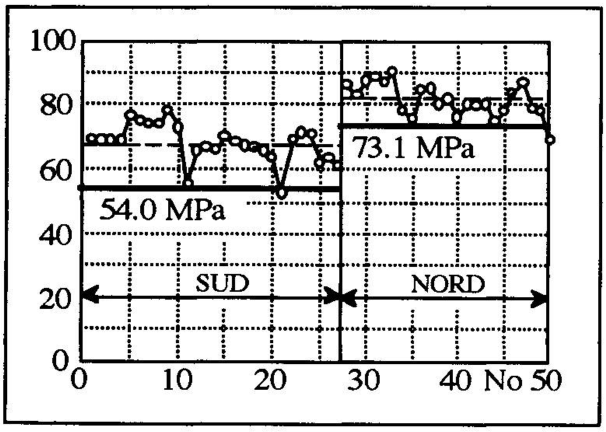

Fig. 3 - Standard resistances of cubic samples

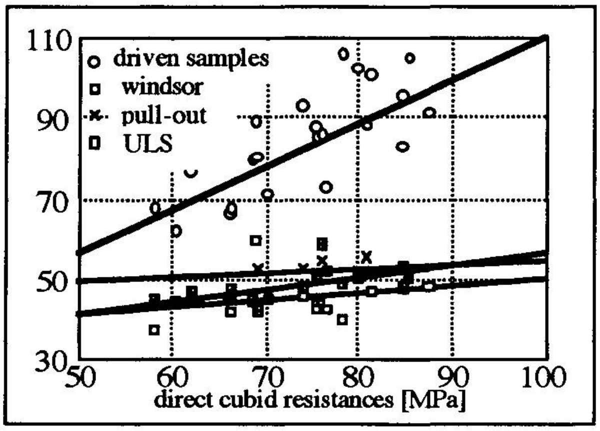

Fig. 4 - Correlation analyses

Some results are shown in Fig. 3, where the standard resistances of one year old concrete cubes are plotted, and in Fig. 4, where typical correlation are represented. The data of static tests are also managed by "ADaMo". Fig. 5 illustrates the representation system. In order to compare the results of tests carried out in different times, the measured displacements are plotted versus the theoretic values calculated from the real positions and intensities of each load.

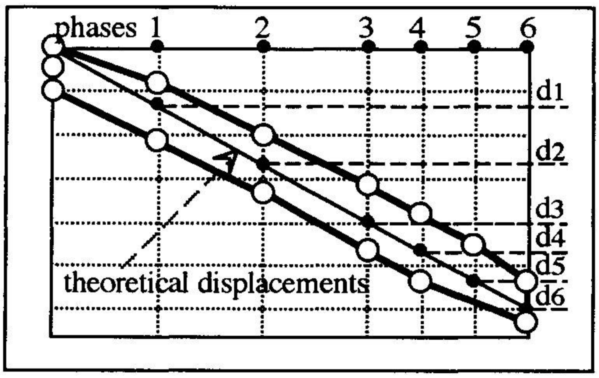

Fig. 5 - Typical result of a load test

Examples of the typical records of "ADaMo" are reproduced in Figs. 6 and 7. The former shows the bearing displacements and the deformations of the nearby expansion joint, plotted versus the time; the latter shows the correlation between two recorded data (longitudinal mid-span deformations and temperature gradient between top and bottom of a boxed section). Finally, the general scheme of the software is sketched in Figs. 8 and 9, while the locations of the permanent monitoring devices are illustrated in Fig. 10.

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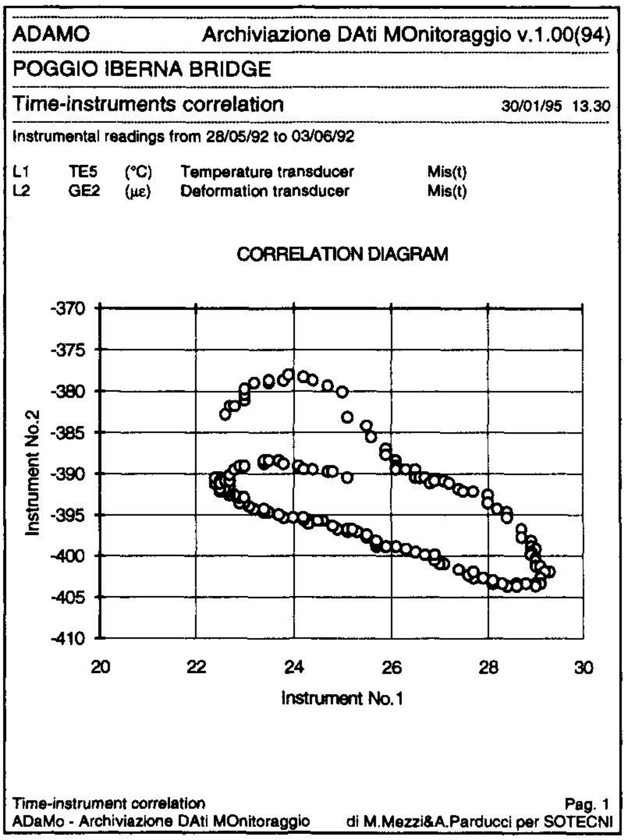


Fig.6 Sample of "ADaMo" report

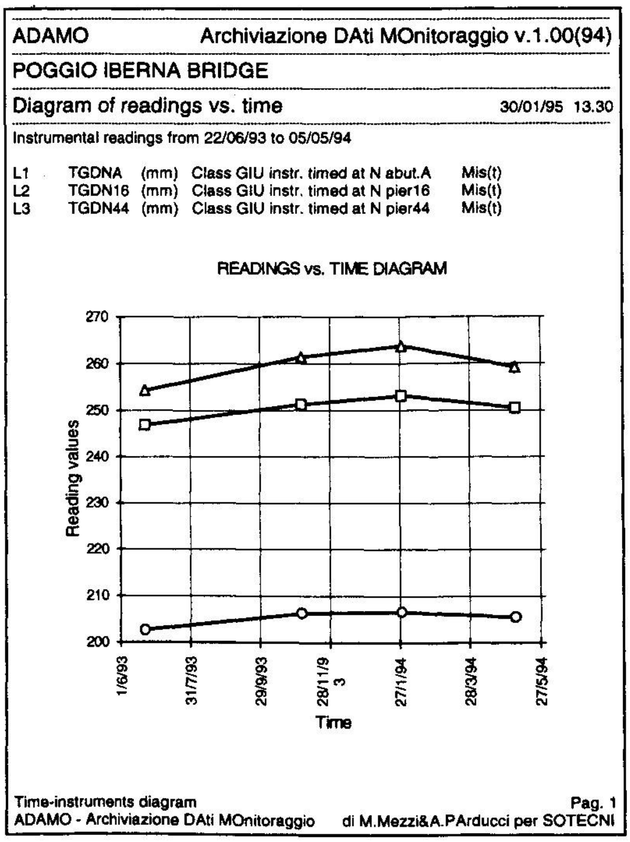


Fig.7 Sample of "ADaMo" report

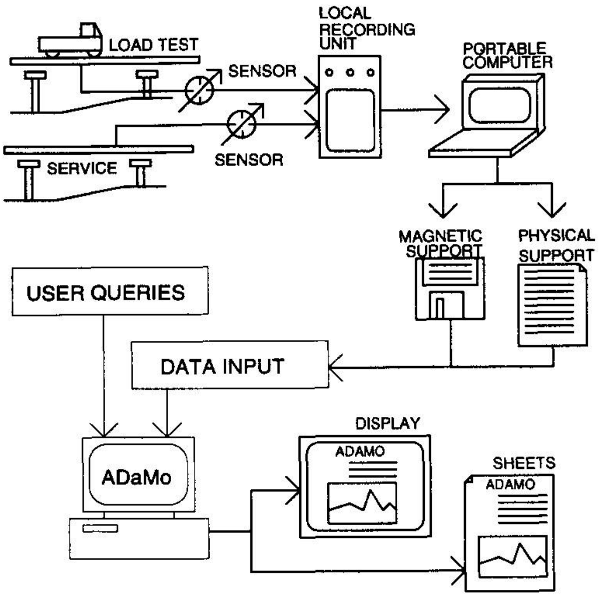


Fig.8 General scheme of the monitoring system

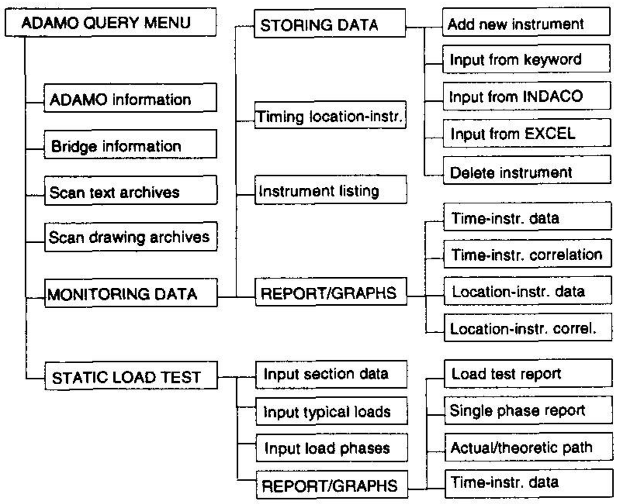
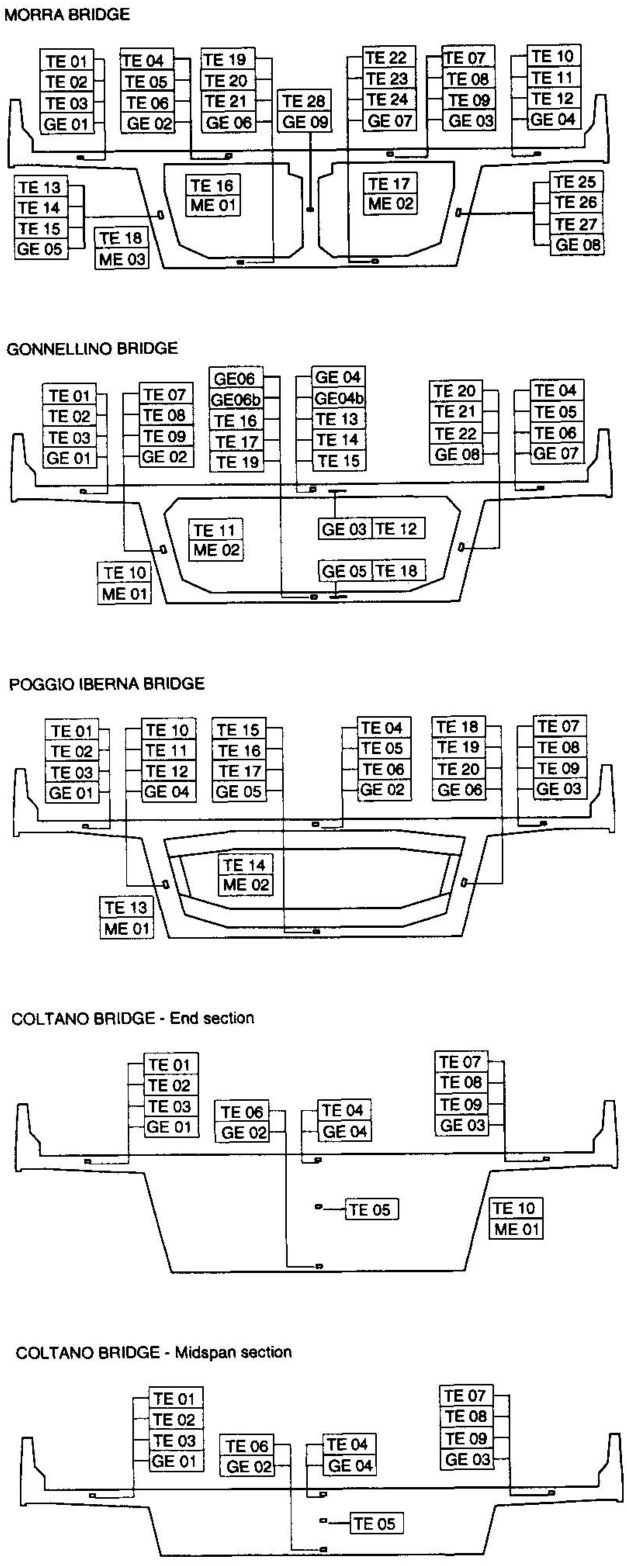


Fig.9 "ADaMo" query structure



Transducers: TE=temperature, ME=humidity, GE=deformation

Fig.10 Locations of the permanent monitoring devices

Monitoring of Granite Structures

Surveillance des ouvrages en granit

Ueberwachung von Granit-Bauwerken

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SUMMARY

The paper presents a programme for monitoring granite structures of architectural monuments. The programme is of particular importance for St. Petersburg, where numerous historic monuments are built with the use of Finnish granite "rappakivi". The necessity of monitoring these monuments is demonstrated by the continuous supervision of the Alexander's Column, one of the largest monolithic granite structures in the world. Practical results of the implementation of modern flaw detection methods are illustrated by the reconstruction of the Petrovsky Bridge in Schlisselburg, near St. Petersburg.

RÉSUMÉ

L'article expose le programme de surveillance structurale des monuments en granit. Ce programme est d'une importance capitale à St-Pétersbourg qui abrite tant de monuments historiques construits avec du granit finlandais "rappakivi". La nécessité d'un tel contrôle est explicitée à partir d'un exemple, la colonne d'Alexandre qui est l'un des plus grands monuments en granit monolithique, dans le monde. A partir de la reconstruction du pont Pétrovsky à Schlisselbourg, près de St-Pétersbourg, l'auteur illustre les résultats pratiques obtenus par l'application de méthodes modernes de détection de défauts.

ZUSAMMENFASSUNG

Der Beitrag äussert sich zum Ueberwachungsprogramm für historisch bedeutsame Granitbauwerke. Dieses Programm ist besonders für St. Petersburg wichtig, wo viele Baudenkmäler aus dem finnischen "Rappakivi-Granit" bestehen. Die Notwendigkeit der Ueberwachung dieser Denkmäler wird am Beispiel der Alexander-Säule, eines der weltgrössten Granitmonolithe, aufgezeigt. Praktische Ergebnisse der Ausführung moderner Fehlstellensuchmethoden werden anhand der Rekonstruktion der Petrovsky-Brücke in Schlisselburg, in der Nähe von St.Petersburg, veranschaulicht.

1. INTRODUCTION

Granite is an intrusive igneous rock material, one of the most plentiful in the Earth's crust; it has been widely used in building since antiquity. Baltic granite "rappakivi" rich deposits are abundant in Finland, Sweden and in the north-west of Russia. Rappakivi is considered a valuable facing and structural material thanks to its porphyry-like structure with large-sized rounded orthoclase formations, rich gamut of colours and easy workability despite its high structural strength.

St.Petersburg, Russia's former capital (1712-1918), is internationally known for the extensive use of granite in its architectural monuments and ensembles. Granite became most popular here in the first half of the 19th century [1]: it was used for facing the embankments of the Neva, minor rivers and channels, bridge abutments; it was also widely used as structural material in cathedrals, palaces and private residences. Among the most spectacular examples of the rappakivi use are the 17m high antium columns of St.Isaak's Cathedral (Fig. 1) built in 1818-1858 to the design and under supervision of architect A.Montferrand; interior columns of the Kazan Cathedral (1801-1811, architect A.Voronikhin); columns of the Mikhailovsky Castle (1797-1800, architect V.Brenna), etc.

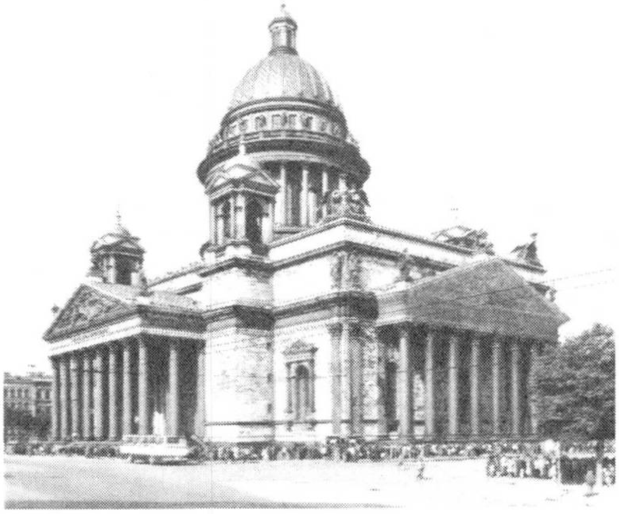

Fig. 1 St.Isaak's Cathedral in St.Petersburg

Granite is reasonably considered one of the most durable building materials. Yet there exist various environmental effects, both of climatic and technogenous origin, which gradually destroy it. Granite being a friable material, is subject to crambling, the most dangerous and destructive flaw for it. Noteworthy also is that crambling, generally of an orderly pattern, is inherent in granite, even in natural beds [2], and practically every large-sized granite element has cracks susceptible to extend in course of time. The history of monitoring the Alexander's Column throughout its lifetime provides a vivid example of the necessity for supervising crack extention in vital elements of constructions, especially those of architecture and historic interest.

2. ALEXANDER'S COLUMN: ERECTION, REPAIRS, DURABILITY FORECASTS

The Alexander's Column (Fig. 2a) was erected in the Palace Square in St.Petersburg in 1832 to the design of A.Montferrand in honour of Russia's victory over Napoleon. The fust of the column is made from pink rappakivi; being 25.6 m high and 3.6-3.2 m in diameter, it is one of the world's largest granite monoliths. It was cut from a boulder extracted from the same natural deposit in the neighbourhood of Vyborg as earlier the columns for St.Isaak's Cathedral. Even when selecting the monolith Montferrand was aware of the Vyborg granite tendency to crack (Finnish "rappakivi" means "rotten stone"), specifically to temperature fluctuations. But his confidence in the monument's durability was based on the absence of visible natural cracks in the monolith as well as on careful surface treatment by means of polishing and soaking with special compositions [3]. Nontheless, as early as in 1836, a few years after the erection of the column, its fust became a subject of concern due to the obvious expansion of surface cracks. In 1838 the Super-

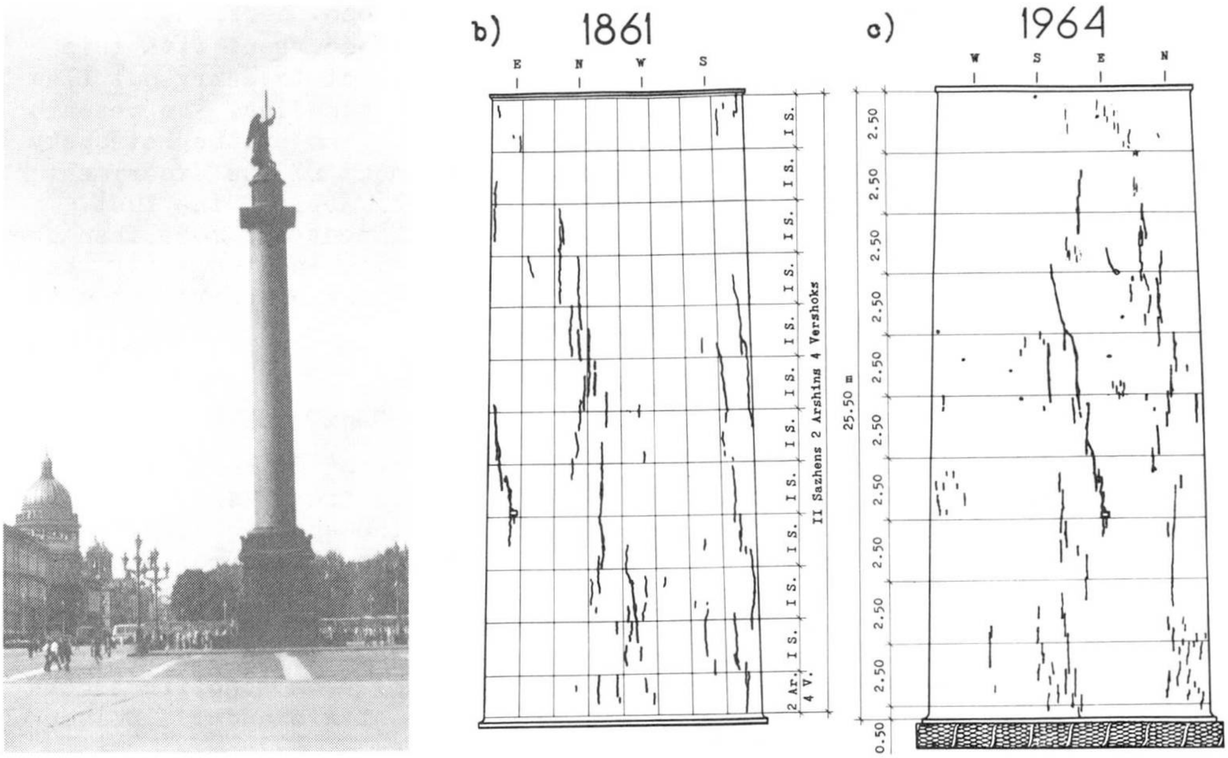

Fig. 2 **Alexander's Column :** a)- **general** view;
b,c)- granite fust **flaw fixing** drawings

vising Commission responsible for the erection of St.Isaak's Cathedral was charged with the research of these cracks, and later, in 1841, a special State Commission was set up for the examination of the column. In course of discussion, top-level Russian experts put forth quite contradictory predictions regarding the construction future. Among others, there was even a proposition to line the column with copper sheets in order to secure passers-by. Yet, largely due to Montferrand's high prestige, the Commission confined to recommendations on minor repairs of the monument.
In 1860 obvious degradation of the granite fust surface caused a new team supervised by colonel-engineer Pauker to turn to the rigidity of the monument; in doing so the commission discovered an error committed in 1841 - at that time no more or less objective data concerning the condition of the granite were fixed. In 1861-1862 photos of the column fust and a set of detailed drawings fixing its surface flaws were made (Fig. 2b, [4]). However these drawings also appeared unadequate for the analyses of crack expansion because they carried no information about the width of crack divergence. This new burst of public attention to the monument gave impetus to close research of the properties of rappakivi [5] and new debates about the expected moment of the column collapse [6]. This time the column fust was repaired more carefully, the cracks were cleared and filled with portland cement based composition.
In the early 1880-s another examination of the column was carried out by a commission of academicians and famous architects and engineers. Examination and restoration of the column were also taken in 1911, 1954 and in 1963-64. But despite permanent controversies as to the column's future [7,8], it was not until the last repair that a thorough drawing of surface flaws layout was performed. However, a comparison of said drawing with the one performed in 1861 (Figs. 2b,c) shows that its authors not only missed a unique chance to follow the crack expansion in the course of the elapsed century, but what is more, they obviously had no intention to make their drawing compatible with the former one as evidenced by the coordinate grid.

In October 1991 a team of mountaineers lead by M.Yu.Anosov performed an examination of the bronze angel crowning the column. Concurrent with this work the author of the present paper pioneered an attempt to carry out flaw inspection of the column's granite fust by ultrasonic sounding. The research was based on the method used in the studies of the columns of the Petrovsky Bridge in Schlisselburg. But lack of adequate background of the experiment made it possible to gain only a few data about a small area of the fust. Thus, despite tremendous unique importance of the monument and more than one-and-a-half-century debate as to its technical condition and longevity there has not been performed any thorough research with the use of current methods of flaw detection up to the present.

3. PETROVSKY BRIDGE IN SCHLISSELBURG: RESEARCH AND RECONSTRUCTION

The Petrovsky Bridge in Schlisselburg (in the vicinity of St.Petersburg) resting on granite columns provides a unique sample of the extention of civil architecture order to bridge architecture. In the epoch of classicism there were built no more than a few bridges of the type all over the world, and the Petrovsky Bridge is the only one that has survived to the present day [9]. The bridge was built in 1820-30-s after the project of French architect P.Bazin; 18 columns and 8 semi-columns of the bridge abutment (Fig. 3) were made by a famous stone-cutter S.Sukhanov from monolith blocks of pink rappakivi structurally allied to the granite of Alexander's Column and the columns of St.Isaak's Cathedral.
Reconstruction of the bridge in 1988-90 revealed notable crambling of the fusts of the columns which cast serious doubt upon their longevity and caused another more sophisticated study of their condition. Ultrasonic sounding detected regular non-uniformity ("orthotropy") of the column material in reference to the velocity of the ultrasonic signal passing through it (Fig. 4), said non-uniformity being much more obvious than that found in the literature [10,11]. The phenomenon is associated with peculiar to the granite monolith mutually parallel microcracks which tend to spread under external loads and other actions in course of time.
To estimate such crambling effect on the overall rigidity of a column, samples of granite identical to the material of the columns were subjected

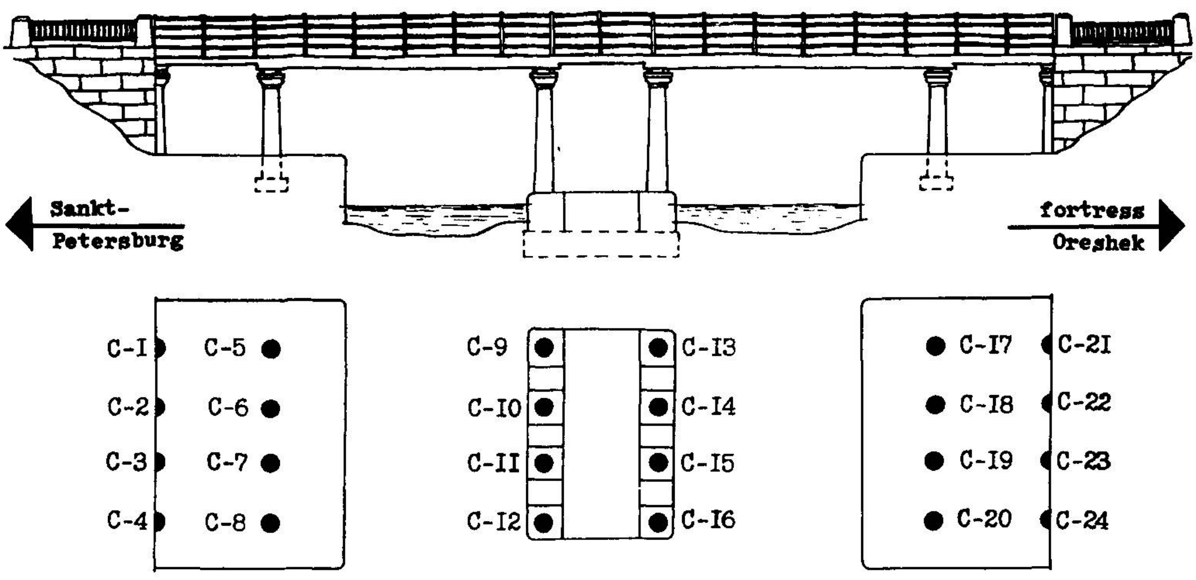

Fig. 3 Petrovsky Bridge in Schlisselburg: construction scheme

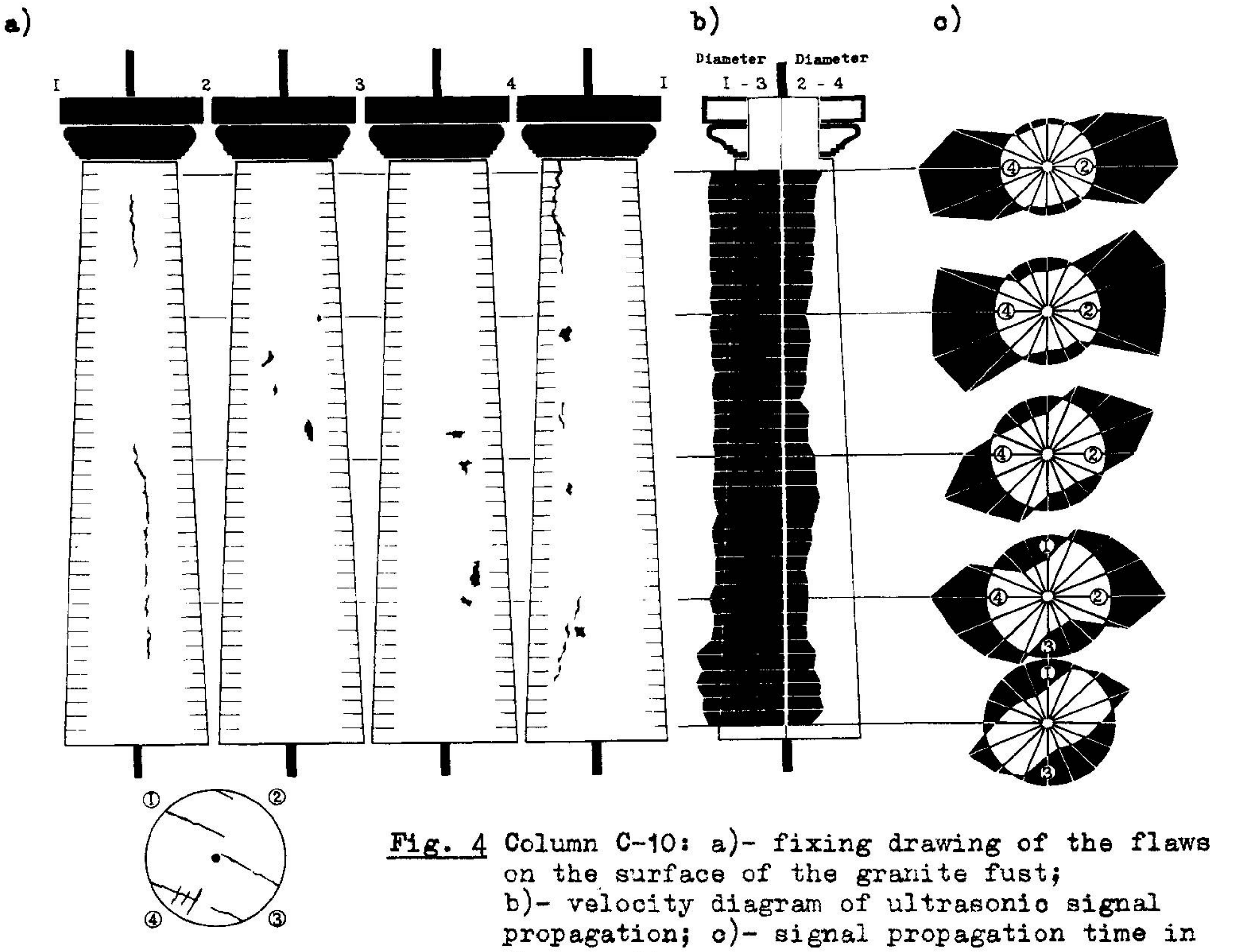

Fig. 4 Column C-10: a)- fixing drawing of the flaws on the surface of the granite fust; b)- velocity diagram of ultrasonic signal propagation; c)- signal propagation time in fust sections

to laboratory mechanical testing, and besides, a full-scale testing to local failure on column end surfaces was performed. The analysis of thus obtained data lead to the conclusion that 23 out of 24 columns could remain valid and only one that had a through lateral crack was substituted. The heads of the columns were strengthened by means of special-purpose casings enclosed in hollow cast iron caps, and in addition, the original design of the bridge span support on the columns was revised in order to reduce bending moments transmitted to them [12,13].

The results of the investigation enabled conservation of the granite elements of the unique historic construction as well as possibility of taking adequate steps regarding their future service.

4. CONCLUSION

The present paper is aimed at substantiation of the development and execution of a wide-scale program for monitoring major bearing granite elements of historic constructions with the use of currently-available flaw detection methods, specifically ultrasonic ones.

Structure disruption (crambling) is the most hazardous for the rigidity of material as friable as granites. On the other hand, essentially every large granite monolith has more or less tangible primary ("born") cracks. Finnish granite rappakivi is particularly typified by crambling. That is why granites unlike other building materials need not only appropriate detection of inner flaws but also feasibility of their evaluation in time. For this

purpose, a system of basic data readily correlated with the results of subsequent measurements should be built up. Such correlation would make it possible to decide adequately whether the construction needs repair, strengthening or substitution of individual elements before the onset of an emergency situation. Another line of this work should involve thorough strength investigation of granites with inner flaws, and revealing the dependence of granite strength on such flaws qualitative and quantitative parameters.
The importance of such research is dictated by high architecture value of numerous historic buildings and constructions with granite principal bearing elements. In this regard St.Petersburg presents a striking evidence. Experience has shown that the problem can be solved on base of up-to-date technical facilities.

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