Zeitschrift: IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht

Band: 13 (1988)

Rubrik: C. Inspection, assessment and maintenance

DAILY THEME C

Inspection, Assessment and Maintenance

Surveillance, évaluation et maintenance

Ueberwachung, Zustandsbewertung und Unterhaltung

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Das Thema wird von zwei eingeladenen Referenten eingeführt, deren Referate im Schlussbericht des Kongresses veröffentlicht werden. Dieser Schlussbericht wird den Teilnehmern nach dem Kongress zugestellt.

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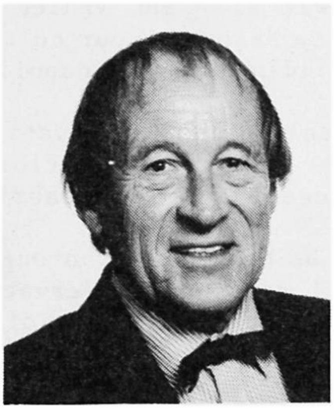
Maintenance Techniques for Historic Building Facades

Technique de maintenance pour des façades de bâtiments historiques

Unterhaltungstechniken für Fassaden historischer Gebäude

Sven. E. THOMASEN
Senior Consultant
Wiss, Janney, Elstner Assoc.
Emeryville, CA, USA



Sven Thomasen has a structural engineering degree from the Technical University of Denmark. While with WJE, he has investigated building failures and material deterioration. He has taught at California Polytechnic in San Luis Obispo and has designed many large projects.

Carolyn L. SEARLS
Senior Engineer
Wiss, Janney, Elstner Assoc.
Emeryville, CA, USA

Carolyn Searls has a BS in Architectural Engineering from Cal Poly, San Luis Obispo, a MS in Structural Engineering from UC Berkeley and has attended the architectural conservation course at ICCROM. She has done field investigations and analysis of structural failures and evalution and rehabilitation of masonry, marble and terra-cotta facades.

SUMMARY
The design of a maintenance program for a historic building facade must be based on a technical understanding of the building as well as sound conservation principles. A thorough investigation of the building's history and existing condition is made, utilizing non-destructive testing and monitoring equipment. The selected maintenance procedures should be compatible with the building, should not damage the existing historic fabric, and should be reversible. Regular maintenance and regular periodic inspections will prolong the life of the building facade and reduce major repair costs.

RÉSUMÉ
Le programme de maintenance pour une façade de bâtiment historique doit être basé sur l'appréciation technique du bâtiment de même que sur des principes de conservation reconnus. Une étude détaillée de l'histoire du bâtiment et des conditions actuelles est réalisée à l'aide de méthodes d'essais non-destructifs et d'un équipement de mesures. Les procédures de maintenance choisies doivent être compatibles avec le bâtiment, ne doivent pas endommager le tissu historique existant, et doivent être réversibles. Une maintenance régulière et des inspections périodiques prolongeront la vie de la façade du bâtiment et réduiront les coûts majeurs de réparation.

ZUSAMMENFASSUNG
Das Programm für die Unterhaltung der Fassade eines historischen Gebäudes beruht auf dem technischen Verständnis des Gebäudes und der Anwendung erprobter Konservierungsmethoden. Mit Hilfe zerstörungsfreier Prüfungen wurden Gebäudezustand und -geschichte festgestellt. Die ausgewählten Unterhaltungsmethoden sollen reversibel sein und auf identischen Materialien basieren. Regelmässige Unterhaltung und periodische Zustandsaufnahmen verlängern das Leben der Gebäudefassade und verringern die Reparaturkosten.

1. INTRODUCTION

Historic buildings in our cities must be preserved for future generations because of their cultural heritage and because they represent a valuable economic resource. The current philosophy of historic preservation is based on the "Venice Charter," a resolution adopted by one of the first international conferences on historic buildings held after World War II. The Venice Charter states that it is the responsibility of all people to safeguard our cultural resources, and to maintain and preserve the historic buildings in as unspoiled state as possible.

Regular maintenance is critical to the conservation of historic buildings. When maintenance is neglected the building deteriorates at an accelerated rate, resulting in loss of irreplaceable historic fabric.

The maintenance program must be based on a thorough understanding of the building systems and materials as well as sound conservation principles. A maintenance program should be based on documentation of the building's history and investigation of existing condition. Successful implementation of a maintenance plan requires financial and time commitment on the part of the building owner and well trained maintenance personnel.

2. PRINCIPLES FOR MAINTENANCE

One objective of preservation maintenance is to reduce deterioration and to increase the service life of the historic fabric. The following guidelines, some of which were adapted from I. Holmstrom and C. Sanstrom of the National Swedish Institute for Building Research [1], are suggested for planning a maintenance program:

1. A historic building must be given constant supervision and maintenance. The maintenance must begin before the deterioration has progressed to the point of requiring major repair.

2. The initiation of a maintenance program must be proceeded by a survey of the history of the building and its technical problems.

3. The design of a maintenance program should account for the unique features of the building, the available preservation technology, and the expertise of the maintenance personnel.

4. Maintenance procedures should be based on respect for the original building. It is preferrable to make repairs using original materials and methods because such materials behave and age in the same way as the surrounding fabric. Each intervention affects the building, sometimes in ways unknown, and all interventions should therefore be reversible, that is they can be removed without harm to the surrounding fabric. Minimum measures are often the most prudent.

5. Knowledge of the long-term effect of any treatment is important and techniques for removal or retreatment should be considered before an intervention is made. For example, while a modern paint may be easier to apply or last longer, it is very difficult to remove, and a traditional oil paint would be less expensive and less harmful to the substrate in the long run. The life span of any new component or treatment must be considered in relation to the other components it is attached to.

6. Structural retrofits must be compatible with the existing load carrying system of the building. Any change in the structural system of the building may have long-term unforseen consequences.

7. Complete documentation of the investigation, reasons for selecting materials, procedures for the interventions and their results are invaluable information for future repairs and maintenance.

3. INITIATING A MAINTENANCE PROGRAM

3.1 Background Information Gathering

Historical records such as old drawings, photographs, or journals often provide documentation on the original design and on the construction history of the building. Old photographs may show items such as cornices or gutters which are altered or perhaps missing. Maintenance records should be examined and personnel interviewed regarding past repairs and their performance. All information should be verified on site, since buildings were sometimes not constructed according to the plans.

Drawings can often be obtained from the building owner or local building authorities. Other sources of information are historical societies, libraries, collections of the original architect's work, and owner's receipts for materials and repairs.

3.2 Visual Condition Assessment

Each material to be maintained is identified and examined, recording its condition and whether it is original, repaired, altered, or a replacement material. Careful attention should be paid to inspecting roofs, cornice gutters, joints, basement walls and other sources of water infiltration, one of the most damaging elements to facades.

Visual observations are made of the materials and the structural elements and signs of distress, such as cracks, spalling, rust spots or evidence of leakage, are recorded on elevation drawings and documented with photographs. The drawings are studied to detect failure trends. Close-up examinations are made at representative areas and at ornamental building sections such as cornices, watertables and finials where fractures and distress might be difficult to observe from a distance and where failure often represents a considerable hazard. The inside of a wall can be examined and photographed with a fiberoptic borescope which requires drilling only a 12mm diameter hole into the wall. The viewing field end of the rod is inserted through the small hole into the wall and the interior of the wall can be viewed by looking into the eyepiece.

3.3 Nondestructive Tests

Nondestructive tests are used when more information about the wall is required than can be obtained using visual examination techniques. When evaluating a masonry or stone wall, the location of metal anchors and ties is important in determining the ability of the wall to resist wind and seismic loads. Such embedded metal items can be located by a pachometer which is a metal detector that measures the magnetic field of the wall surface. The pachometer is most useful when the metal is fairly close to the surface which is normal for shelf angles but not always for metal anchors. Gamma radiography and x-ray radiography can also be used to locate embedded steel but both these methods are expensive and somewhat cumbersome to use at a building facade.

The pachometer locates the reinforcing bars, but it provides no information about the condition of the steel such as the presence of corrosion. The copper-copper sulfate test will detect active corrosion in the embedded steel by measuring the voltage drop between the steel and the surface of the concrete.

A moisture meter can help locate wall areas subject to deterioration from water. Readings should be verified by obtaining samples for laboratory analysis.

Ultrasonic testing is used to detect cracks, flaws, or voids in materials such as concrete, masonry, or steel. A low frequency wave is sent from a transducer through a material to a receiver, and the travel time of the wave is recorded. The presence of weak areas, voids, or cracks is observed as a delay of the velocity and travel time of the ultrasonic wave.

The location of internal cracking or delamination often can be found by tapping the surface with a rubber or wooden mallet. The sound made by the mallet is different on the debonded material than on the undamaged wall.

3.4 Monitoring Systems

The behavior and movement of a building is often monitored either continuously or intermittently to determine the cause of observed distress or to help in selection of appropriate repairs. For example, cracks in building walls may have both daily and seasonal changes in width as well as long-term growth. Before repairing a cracked wall, it is useful to know the cause of the cracking and whether the cracks are still moving. Inexpensive plastic "tell-tale" gages installed across the crack can be read periodically to measure changes in the crack width.

Continuous monitoring of cracks can be made using a gage which records crack movements as scratches on a replaceable brass button. The scratches on the button are read with a calibrated microscope. More sophisticated readings can be obtained by installing a small "bridge" with a built-in electronic transducer across the crack. The "bridge" magnifies the crack movement and the transducer, when connected to an automatic recorder, can provide continuous monitoring. The system can be programmed to sound an alarm when unusual crack movements occur.

Monitoring of material deterioration is done, especially in areas with industrial pollution, on original and restoration materials. One method commonly used is photography, either in the form of simple photographs, rectified photographs or photogrammetric mapping from which complete drawings of surface contours can be made. The periodic measurement of the facade should be augmented by monitoring of air quality and meteorological events and the recording of special conditions that affect the site, such as humidity, temperature and hours of sunshine.

Other types of monitoring systems are designed to record wind-induced sway of high-rise buildings, measure vibrations from traffic, or obtain response characteristics of the structure from measurements of induced dynamic forces.

3.5 Field Tests

Field tests are used as investigative tools to assess safety or performance of existing building elements, to establish strength of building materials, and to evaluate proposed methods of repair.

Water permeance of masonry walls is measured in a field test using a frame attached to the exterior wall while air pressure and water spray are applied from nozzles inside the frame. Strain relief tests are used to measure the amount of accumulated built-up strain in masonry facades by attaching electrical resistance strain gages to the face of the wall, reading the gages, cutting out the segment of wall to which the gages are attached and again reading the gages.

It is often desirable to test maintenance procedures on a small portion of the structure before treating the entire building. When making the test, the procedures and conditions should be as close as possible to those which will be used on the entire facade. If the test involves patching or coating it should be allowed to weather before final selection is made. Evaluation criteria must be established for judging the acceptability of the tested procedure.

3.6 Laboratory Analysis

Laboratory analysis is an essential part of many investigations and is performed both on specimens from existing walls and on proposed repair material. Specimens from the walls are cut out and brought to the laboratory where they are prepared and tested. The cutting, transporting, and preparation of samples call for great care in order not to disturb the material. This type of testing is generally only feasible for small specimen sizes.

Commonly performed laboratory tests include standard tests for compressive strength, shear, modulus of rupture, and cyclic tests in which a reversible load or simulated environmental exposure is applied to the specimen and the performance is monitored through many load cycles. Laboratory tests are also performed on proposed repair materials. Their composition and compatibility with existing materials are analyzed and their long-term performance and durability are evaluated by accelerated weathering tests.

4. DESIGN OF A MAINTENANCE PROGRAM

Design of the maintenance program, which is defined in a Maintenance Manual, should consider the following items [2,3]:

1. Documentation: Information gathered during the planning of a maintenance program must be documented and these documents will be included in the maintenance manual. In addition, provisions should be made for recording all future inspections, maintenance and repair work on the building.

2. Building history: A brief history of the building and its past maintenance as obtained from the Background Information Gathering should be included. Copies of all known documents should be included or should be referenced.

3. Condition assessment: The physical structure and building materials are described based on results from visual observations and field and laboratory testing. The description should be updated during subsequent inspections.

4. Inspections: Routine inspections by maintenance personnel are undertaken at regular intervals and sometimes on a daily basis. Periodic inspections of the facade should be performed by an architect or a structural engineer. The inspection interval, depending on the building condition, should not exceed five years. The results of each inspection should become part of the maintenance manual.

5. Treatment specifications: The manual should specify treatments, their location, frequency of the treatment and method for

evaluating the success of the treatment. The criteria for applying treatment should also be included. The specifications should list all materials, list of suppliers and any specialized tools required. A summary of maintenance treatment for the entire facade should be included together with forms to record dates and treatments of each building component.

6. Personnel: The names and contacts of all consultants and contractors and a description of their scope of work on the building should be listed.

7. Maintenance budget: The cost of regular maintenance and of periodic maintenance should be documented in the maintenance manual to help management in budgeting funds necessary to perform the maintenance program.

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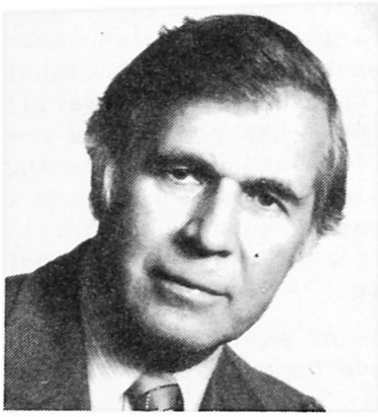
Bridge Decks, Joints and Bearings Assessment and Maintenance Methods

Méthode d'evaluation et d'entretien des tabliers, des joints de dilatation et des appuis de ponts

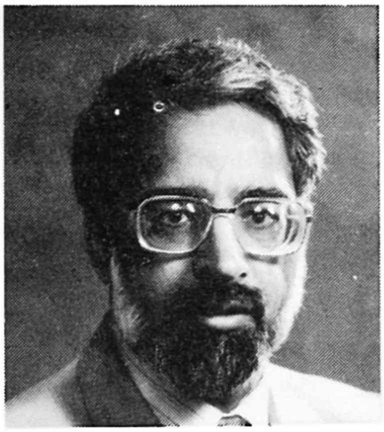
Beurteilungs- und Instandhaltungsmethoden für Fahrbahnplatten, Dehnungsfugen und Lager

Roger DORTON
Manager, Structural Office
Min. of Transp. of Ontario
Downsview, ON, Canada



Roger Dorton received his Ph.D. from the University of Nottingham in 1954. He was a consulting engineer specializing in long span bridges before joining the Ontario Government in 1972. He is chairman of the Ontario Highway Bridge Design Code Committee, and chairman of the Canadian Group of IABSE.

Ranjit REEL
Head, Bridge Management Section
Min. of Transp. of Ontario
Downsview, ON, Canada



Ranjit Reel received his civil engineering degree at the University of Glasgow. He has been with the Structural Office for the last twenty years, and is now responsible for setting policies for rehabilitation of bridges, and for reviewing contract documents for rehabilitation projects.

SUMMARY
Bridge decks, expansion joints and bearings have been the highway bridge elements most susceptible to deterioration from the application of winter deicing salts. Extensive investigation and testing in Ontario have produced methods to assess damage and more effectively maintain bridges. This experience has led to changes in new construction to prevent a repetition of the rapid deterioration evident in many bridges built 20 to 30 years ago.

RÉSUMÉ
Le tablier des ponts, les joints de dilatation et les appuis ont toujours été les éléments des ponts-routes les plus susceptibles de se détériorer du fait de l'utilisation du sel d'épandage en hiver. Des recherches et des expériences extensives ont éte faites en Ontario et ont abouti à la mise au point de méthodes d'évaluation des dommages et de protection plus efficace des ponts. Cette expérience a conduit à modifier la construction des ponts afin d'éviter que ne se produise une détérioration rapide comparable à celle des ponts construits il y a vingt ou trente ans.

ZUSAMMENFASSUNG:
Brückenfahrbahnen, Dehnungsfugen und Lager sind am Meisten dem vorzeitigen Verschleiss und Beschädigungen ausgesetzt, welche durch das im Winter verwendete Streusalz gegen Vereisung der Fahrbahn hervorgerufen wird. Durch weitgehende Untersuchungen wurden in Ontario neue Beurteilungsmethoden entwickelt, welche zur frühzeitigen Entdeckung von Schäden und zur rechtzeitigen Reparatur führen. Die auf diesem Gebiet gemachten Erfahrungen haben auch zu neuen Konstruktionsmethoden beim Bau von neuen Brücken geführt, um eben diese Schäden von Anfang an zu verhindern.

1. INTRODUCTION

Following the large expansion of highway facilities in North America in the 1960's, many of the highway bridges built at that time and since have shown unanticipated early deterioration. This is particularly true in northern areas where there has been extensive winter salting, such as in Ontario. The component most susceptible to this deterioration has been concrete bridge decks, but due to leaking expansion joints many bridge bearings and bridge seats have been similarly affected.

The problems with decks, joints and bearings have been the subject of extensive research and testing in Ontario to determine improved maintenance and assessment methods, as well as rehabilitation techniques and improved new design and construction procedures to ensure greater durability. The Ministry of Transportation of Ontario (MTO) has produced a new bridge design code [1] addressing improved durability, and has an ongoing committee on durable structures to ensure early implementation of new technology. A bridge deck rehabilitation manual has been issued [2] and all deck joints and bearings undergo extensive laboratory and field testing before being accepted and placed on a designated sources list.

This paper describes methods of assessment and maintenance for concrete decks, joints and bearings that have become established practice in the Province of Ontario.

2. DECKS

2.1 Background

In the past, the lack of care in achieving the specified cover to the top layer of reinforcement in deck slabs, the impracticality of putting four layers of steel into the 190 mm deck slab, or the use of improper waterproofing membranes and exposed decks has led to severe deterioration in decks, sometimes in less than ten years from the time of construction.

2.2 Condition Assessment Methods

The first indication that there is a problem with bridge decks usually comes through the biennial inspections. These, however, may not provide sufficient information so general surveys, included chain dragging of the bare concrete decks and limited coring of both bare and asphalt covered decks, may be carried out to establish the extent of the problem, set policies and develop rehabilitation programs.

Detailed deck surveys are also carried out on decks that are on a road paving contract so that the condition of the decks can be assessed and any necessary repairs designed to coincide with the paving contract.

Detailed deck condition surveys[2] include:
- Delamination survey with a heavy chain.
- Corrosion potential survey with a copper copper-sulphate half cell.
- Concrete cover survey using a pachometer (covermeter).
- Concrete coring, and asphalt sawn samples.

The assessment of decks by radar and thermography, DART,[3] gives information on asphalt thicknesses, scaling and delamination of concrete, and concrete cover to reinforcement. This information can be collected rapidly, and with some additional information obtained through coring and half cell surveys will, in future, reduce the time needed for condition surveys.

2.3 Maintenance Practices

Washing of bridge decks, expansion joints, bearings and other components exposed to salt spray is carried out at least once every year on all bridges. Minor repairs are undertaken as required by the bridge crew.

2.4 Rehabilitation Practices

The options for concrete deck rehabilitation used in Ontario are:
- Patch, waterproof and pave.
- Cathodic protection.
- 60 mm of normal slump concrete overlay with waterproofing and paving.
- 50 mm of latex modified concrete overlay.

The selection of the rehabilitation method is based upon practical, economic and technical considerations. The decision matrix for selection of deck rehabilitation method based upon technical consideration is given in Table 1. Quite often the selection criteria does not yield any one type of rehabilitation. In such cases judgement has to be used to arrive at the best solution.

Criterion	Concrete Overlay	Waterproof and pave	Cathodic Protect.
-Delamination and spalls exceeding 10% of the deck area.		No	No
-Corrosion potential more negative than 0.35V over more than 20% of the deck area.		No	
-Moderate or heavy scaling exceeding 10% of the deck area.		No	No
-Active cracks.	No		
-Remaining life less than 10 years.	No		No
-Concrete not properly air entrained.			No
-Complex deck geometry.	No		
-Limited load capacity.		No	No
-Electrical power unavailable.			No
-Epoxy injection repairs.			No

Table 1 Decision Matrix for Selection of Deck Rehabilitation Method

2.5 New Construction

Durability of new concrete construction is assured by a combination of strategies [4],[5].
- The thickness of concrete slab decks has been increased to 225 mm which allows for the tolerances of placing the four layers of reinforcement and still maintain the specified minimum covers to the reinforcement.
- The concrete cover to the top surface of the deck has been increased to 70 $\pm$ 20 mm. Also, the screeds are adjusted to maintain this cover as checked through a dry run of the finishing machine.
- Epoxy coated reinforcing steel is used in surfaces exposed to salt application or spray.
- All concrete decks are waterproofed and paved with asphaltic concrete.
- Details at expansion joints have been improved and all transverse and longitudinal joints are sealed.
- Drip details have been improved.

3. EXPANSION JOINTS

3.1 Background

Thirty years ago most expansion joints were of the open type. With the development of so-called sealed system, these have been used almost without exception to try to prevent corrosive run-off reaching the bearings, bridge seats and other components. Unfortunately these systems have rarely been fully watertight in practice, and the gradual seepage of brine without the flushing effect of summer rainfall has perpetuated the same concerns regarding corrosion.

The ride quality when crossing expansion joints has been an ongoing problem, as has the anchorage of joint components, and traffic disruption caused by joint maintenance and replacement. In an effort to improve joint quality and durability these questions have been addressed by MTO through design provisions in the Ontario bridge code [1], improved maintenance practices, extensive condition assessment of existing joints and testing of new products.

3.2 Condition Assessment

All joints are assessed in the detailed inspection which take place every two years and recorded on the inventory system. Problems between inspections usually become readily apparent by poor ride quality, or evidence of excessive leakage. A survey of expansion joint performance on selected bridges has led to the elimination of elastomeric compression seals as an acceptable sealing system.

The assessment of various methods of anchoring the steel components has led to a standardized system using reinforcing bars cast into the deck (Fig.1). This figure shows the joint system cast into a substantial concrete dam with steel angle armouring. This system has been found to be less susceptible to failures of the joint and adjacent pavement.

Various types of proprietary sealing systems may be used between the steel angles shown (Fig.1) but they have to be approved before being placed on the MTO designated sources list[6]. Approval is only granted after review of detail drawings, laboratory testing of seal material and joint movement, followed by a trial installation and performance assessment after one year in service. The trial installation, and all new joint construction, is subjected to a water leakage test before opening the bridge to traffic. This test is performed by ponding water across the joint through the use of temporary dams, and checking for leakage below the joint. This test is considered to be one of the most important procedures introduced to improve performance, as evidenced by frequent leakage when joints are first installed and tested.

3.3 Maintenance Practices

Sealed joints are cleaned out periodically, but there is usually an accumulation of sand and debris that can inhibit the free closure of the joint in hot weather. In cases of extreme wear or damage, the seals may be replaced as a maintenance item. To minimize possible leaks at jointing of seals, full length seals are preferred.

To facilitate cleaning of any salt water leakage through the joint, a minimum gap of 200 mm is specified between the deck and the face of the abutment wall (Fig.2). All construction formwork must be removed from this gap.

3.4 Joint Selection

The designated sources list[6] has a number of approved joints, but the most widely used are strip seal joints anchored in concrete. The joints in this category have been placed in a number of classes, according to the various seal clamping methods used. Based on the asessment and field performance results, guidelines have been prepared showing the suitability of the joint classes for various highway classifications[7]. For high volume freeway applications, classes using a heavy horizontal steel plate to clamp the seal are preferred (Fig.1). For the most severe applications the press fit seal retainer class is not used. The guidelines, which are for the use of the designer, also list other characteristics, both positive and negative, such as robustness, installation cost, seal retention, ease of installation and replacement, and access for inspection.

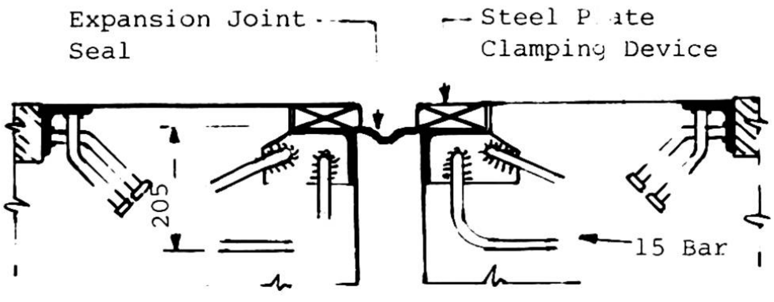


Fig. 1 Expansion Joint Anchorage

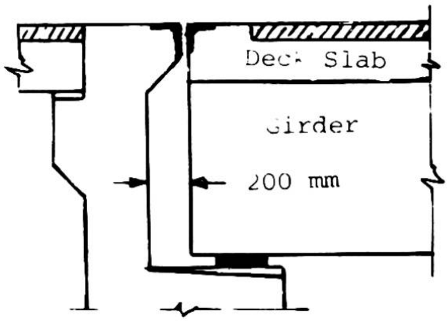


Fig. 2 Detail at Abutment

4.0 BEARINGS

4.1 Background

Simple bearings such as sliding plates, rockers and rollers were widely used in Ontario until the mid 1950's. These types were particularly susceptible to corrosion and seizure with the open expansion joints prevelent at that time. Since the introduction of steel laminated elastomeric bearings in 1957, this type has been commonly used.

In the early 1960's the newly developed pot bearings proved to be the preferred type for the single round columns which were extensively used for post-tensioned concrete construction. This type of construction has become the first choice for expressway structures, and TFE lined spherical bearings and polyurethane disc bearings are now also used in this type and other long span structures.

4.2 Condition Assessment

Bridge bearings are inspected every two years on a regular basis, but in 1986 the MTO carried out a special assessment to cover all types of bearings in use at that time. The results of this study have been published[8], and will have a significant affect on bearing design, selection, and maintenance in Ontario.

Unconfined elastomeric bearings have been used for 30 years and their satisfactory performance was confirmed by this study. Elastomeric bearings have been subject to testing of materials in the laboratory and samples taken at the site for quality assurance testing. There have been many failures, particularly of bond between laminates during these pre-installation tests.

The pot, spherical and disc bearings are more complex in their manufacture and operation than the elastomeric bearings. As might be expected several types of deficiency and failure were observed. The study lists eleven types of defect on pot bearings and TFE sliding surfaces, with possible causes and corrective action to be taken. Largely as a result of this report, the Ontario specifications for bearings have been re-written [9].

The MTO uses proprietary type bearings which are prequalified and placed on the designated sources list. This list is presently being revised following new bearing qualification methods. To be designated, a bearing manufacturer must submit shop drawings and design calculations for approval, a sample of elastomer for testing, and finally a sample bearing for evaluation.

4.3 Maintenance Practices

Bearings have not been cleaned well in the past, and steel parts have corroded badly. To facilitate general cleaning, a 200 mm gap is provided to the abutment wall (Fig.2). A higher degree of corrosion protection is now specified for new bearings, and bearings must be replaceable without damage to the structure. To assist free draining around the bearing, inclined bridge seats are now specified (Fig.2).

5. CONCLUDING REMARKS

The methods for carrying out condition surveys and asessment of decks in Ontario should ensure a better selection of rehabilitation methods than in the past. The emphasis on durability in design codes should provide a longer life for new deck construction.

Proprietary expansion joints and bearings are used, but performance specifications or guarantees are not considered practical at this time. Instead, comprehensive requirements are given in the design code and the material and construction specifications, with prequalification of suppliers by extensive testing and inspection. Combined with improved maintenance methods, this approach is expected to provide more durable joints and bearings to combat the corrosive Ontario environment.

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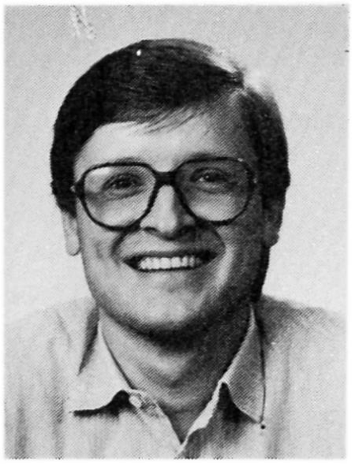
Condition Assessment of Facades with Infrared Camera

Auscultation des façades à l'aide de la thermographie infrarouge

Zustandsaufnahmen von Fassaden mit Infrarot-Thermographie

Harri KAASINEN
M.Sc. (Eng.)
Techn. Res. Centre of Finland
Espoo, Finland



Harri Kaasinen, born 1957, received his master degree at the Helsinki University of Technology. For two and half years he was structural designer in a consulting firm. Now he is a researcher and involved in building physics.

SUMMARY
Infrared thermography is widely used for localization of air leakages, cold bridges and other thermal failures in the building envelope. In this article several new applications are proposed. The results of some investigations carried out in the laboratory and field are presented. The problems of outdoor investigation are also discussed.

RÉSUMÉ
La thermographie infrarouge est largement appliquée pour localliser des bulles d'air, des ponts thermiques et d'autres défauts thermiques de l'enveloppe du bâtiment. De nouvelles applications sont proposées. Les résultats de quelques recherches de laboratoire et dans le terrain sont présentés. Les problèmes de recherches à l'extérieur sont également discutés.

ZUSAMMENFASSUNG
Infrarot-Thermografie wird allgemein eingesetzt für die Feststellung von Leckagen und anderen wärmetechnischen Mängeln der Aussenschale eines Gebäudes. In diesem Beitrag werden zahlreiche neue Einsatzmöglichkeiten vorgestellt, neben Untersuchungsergebnissen von einigen Labor- und Felduntersuchungen. Verschiedene Probleme der im Freien durchgeführten Untersuchungen werden ebenfalls erläutert.

1. BACKGROUND

Infrared radiation is part of the electromagnetic wave spectrum. It is also known that every object with a temperature higher than absolute zero kelvin (-273 ^0C), emits, reflects and absorbs electromagnetic radiation at its surface. The amount and wavelength of the emitted radiation are dependent on the object's surface temperature and its ability to emit energy, called emissivity.

Infrared camera is a device which responds to radiation emitted and reflected from object's surface and converts this radiation into black and white or colour image. Variations in image brightness or colours indicate varying levels of radiation. Most infrared cameras are sensitive to wavelengths of 2 - 5,6 μm or 8 - 12 μm. Wavelegths of 0,38 - 0,78 μm are detectable by the naked eye, which means that the human eye cannot see the light to which the infrared camera is sensitive and vice versa.

2. APPLICATIONS OF IR-THERMOGRAPHY FOR CONDITION ASSESSMENT IN BUILDINGS

2.1 Current applications

Infrared thermography has in recent years become an important method for detecting air leakages, cold bridges and other thermal failures in the building envelope. Another popular application of IR-thermography has been to use it as a nondestructive method to detect wet insulation in flat roofs.

There are several possible new applications proposed in many articles. Principally the IR-camera could be used for investigation of any kind of effects in which temperature differences exist or occur.

2.2 Delamination of layers

Delaminated areas in the surface of the structure emit infrared radiation different from that of sound areas, because heat transfer from or to the inner structure is interrupted. As an example some asphalt autobahn decks and concrete bridge desks have been investigated to localize possible delaminations [1], [2].

As an other example, thermography has been used to assess the conditon of walls bearing ancient paintings. Paintings were heated prior to the test and delaminations were detected with an IR-camera.

2.3 Moisture penetration

Moist surfaces demand heat for drying and therefore show a lower surface temperature than dry areas. This phenomenon is emphasized if a radiator is used. In this case radiator and camera are positioned in such a manner that radiaton from the radiator is reflected on the surface of the body to be investigated and then arrives at the camera. If there are moist areas at the surface, they absorb more infrared radiaton than dry ones. Therefore the camera senses a lower level of reflected radiaton. This method is called infrared reflectography and it has been used to detect moisture on masonry [1], [3].

3. TESTS

3.1 Laboratory tests

3.1.1 Threshold moisture content

The lowest moisture contents of some building materials that IR-camera can detect were determined. Thermal images of specimens with different moisture contents were taken. Moist specimen emits less radiation than a dry one, and this difference can be detected with the IR-camera. The lowest difference which can be measured is called threshold moisture content.

Specimens made of concrete, cellular concrete, timber (with and without painting) and brick were tested. The threshold moisture values of all these materials were within the hygroscopic moisture range of that material. At the same time the effect of IR-radiators were tested. The radiator emphasized differences between moist and dry surfaces, but it may be too complicated to use in field tests.

3.1.2 Test walls of brickwork

Thermal images of some plastered test walls were taken in order to find delaminations between plaster and bricks. Specimens were kept in cold conditions at -20 ^{0}C temperature before taking the infrared images. First warm water was sprayed on the surface of the specimen to emphasize temperature differences. Later the same effect was created with the aid of fan. Both methods worked well and delaminated areas are visible in figures 1 and 2.

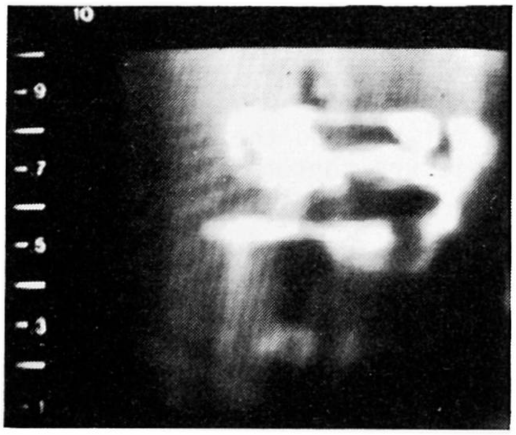


Fig. 1. Delamination of plaster, sprayed warm water.

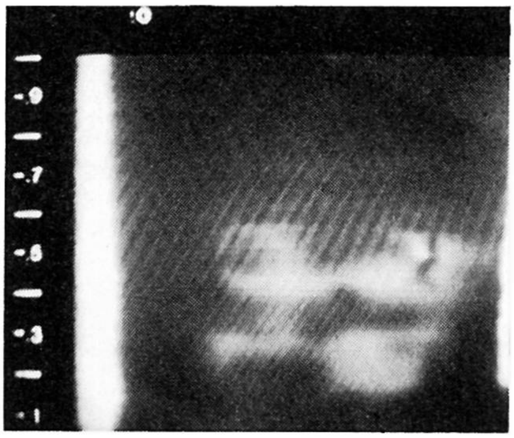


Fig. 2. Delamination of plaster, warm air fan.

Another interesting case was to study the rising moisture in masonry wall. A test wall was constructed of old bricks and laid in a water tank. Thermal images were taken before (fig. 3) and after (fig. 4) injection. Moisture penetration limits and the effect of injection are clearly seen in figures 3 and 4.

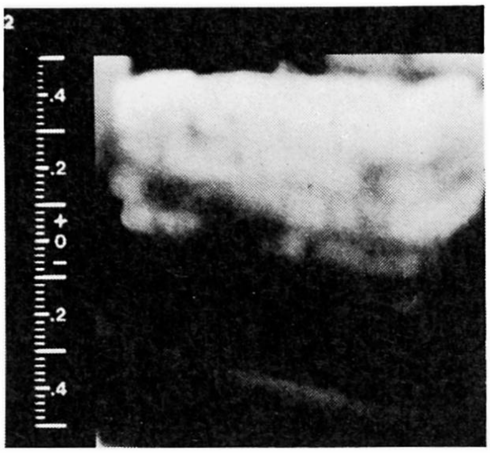


Fig. 3. Moisture penetration before injection.

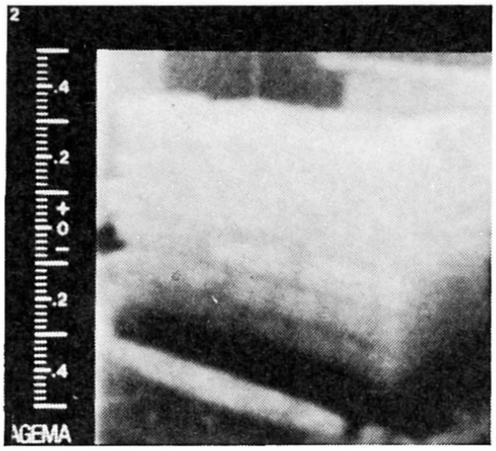


Fig. 4. Moisture penetration after injection.

3.2 Field tests

There are several restrictions to and distrubances in outdoor infrared testing. The temperature of air and structure are changing continuously as sun radiation changes in direction and intensity, shadows must be taken into consideration etc.. On the other hand if we, e.g., need to localize delaminations in facades by means of IR-camera there must be some kind of energy source which creates temperature differentials on the surface. Great care must be taken when interpreting thermal images, because thermal differentials could be caused by variations in sun radiation or reflections and not by any delamination.

As an example the facades of a supermarket were investigated. Some of the ceramic tiles had come loose and dropped on the street. Thermal images of facades were taken in order to localize possible loose tiles. The investigation was conducted in spring when the nights were cold but during the day the facades were warmed by the sun. Some posible loose tiles were found (fig. 5). These areas were high up in the facade and therefore they coud hot be checked in an other way.

In an other example IR-camera were used to localize moisture on the surface of the masonry exterior wall. Samples were taken and moisture contents measured after that and excess moisture were found.

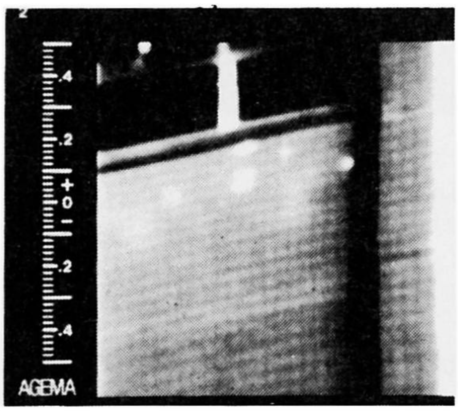


Fig 5. Loose ceramic tiles in facade.

4. CONCLUSIONS

It can be stated that there are many potential applications for IR-technics in nondestructive condition assesement of facades. Primarily the IR-camera can be used for investigation of any kinds of effects where temperature differences exist or occur. However, due to climatic conditions there are several restrictions to outdoor investigation. Still more research is therefore needed.

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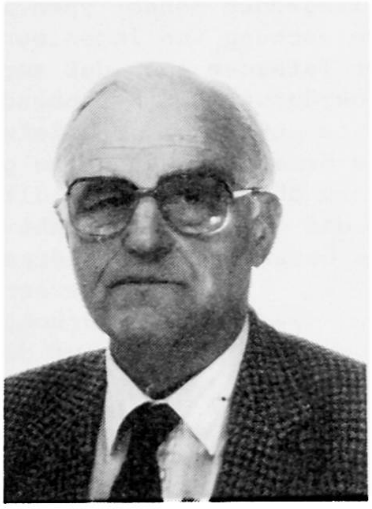
Spannkraftmessungen mit integrierten Lichtwellenleitersensoren

Monotoring Stressing Behaviour with Integrated Optical Fibre Sensors

Tensions mesurées à l'aide de détecteurs à fibre optique

Friedrich Karl LEVACHER
Dr. rer. nat.
Felten & Guilleaume
Köln-Mühlheim,
Bundesrep. Deutschland



Friedrich Karl Levacher, geb. 1924. Promotion an der Universität Köln, Seit 1955 Mitarbeiter der Firma Felten & Guilleaume Carlswerk AG, Köln-Mühlheim, zunächst als wiss. Assistent, dann als Leiter der Versuchsanstalt für Nachrichtentechnik, heute als Leiter des Fachbereiches Entwicklung.

Hans-Joachim MIEßELER
Dipl-Ing.
Strabag Bau AG
Köln, Bundesrep. Deutschland

Hans-Joachim Mießeler, geb. 1944. Studium in Wuppertal, von 1967 im Technischen Büro der Strabag Hauptverwaltung in Köln, seit 1987 in der Hauptabteilung Technik und Pro-Projektbearbeitung Leiter der Abteilung Forschung und Entwicklung.

ZUSAMMENFASSUNG
An die Früherkennung irgendwelcher Schäden an den Tragwerksstrukturen werden heute hohe Anforderungen gestellt. Durch Integration von Sensoren in Faserverbundwerkstoffe, die als Vorspannbewehrung Anwendung finden, werden die Verbundwerkstoffe kontrollierbar, man kann von sogenannten "intelligenten Tragstrukturen" sprechen. In dem Beitrag werden die Funktionsweise dieses permanenten Ueberwachungsverfahrens und die ersten Anwendungen vorgestellt.

SUMMARY
Today great emphasis is placed on the early detection of defects in load bearing structures. By integrating sensors into composite fibre materials which are used as prestressing reinforcement, the composite materials become controllable and one speaks of a so-called "intelligent load bearing structure". The structural application of this permanent monotoring process and its initial cases of application are shown in this article.

RÉSUMÉ
Il devient de plus en plus important de reconnaitre en temps utile des détériorations dans des structures porteuses. En intégrant des détecteurs dans les matériaux composites renforcés par des fibres – lesquels servent de précontrainte – les matériaux composites deviennent contrôlables. Il est possible d'é voquer le terme de "structures porteuses intelligentes". L'article explique le fonctionnement de tels procédés de surveillance permanente et les premières applications.

1. DIE WIRKUNGSWEISE DES LWL-DEHNUNGS-SENSORS

Die Entwicklung von Lichtwellenleiter-Sensoren (LWL-Sensoren) fußt auf dem gesicherten Kenntnis- und Qualitätsstand bezüglich der Lichtwellenleiter, die seit nahezu einem Jahrzehnt in Nachrichtenkabeln eingesetzt werden.

Unter den verschiedenen vorliegenden Sensortypen ist der LWL-Dehnungs-Sensor besonders für die Zustandsüberwachung von Ingenieurbauwerken geeignet. Seine Wirkungsweise geht von der Tatsache aus, daß mechanische Einflüsse, die an der Oberfläche von LWL Mikrokrümmungen (Microbending) erzeugen, zu einer Schwächung (Dämpfung) des im LWL fortgeleiteten Lichtes führen. Dieser Effekt wird beim LWL-Dehnungs-Sensor in der Weise genutzt, daß ein Gradient-LWL eine Bewicklung aus einem dünnen Draht erhält (Abb. 1). Die Schlaglänge des Drahtes ist so bemessen, daß der von der Drahtwendel umschriebene Kreis bei einer axialen Dehnung des Gesamtgebildes stärker abnimmt, als der Durchmesser des LWL. Infolgedessen drückt die Drahtwendel radial auf den LWL. Sie erzeugt an ihm Mikrokrümmungen, die entsprechende Dämpfungszunahmen verursachen und damit den LWL zu einem LWL-Dehnungs-Sensor machen. Zum mechanischen Schutz umgibt kraftschlüssig den LWL-Dehnungs-Sensor eine Hülle, beispielsweise aus längslaufenden in eine Harzmatrix eingebetteten Glasfasern.

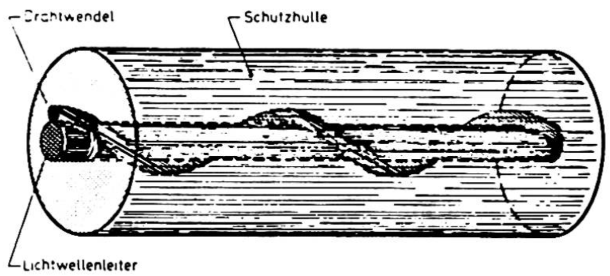


Abb. 1: Aufbau des LWL-Dehnungssensors

Zur Untersuchung der Tauglichkeit des LWL-Dehnungs-Sensors für die Überwachung von Ingenieurbauwerken wurden mit ihm im Institut für Baustoffe, Massivbau und Brandschutz der TU Braunschweig praxisnahe Untersuchungen, unter anderem an einer Stahlbetonplatte von 180 mm Dicke und 3620 mm Belastungslänge, durchgeführt. Der Sensor war zu diesem Zweck in einer längslaufenden Nut an der sich beim Biegen wölbenden Seite der Platte eingeklebt worden. Über die Untersuchungsergebnisse geben die Abbildungen 2 und 3 Auskunft. Hierin sind die Dämpfung des LWL-Dehnungs-Sensors als Funktion der Durchbiegung bzw. der Biegekraft der Stahlbetonplatte aufgetragen. Beide Kurven ergänzen einander. Sie informieren über die Größe der Durchbiegung und zeigen gleichzeitig durch ihre Unstetigkeitsstellen an, bei welchen Belastungsgrößen spontane Änderungen, d.h. Risse im Betongefüge aufgetreten sind.

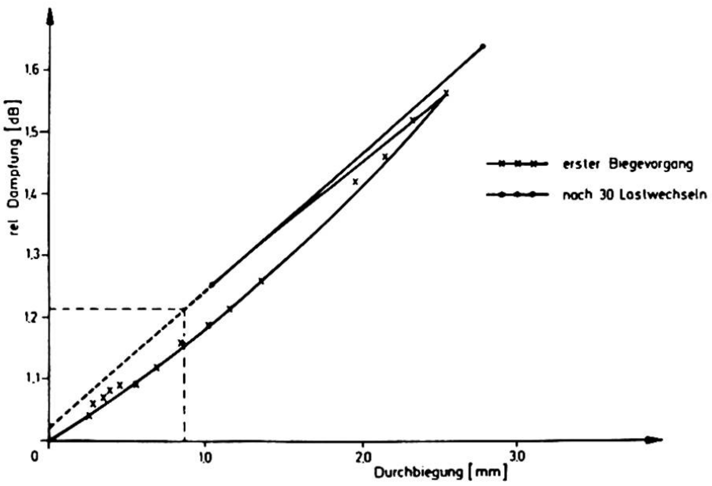


Abb. 2: Dämpfung eines LWL-Dehnungssensensors in einer Betonplatte (180x3920 mm) als Funktion der Durchbiegung der Platte

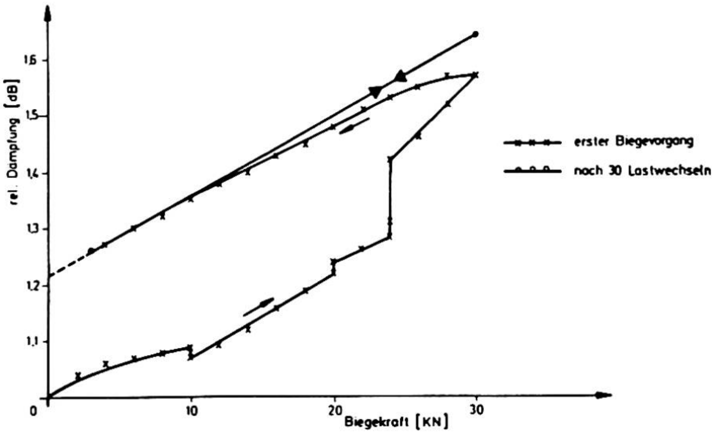


Abb. 3: Dämpfung eines LWL-Dehnungs-Sensors in einer Betonplatte (180 x 3920 mm) als Funktion der auf die Platte wirkende Biegekraft

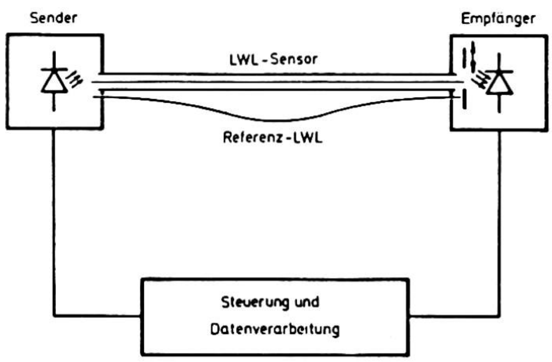


Abb. 4: Schema des temperaturkompensierten Dämpfungsmeßverfahrens

Das benutzte Meßverfahren mit optischem Sender bzw. Empfänger am Anfang bzw. Ende des Sensors gibt Information über die Beeinflussung auf der Gesamtlänge des Sensors. Für diese integrale Bewertung verfügt der Entwickler heute über zwei Meßprinzipien. Das Grundsätzliche hierzu zeigt schematisch die Abb. 4. Hierin dient der Referenz-LWL zur Temperaturkompensation und zur Korrektur eventueller Veränderungen der Senderleistung und der Empfängerempfindlichkeit.

Das zweite integrale Meßverfahren (Abb. 5) ist dadurch gekennzeichnet, daß Sender und Empfänger über Strahlenteiler am gleichen Ende mit dem LWL-Dehnungs-Sensor und dem Referenz-LWL verbunden sind. Am entgegengesetzten Ende werden die LWL beider Sensoren verspiegelt. Das dort ankommende Licht des Senders wird somit reflektiert. Es gelangt wieder zum Ausgangsort zurück und wird dort über Strahlteiler zum Pegelempfänger abgezweigt und bewertet. Dieses Verfahren bietet überall dort Vorteile, wo nach der Montage ein Ende der Sensoren nicht mehr zugänglich ist.

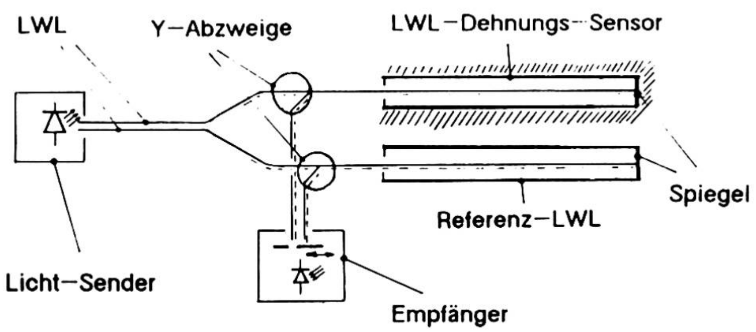


Abb. 5: Meßprinzip für die temperaturkompensierte Dämpfung von LWL-Dehnungs-Sensoren; von einem Sensorende aus gemessen.

Ergänzend zu den für die integrale Dauerüberwachung von LWL-Dehnungs-Sensoren entwickelten Verfahren und Geräten ermöglicht das aus der optischen Nachrichtenkabeltechnik bekannte Reflexionsverfahren, das örtliche Dämpfungsverhalten von LWL-Sensoren zu messen und damit räumlich verteilte Belastungsschwerpunkte dezimetergenau zu orten wie etwa Dehnungsveränderungen in Glasfaser-Spanngliedern. Dabei können die Sensorstrecken im Kilometerbereich liegen.

2. DAS "INTELLIGENTE" SPANNGLIED

Glasfaserverbundwerkstoffe für die Anwendung im Spannbetonbau, von der Arbeitsgemeinschaft HLV-Elemente in einem langjährigen vom Bundesministerium für Forschung und Technologie geförderten Forschungsvorhaben entwickelt und erstmalig eingesetzt, sind die korrosionsbeständige Alternative zum herkömmlichen Spannstahl.

Die daraus hergestellten Spannglieder bestehen aus 19 Glasfaserstäben (HLV-Stäbe) mit einem Nenndurchmesser von 7,5 mm und haben eine Gebrauchslast von 660 kN (Abb. 6). Der Querschnitt eines einzelnen Glasfaserstabes enthält ca. 68 % Glasfasern und 32 % ungesättigte Polyesterharze bzw. Epoxidharze. Die Längszugfestigkeit des Werkstoffes von 1670 N/mm2 ist Folge des hohen Glasfaseranteiles mit strenger unidirektionaler Orientierung (Abb. 7). Die bei der Herstellung in die einzelnen Glasfaserstäbe integrierten Lichtwellenleiter ermöglichen infolge einer besonderen Präparation den Einblick in das Spannungsdehnungsverhalten des damit vorgespannten Bauteils. Die Lichtwellenleiter wirken als Dehnungssensoren und lassen Rückschlüsse auf Veränderungen des Spannungszustandes und deren Lokalisierung zu. Durch die integrierten Sensoren werden vorgespannte Konstruktionen kontrollierbar, man kann von sogenannten "intelligenten Tragstrukturen" sprechen. Die Arbeiten hierzu werden auch über ein Forschungs- und Entwicklungsvorhaben der EG (BRITE Projekt 1353) gefördert.

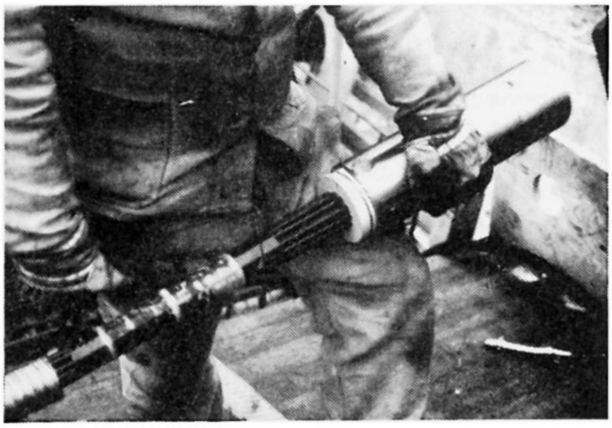


Abb. 6: HLV-Sapnnglied mit 19 Stäben

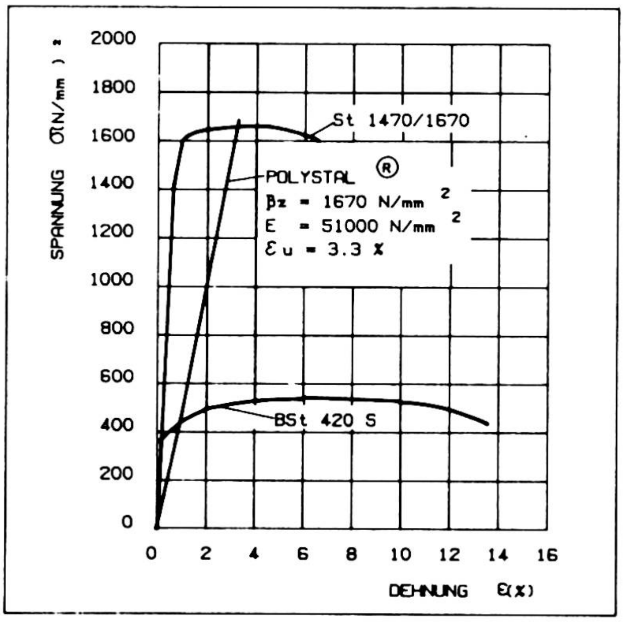
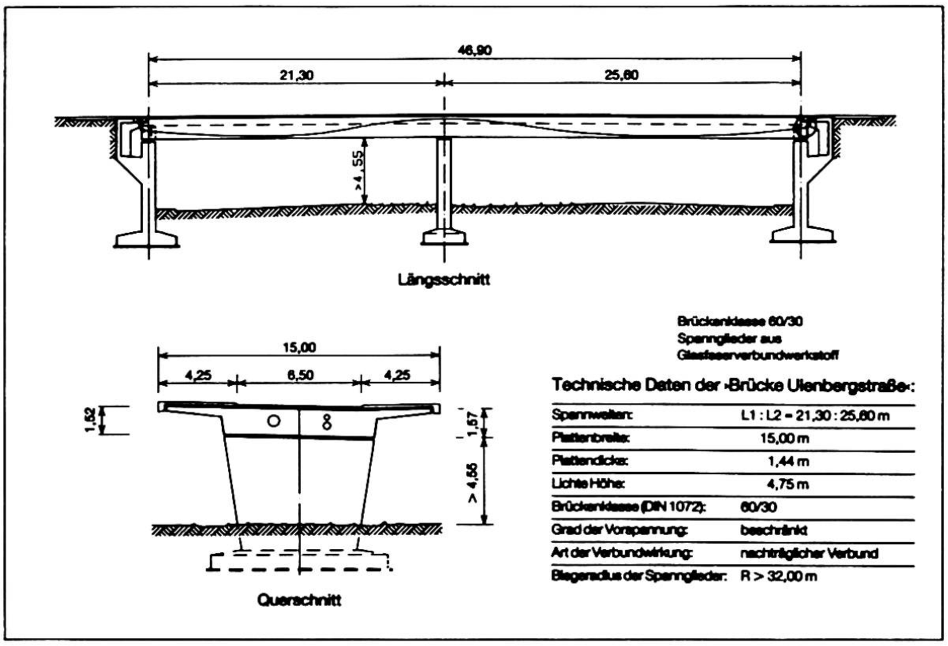


Abb. 7: Spannungsdehnungsdiagramm von Glasfaserverbundstäben im Vergleich mit Beton- und Spannstahl

3. DIE ANWENDUNGEN, BRÜCKE ULENBERGSTRASSE DÜSSELDORF

Diese weltweit erste für schwerste Lasten des Straßenverkehrs ausgelegte Spannbetonbrücke (Brückenklasse 60/30), anstatt der bisher üblichen Stahlspannglieder mit Spanngliedern aus hochzugfesten Glasfaserstäben vorgespannt, wurde im Auftrage der Stadt Düsseldorf durch die Arge HLV-Elemente als Demonstrationsbauvorhaben des Bundesministers für Forschung und Technologie in den Jahren 1985/1986 errichtet.

Die Brücke "Ulenbergstraße" ist eine zweifeldrige massive Plattenbrücke, die durch 59 Stück HLV-Spannglieder mit je 19 HLV-Stäben, Nenndurchmesser 7,5mm, vorgespannt wird (Abb. 8).



Technische Daten der »Brücke Ulenbergstraße«:

Spannweiten:	L1 : L2 = 21,30 : 25,60 m
Plattenbreite:	15,00 m
Plattendicke:	1,44 m
Lichte Höhe:	4,75 m
Brückenklasse (DIN 1072):	60/30
Grad der Vorspannung:	beschränkt
Art der Verbundwirkung:	nachträglicher Verbund
Biegeradius der Spannglieder:	R > 32,00 m

Abb. 8: Technische Daten Brücke "Ulenbergstraße"

In diesem Brückenquerschnitt werden Lichtwellenleiterdehnungssensoren, zwei in den Beton der Fahrbahnplatte und je einer in einem Glasfaserstab zweier HLV-Spannglieder integriert, permanent überwacht (Abb. 9). Diese moderne Form der Bauwerksüberwachung wird erstmalig an der Brücke Ulenbergstraße seit der Verkehrsübergabe der Brücke im Juli 1986 durchgeführt.

Spannglied Nr.21 u. 22 mit LWL-Sensoren

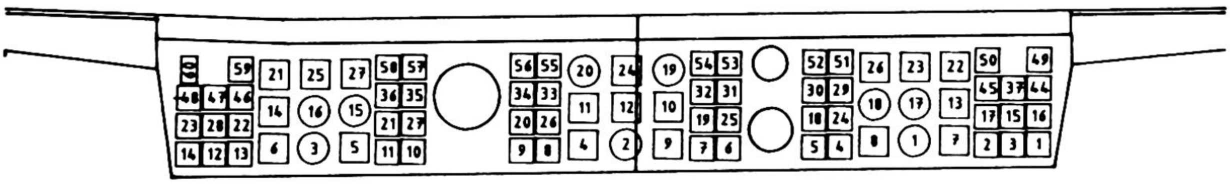
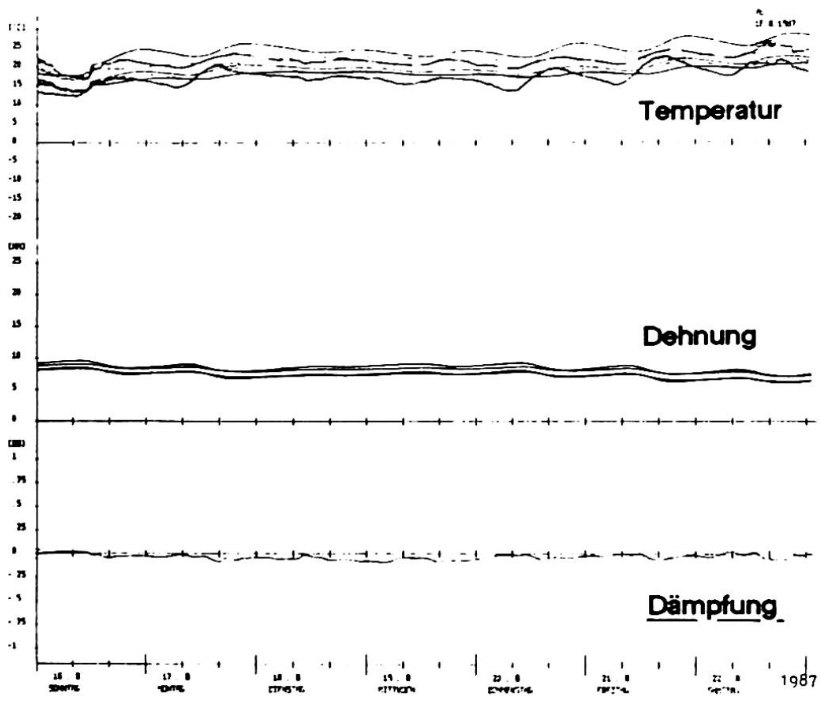


Abb. 9: Anordnung der LWL-Sensoren im Querschnitt

Das Ergebnis zeigt Abb. 10 als Beispiel für den Zeitraum von einer Woche im August 1987. Mit der Sensordämpfung werden hierin verglichen die im Brückeninneren laufend gemessenen Temperaturen sowie die an den Brückenköpfen gemessenen Dehnungen der Brücke. Aus den Aufzeichnungen wird erkennbar, daß sich der Temperaturgang auf die Dehnung der Brücke und damit auf den Dämpfungs-



verlauf des Sensors auswirkt. Das heißt, daß der LWL-Dehnungs-Sensor die Dehnung der Brücke ordnungsgemäß wiedergibt. Der Dämpfungsverlauf läßt dabei keine Unstetigkeiten erkennen. Das ist ein Hinweis dafür, daß im Beobachtungszeitraum praktisch keine Risse im Beton entstanden sein können. Diese Feststellung gilt für den gesamten bisherigen Beobachtungszeitraum von mehr als einem Jahr. Die Sensor-Überwachung an der Brücke Ulenbergstrasse, Düsseldorf, bestätigt somit ein unter den genannten Gesichtspunkten bewertetes völlig normales Verhalten dieses Bauwerkes.

Abb. 10: Überwachung der Brücke Ulenbergstraße durch Dämpfungsmessungen an einem LWL-Dehnungs-Sensor

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Maintenance and Monitoring of Concrete Bridges

Maintenance et surveillance des ponts en béton

Wartung und Kontrollmessungen von Stahlbetonbrücken

D. V. MALLICK
Technical Advisor,
National Consulting Bureau,
Tripoli, Libya

Prof. Mallick, born 1936, received his doctorate at Bristol University, U.K. Professor and Consultant for 12 years. Prof. Mallick, now in a consulting firm, is a technical advisor. Fellow ICE (India) and ASCE.

M. M. TAWIL
Prof. of Civil Eng.
University of El Fateh,
Tripoli, Libya

Prof. Tawil, born 1945, received his doctorate at University of Colorado, U.S.A. Prof. Tawil is a University staff member for 12 years. He has also been a part-time technical advisor to the National Consulting Bureau, Tripoli for the last 10 years.

A. A. SHIBANI
Chairman,
National Consulting Bureau
Tripoli, Libya

Dr. Shibani, born 1944, received his doctorate at North Carolina State University, U.S.A. Dr. Shibani, besides his duties at NCB, has been a part-time staff member in the Mechanical Engg. Dept., University of El Fateh, Tripoli for the last 12 years.

SUMMARY
Highway concrete bridges of the Tripoli second ring road suffered cracking due to structural and non-structural effects. A program of regular inspection, monitoring and maintenance of these bridges has been proposed to ensure durability and proper functioning. Stress has been laid on simple and accurate measuring instruments which led to the development of a Plumbline Deflectometer, described in this paper, for measuring deflections of bridge elements.

RÉSUMÉ
Les ponts en béton du deuxième périphérique autoroutier de Tripoli présentent des fissures dues à des causes structurales et non structurales. Un programme d'inspection, de maintenance et de surveillance a été proposé afin d'assurer la durabilité de ces ouvrages. La volonté d'obtenir des mesures précises de contraintes a conduit à l'élaboration d'un déflectomètre à fil à plomb conçu pour la mesure des déformations de chaque élément de pont.

ZUSAMMENFASSUNG
Die Stahlbetonbrücken auf der Schnellstraße der Zweiten Ringstraße von Tripolis haben durch das Auftreten von struktuellen und nichtstruktuellen Rissen Schaden erlitten. Ein Programm von regelmäßigen Untersuchungen, Kontrollen und Wartungsarbeiten wird zur Sicherstellung der Dauerhaftigkeit und der einwandfreien Funktionsfähigkeit vorgeschlagen. Die Beanspruchungen, resp. Dehnungen wurden mit einem einfachen, jedoch genauen Messinstrument, nämlich einem Schnurlot-Durchbiegungsmesser, bei den einzelnen Brückenelementen aufgenommen.

1. INTRODUCTION

Durable bridges ensure fullfilment of their intended function for whole of their design life. Durability reflects the overall strength, serviceability and resistance to environmental effects of the materials forming the structure. It is not only dependent on the original strength design and assumed characteristics of construction materials but is a function of the materials used in the construction, methods of construction, workmanship, environmental effects, applied load intensity and their duration, accidental loads and regular maintenance repairs. The construction of bridges in both developing and underdeveloped countries is also influenced by the consultants. It is often the case that foreign consultants are not fully aware of the environmental conditions of the clients country and generally base their designs on the standards, material specifications and the degree of workmanship prevalent in their own countries. The effect of sophisticated computer oriented analysis and design procedures will be invariably offset by the use of poor quality of local aggregates, occasional poor workmanship of the contractor, inadequate site supervision and lack of maintenance due to shortage of manpower and/or finances.

2. TSRR HIGHWAY BRIDGES

The main theme of this paper is a case study of Tripoli Second Ring Road (TSRR) highway bridges, completed in 1986,consisting of reinforced concrete decks supported on end abutments and intermediate portal frames with overhangs for multiple spans and, on end abutments only for single spans. Some of the decks are post-tensioned and the others are of reinforced concrete. These bridges became the victim of the problems of a developing country as mentioned above. Even during the construction, plastic shrinkage and plastic settlement cracks were identified in bridge decks and, thermal contraction and drying shrinkage cracks in the supporting end abutments. In addition to these cracks some structural cracks were observed, with a maximum width of 0.4 mm, in the cantilevers of supporting portal frame elements. Investigations were carried out according to the scheme presented (Fig.1). Some of these typical cracks in the supporting elements of the highway bridges are shown (Fig.2).

3. CAUSES OF CRACKS

Causes of concrete cracking of TSRR bridges are stated in [1]. The main causes of non-structural cracks can be attributed to type of aggregates, poor fines grading, dust content of aggregates, high water content of the mix, low humidity, high ambient temperatures, hot concrete, very large sections and poor workmanship, etc. The structural cracks occured due to improper reinforcement detailing , poor quality of concrete near the top surfaces of supporting cantilever elements, large concrete cover, premature loading, concentration of stresses, sudden change in sections and sequence of deck slab construction. The study of cracks in the TSRR bridge elements concluded that the observed non-structural cracks are mostly within the permissible limits, and are not structurally dangerous provided they are properly repaired and maintained. Some of them cause unsightly appearance and require cosmetic treatment. The other cracks can be treated with epoxy injection. The structural cracks in the cantilevers of the supporting portal frame structure required immediate attention from serviceability consideration. These elements have been strengthened by providing additional concrete fins to ensure durability. Both these types of cracks require regular inspection, monitoring and maintenance.

4. PERIODIC INSPECTION AND MONITORING

A satisfactory structural response encompassing the durability aspect of TSRR bridges can be assured by carrying out periodic visual and physical inspection by qualified personnel in accordance with the provision of [2] followed by proper maintenance and repairs program.

Visual inspection will concentrate on examining all exposed concrete for the existence and severity of cracks and any deterioration of concrete itself due to chloride ingression and/or chemical reactions. The elements need to be inspected are the end abutments, intermediate supporting piers and their cross beams, bearings, deck slabs and the wing walls. When cracking is found, location of the cracks and their size should be carefully recorded for future reference and comparison.

Physical inspection will require taking some measurements of the response of bridges. It will involve recording the width and the depth of penetration of all cracks, measuring the deflection of the overhangs of the cross beams of the portal frames and deck slabs, settlement of foundations of the supporting elements, deformation of bearings specially for high skew deck slabs and, monitoring the amplitude of vibration of the deck slab under the moving traffic. It is well known that the stiffness of a structure can be best monitored by measuring dynamic characteristics like the natural frequency. A change in the frequency will directly reflect any change in stiffness of the structure.

Undeniably, there has been a serious concern by the public about the safety and durability of TSRR bridges after the addition of so called concrete fins to increase the strength of overhangs just before and after opening of the ring road. The present authors were involved in the planning of maintenance program of these bridges for the future.

5. INSTRUMENTATION

Certain techniques and instruments are already available for the inspection of bridges for monitoring their structural response [3]. But there is a need for simple and easy operating instruments, and procedures in developing countries. Some of them are recommended in Table 1.

The Plumbline Deflectometer (Fig.3) was developed by the authors to facilitate monitoring of vertical deflections of the overhangs of the cross beams of the portal frame supporting elements and that of the bridge decks. This instrument requires no scaffolding. It requires only two stainless steel studs to be fixed - one projected from the concrete element whose deflection is being measured and the other at the base. At the time of reading, the micrometer of the deflectometer is adjusted to make the string tight which occurs when the circular face of the conical plumb bob touches the bottom surface of the lower steel stud. It is a very light, handy and accurate instrument, having a range of 50 mm and a least count of 0.01 mm, which does not require much skill to operate it and can be used repeatedly. Amplitude and frequency of vibration can be measured by using an accelerometer transducer and a vibration meter.

6. ACCEPTANCE LOAD TEST

Acceptance load tests of some bridges were the requirement of the client before handing over of the bridges in the light of the occurrence of cracks in the supporting elements of various bridges. The acceptance tests were conducted on two bridges and their results were set against predictions of structural behaviour rather than against any arbitrary criteria under static vehicle loading applied for a short duration. The usual procedure of conducting load tests is according to CP 110 or ACI 318 - 83 which is applicable to reinforced concrete structures. The validity of long term load test, according to these standards is open to question since bridges are subjected to transient or time varying loads. For acceptance tests, loading for a short duration - necessary to develope static equilibrium before taking measurements - should suffice. There is an urgent need for writing specifications for accepting load test of bridges.

7. CONCLUSIONS

Inspection and monitoring the response of bridges should be conducted in a systematic and organised manner for as not to interfere with the flow of traffic for a long time by choosing instruments which are quite accurate, simple to operate and easy to install. Each bridge is to be inspected at regular intervals not to exceed 2 years by qualified personnel. Inspection should not be confined to searching for cracks which may exist but should include measurements for investigating the response of a structure during its use. Finally, regular and timely maintenance repairs of bridges will ensure durability for the whole of their design life.

FEATURE	PARAMETER	POSSIBLE INSTRUMENTS
Cracks	Width	Telescopic crack width recorder, demec gauge, glass tell tale or gypsum coating.
	Depth	Chisle & hammer, core samples, ultra-sonic pulse velocity (U.P.V.).
	Length	Ruler or a flexible cord.
Displacement	Vertical deflection	Plumbline Deflectometer, dial gauges, precision level or wire resistance strain gauge gadgets.
	Angular rotation	Clinometer, Plumbline Deflectometer for vertical surfaces.
Settlement	Foundations	Precision level (reference Mark)
Reinforcement	Stresses	Wire resistance strain gauges
	Bar size / Spacing	Profometer
Concrete	Quality	U.P.V. tests, Schmidt hammer
	Strength	Core test, U.P.V. tests.
Dynamic Response	Frequency & Amplitude	Vibration meter + Accelerometer

Table 1 Instrumentation for inspection and measuring the response of concrete highway bridges

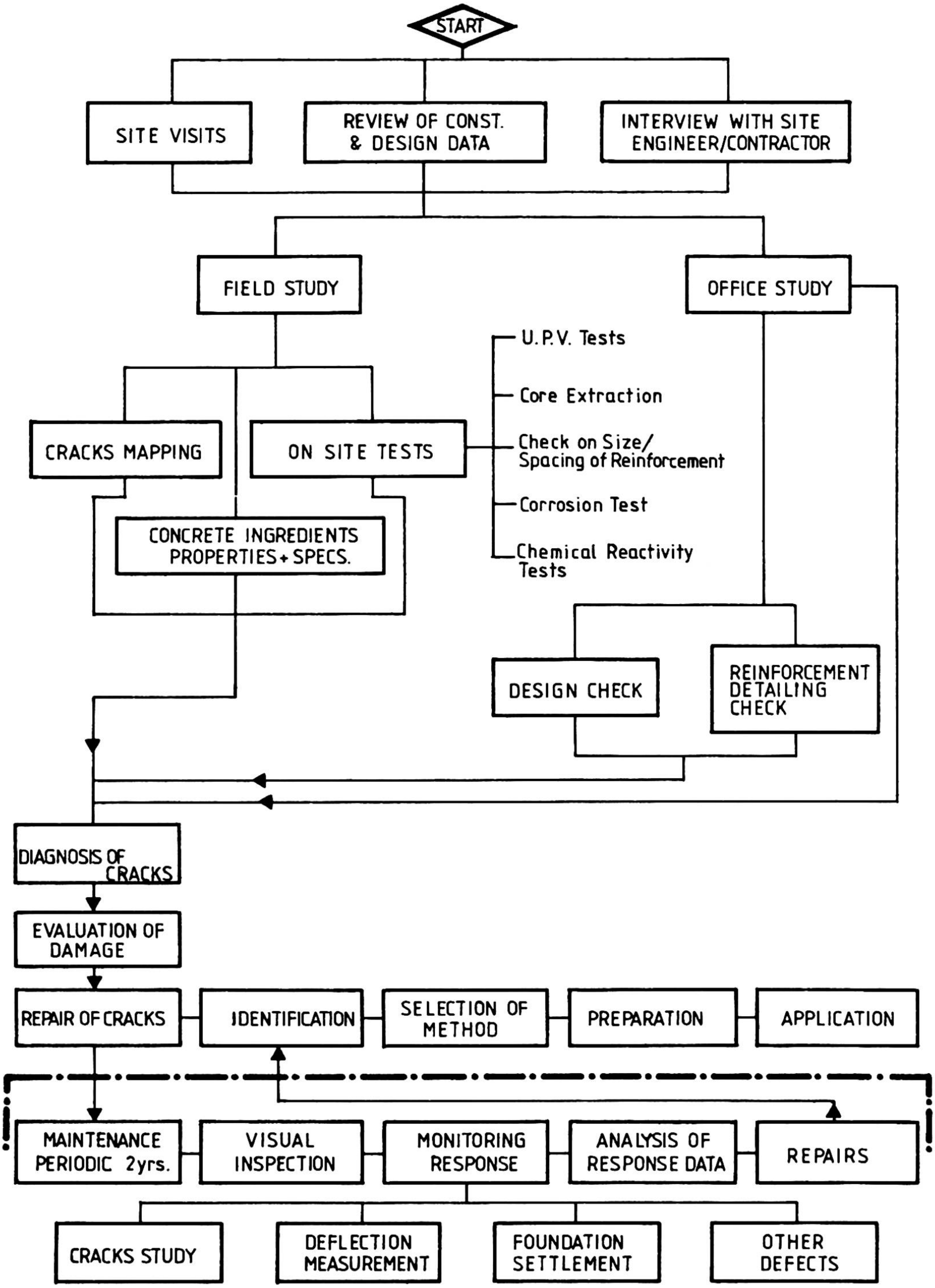

Fig. 1 Scheme of inspection, monitoring and maintenance

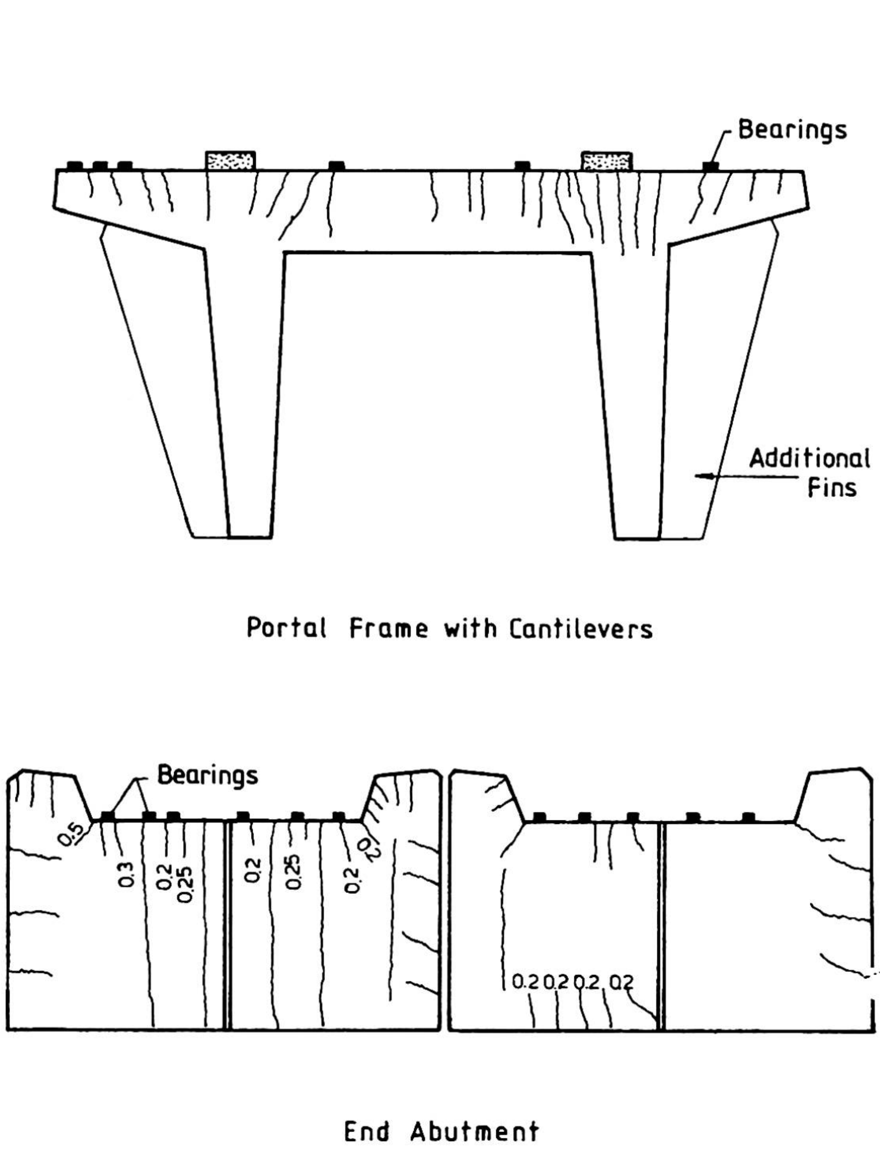

Fig.2 Typical cracks in the supporting elements of TSRR highway bridges

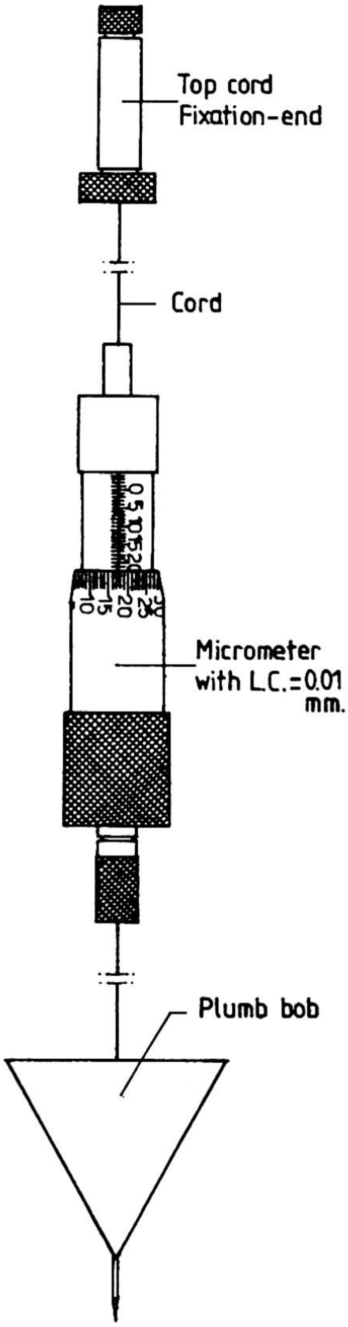

Fig.3 Plumbline Deflectometer

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Evaluation de l'état mécanique réel de ponts en béton précontraint

Bestimmung des tatsächlichen mechanischen Zustandes von Spannbetonbrücken

Evaluation of the actual mechanical behaviour of prestressed concrete bridges

Jacques CHATELAIN
Ingénieur ETP
LCPC
Paris, France

Jacques Chatelain, né en 1935 obtient son diplôme d'Ingénieur à l'Ecole Spéciale des Travaux Publics, puis reçoit le diplôme du CHEBAP. Entré au LCPC en 1958, spécialiste des problèmes de pathologie d'Ouvrages d'Art, il est actuellement Chef de la Division Surveillance et Pathologie des Ouvrages d'Art.

Bruno GODART
Ingfénieur TPE
LCPC
Paris, France

Bruno Godart, né en 1956, obtient son diplôme d'Ingénieur à l'Ecole Nationale des Travaux Publics de l'Etat, puis reçoit le diplôme de Master of Science de Stanford University. Entré au LCPC en 1980, il est actuellement Chef de la Section d'Intervention sur Ouvrages d'Art.

RÉSUMÉ
Certains ponts en béton précontraint construits par encorbellements successifs peuvent présenter des insuffisances de résistance à la flexion se manifestant par des ouvertures de joints. L'article présente la stratégie d'auscultation développée pour évaluer l'état mécanique réel de ces ponts et préciser leur renforcement éventuel par précontrainte additionnelle.

ZUSAMMENFASSUNG
Bei gewissen segmentbauartigen Spannbetonbrücken können sich infolge ungenügender Biegefestigkeit die Arbeitsfugen öffnen. Dieser Aufsatz schildert die Untersuchungsstrategie zur Bestimmung des mechanischen Zustandes dieser Brücken und ihrer eventuellen Verstärkung durch zusätzliche Vorspannung.

SUMMARY
Certain prestressed concrete bridges built by the balanced cantilever method exhibit transverse cracking at a few joints, a cracking characteristic of insufficient bending strength. This paper presents the site testing strategy developed in order to assess the actual mechanical behaviour of these bridges and to specify their eventual strengthtening by additionnal prestressing.

1. INTRODUCTION

Le béton précontraint a connu un grand essor en France après la seconde guerre mondiale. A partir de 1960 la construction de tabliers "par encorbellements successifs" s'est imposée lorsque l'on envisageait de franchir des portées supérieures à 50 m. Il s'agissait alors, presque toujours, de poutres caissons continues à voussoirs préfabriqués ou coulés en place.

C'est pendant les années 72-75 que l'on a découvert que ce type de structures pouvait présenter divers défauts : fissuration de diffusion et d'entraînement de précontrainte, poussée au vide de hourdis inférieurs courbes, fissuration d'effort tranchant..., mais surtout, des désordres liés à la flexion générale. Evidemment, depuis 1975, la situation a été redressée : certains ouvrages anciens ont été renforcés par ajout de précontrainte et les nouveaux ouvrages ont bénéficié de nouvelles règles de dimensionnements.

La pathologie attachée à ce type de construction nous a amené à développer des méthodes d'investigations spécifiques destinées à apprécier le mieux possible l'état mécanique réel d'un tablier présentant des signes de maladie (signes quelquefois très peu visibles).

Nous décrivons ici la méthodologie maintenant adoptée qui permet de définir le degré d'insuffisance de résistance d'un tablier, et par là même, d'ajuster au mieux le renforcement par précontrainte additionnelle.

2. MANIFESTATION EXTERIEURE DES DESORDRES PROVOQUES PAR UNE INSUFFISANCE A LA FLEXION

Les désordres se signalent généralement sous la forme d'une fissuration qui se localise dans les joints entre voussoirs et qui affecte le hourdis inférieur des poutres caisson en remontant plus ou moins dans les âmes. Ces désordres se situent préférentiellement dans les zones dites de "moment nul" au voisinage des foyers, et parfois, au milieu des travées principales. Pour les premiers cas pathologiques découverts, les signes extérieurs de fissuration étaient suffisamment nets pour prononcer un premier diagnostic. Par la suite, on s'est aperçu que le simple examen visuel ne pouvait plus suffire pour se prononcer de façon objective sur l'état réel de la structure, et une démarche plus systématique a alors dû être employée.

3. DEMARCHE GENERALE DE L'AUSCULTATION

La démarche à suivre lors de l'auscultation d'un pont comporte trois étapes :

a) <u>le dépistage</u> qui consiste à identifier les ouvrages insuffisants. Ce dépistage est assuré par une surveillance basée sur des examens visuels périodiques, surveillance prévue par la règlementation en France /1/.

b) <u>l'exploration préliminaire</u>, qui, au moyen d'une instrumentation légère (essentiellement des capteurs de déplacement mis en place sur les joints) permet d'évaluer l'étendue des désordres, c'est-à-dire le nombre de joints qui "respirent" sous le trafic supporté par l'ouvrage.

c) <u>l'auscultation</u> proprement dite qui fait l'objet de cette communication.

Nous nous plaçons donc maintenant dans l'hypothèse où un pont est déficient et où 2 à 3 joints reconnus comme étant parmi les plus faibles ont été choisis, puis instrumentés "lourdement" et analysés suivant la méthode décrite ci-après. Mais auparavant il apparaît nécessaire d'évoquer les problèmes posés par le fonctionnement réel des joints.

4. COMPORTEMENT MECANIQUE D'UN JOINT FISSURE

Un joint courant de pont en béton précontraint comporte des câbles de fléau ancrés dans les âmes, et est traversé en sa partie inférieure par des câbles de continuité. Si un faible pourcentage d'aciers passifs traversent les joints entre voussoirs coulés en place, en revanche, il n'existe pas d'aciers passifs dans les joints entre voussoirs préfabriqués. Si l'on suppose une injection parfaite au coulis de ciment des conduits de précontrainte, alors on peut admettre que les câbles de précontrainte adhérents au béton se comportent comme des armatures de béton armé lorsqu'il y a ouverture de fissure. Ainsi, sous sollicitations de moment fléchissant croissant, un joint peut-il être considéré comme une section de béton armé soumise à la flexion composée. Ceci est illustré par la figure 1 qui donne la variation de contrainte $\Delta\sigma a$ (que l'on désigne souvent par "surtension" dans le cas d'armatures de précontrainte) en fonction du moment fléchissant sollicitant ΔM.

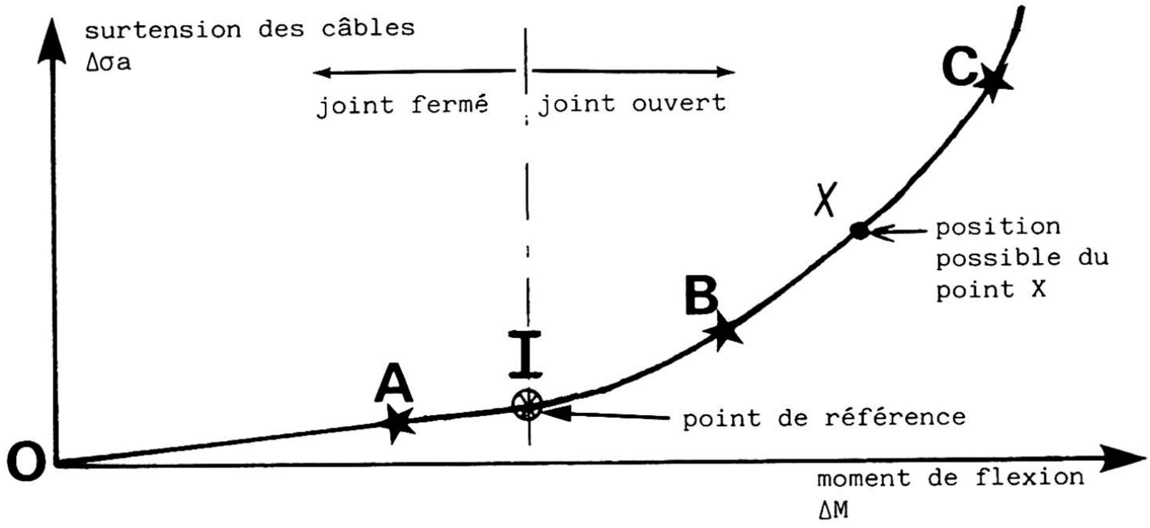

Fig. 1 Evolution de la surtension des câbles en fonction du moment

Jusqu'au point I, le joint reste fermé et se comporte en section homogène : la déformation de l'armature est égale à celle du béton ; au délà du point I, il y a ouverture du joint qui se comporte alors en section de béton armé soumise à la flexion composée : la surtension $\Delta\sigma a$ est une fonction non linéaire du moment ΔM.

L'apprehénsion par le calcul d'un tel comportement est tout à fait classique et ne présente pas de difficulté pourvu que l'on connaisse la position réelle du point I : tel est l'objectif des essais dits "des moments de décompression" /2/.

Malheureusement, dans la réalité, le comportement d'un joint qui s'ouvre ne se caractérise pas toujours de façon aussi simple, et il faut tenir compte, au plan expérimental, de toute perturbation pouvant affecter un résultat de mesure. Ces perturbations sont dues principalement à trois causes :

■ la répartition des contraintes dans un joint n'est pas aussi régulière que celle estimée en section courante par la théorie des poutres. En effet, les déformations d'origine thermique lors du coulage des voussoirs, l'ancrage des câbles de fléau en tranche de voussoir, et le retrait différentiel des parties d'un même voussoir peuvent engendrer une "déconjugaison" des joints qui se constate aussi bien pour les voussoirs coulés en place que pour les voussoirs préfabriqués (cf /3/). La position des câbles de continuité (répartis dans le hourdis inférieur ou regroupés dans les goussets inférieurs) joue aussi un rôle important sur le mode d'ouverture du joint.

■ il existe parfois une fissuration locale qui résulte des efforts de diffusion et d'entraînement des ancrages de câbles de continuité situés à proximité des joints.

■ les déformations des câbles dans un joint dépendent de la qualité mécanique des injections des conduits de précontrainte ; cette qualité est variable suivant les conduits et entraîne des différences entre les longueurs d'ancrage des câbles de part et d'autre du joint qui sont particulièrement difficiles à apprécier.

Malgré la complexité du fonctionnement des joints entre voussoirs, l'expérience accumulée jusqu'à présent sur une trentaine d'ouvrages nous amène à considérer que l'évolution de la déformation des câbles en fonction du moment fléchissant appliqué dans le joint est finalement l'information la plus importante à acquérir et constitue un bon critère d'ouverture de joint.

5. STRATEGIE AUSCULTATOIRE

Nous allons donc raisonner sur cette relation déformation des câbles-moment pour illustrer la stratégie suivie qui sera ici réduite à l'étude de 3 cas : A, B, et C (voir figure 1).

Soit I le point correspondant à l'ouverture du joint. Si l'on définit maintenant l'état à vide d'un joint comme l'état dans lequel se trouve ce joint en l'absence de charges roulantes et de sollicitations thermiques sur l'ouvrage, appelons X le point situé sur la courbe de la figure 1 et correspondant à l'état à vide d'un joint de la structure auscultée. Notre stratégie consiste à localiser le point X par rapport au point I.
Examinons maintenant, la stratégie suivie selon la position relative de X par rapport à I, en raisonnant comme si on connaissait déjà les résultats de l'auscultation.

Si X = C : la détermination de cet état peut résulter de simples mesures exploratoires (constatation d'une ouverture importante du joint égale à 1 ou 2 mm) ou d'une auscultation d'un joint faiblement ouvert à vide (0,3 - 0,5 mm) et ayant une injection correcte des câbles. Dans ce cas, le renforcement par précontrainte additionnelle s'avère nécessaire, après avoir au préalable procédé à une injection de résine époxydique dans les joints, injection destinée à rendre l'ouvrage monolithique et à assurer ainsi une répartition convenable et une bonne rentabilité de la précontrainte. L'injection est réalisée en ouvrant les joints au maximum à l'aide d'un chargement de camions qui restent sur l'ouvrage pendant le temps nécessaire à la polymérisation de la résine, et en mettant à profit l'occurrence d'un gradient thermique. Les mesures de déformations réalisées lors du départ des camions nous montrent que ce type d'opération amène des contraintes de compression non négligeables ; cependant, celles-ci ne sont pas prises en compte dans les calculs en raison des lacunes existant dans la connaissance du comportement à long terme des résines injectées. Puis la précontrainte de renfort dimensionnée pour reprendre les actions variables est mise en place.

Si cette méthode de renforcement a l'avantage de redéfinir un nouvel état mécanique pour l'ouvrage, elle présente cependant l'inconvénient d'emprisonner des surtensions importantes dans les armatures de précontrainte (au-delà du point C).

Si X = A : on charge progressivement l'ouvrage pour décomprimer le joint et repérer la position de A par rapport à I, en appliquant la méthode dite des moments de décompression /2/.

Cette opération nous donne la réserve de moment existant à vide dans le joint. Si celle-ci s'avère insuffisante, un renforcement est alors envisageable et la précontrainte additionnelle est calculée en fonction de la réserve ainsi estimée.

Si X = B : Si le joint est injectable, alors la technique décrite en C peut être appliquée.

En revanche, si l'injection est impossible, des chargements sont entrepris en essayant de profiter des cas de chargement en moment négatif pour refermer le joint ; si cette opération réussit le point B peut-être situé par rapport à I et le renforcement éventuel tient compte du déficit de moment ainsi estimé.

Par contre si la tentative de fermeture du joint échoue, ce cas devient difficile à résoudre. La seule solution consiste à exécuter un calcul de type béton armé en flexion composée et à faire coïncider la courbe théorique $\varepsilon a = f(M)$ ainsi obtenue avec la courbe expérimentale. Dans le cas où l'on assiste à une remontée progressive de la fissure, ce calcul peut être aisément entrepris pourvu que l'on connaisse l'effort normal de précontrainte appliqué dans le joint. La coïncidence, cependant, peut poser quelques problèmes, dans la mesure où l'échelle des moments de la courbe expérimentale peut être faussée par une ouverture importante des joints modifiant la répartition des moments fléchissants sur une structure hyperstatique. Cette erreur peut être corrigée en modélisant les joints fortement ouverts d'une structure par des rotules élastiques dont les coefficients de rigidité sont extraits de mesures de rotation par inclinométrie ou extensométrie /4/. La validité du modèle peut être confortée par la mesure de l'évolution des réactions d'appui sous chargement de la structure. Lorsque la superposition des courbes expérimentale et théorique est achevée (voir figure 2), il devient alors possible de fixer la position de B par rapport à I puis de dimensionner la précontrainte supplémentaire.

Dans le cas où l'ouverture du joint se fait de façon "anarchique", le calcul en flexion composée est plus difficile à réaliser.

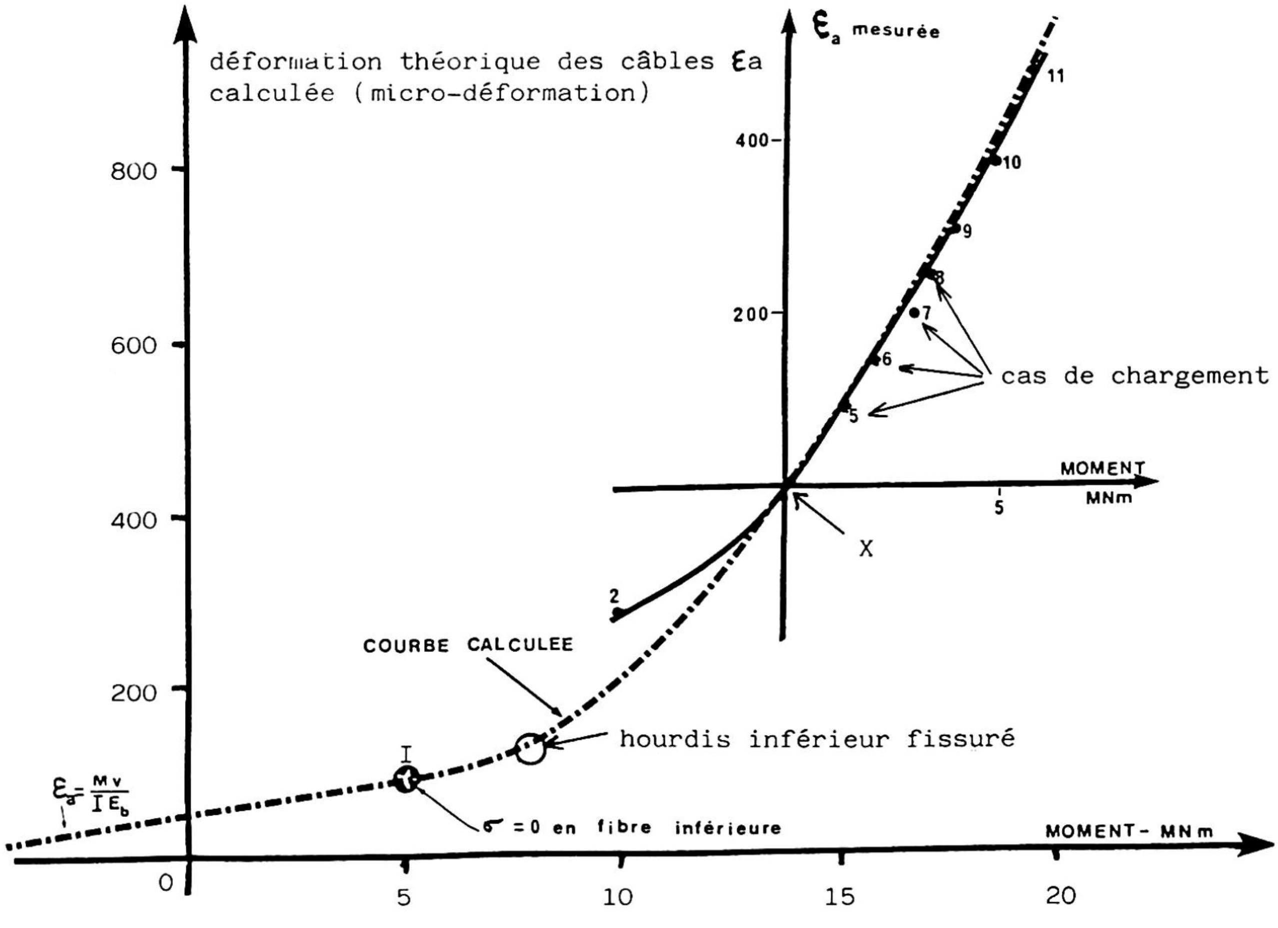

Fig. 2 Tentative de coïncidence entre courbes expérimentale et théorique

Enfin, pour certains ouvrages dont le point représentatif X se situe entre A et B, une étude en fatigue peut être menée à partir d'une acquisition simultanée du trafic, du gradient thermique et des déformations. Suivant les résultats de l'estimation de la durée de vie en fatigue des câbles de précontrainte, la décision peut être de renforcer immédiatement ou de surseoir à son exécution.

CONCLUSION

Lorsqu'un ouvrage en béton précontraint présente une insuffisance de résistance à la flexion, si un recalcul de l'ouvrage est obligatoire, en revanche dans la plupart des cas, le recalcul n'est pas suffisant. Pour aboutir au meilleur diagnostic, une association étroite entre mesures et calculs doit exister en tenant compte de tous les paramètres mesurables tels que les réactions d'appuis à vide, les rotations de section et surtout les déformations des câbles de précontrainte traversant les joints.

Cet article montre qu'une stratégie d'auscultation a pu être élaborée à partir de l'expérience acquise sur un certain nombre de cas pathologiques, et que cette stratégie dépend de la façon dont le renforcement éventuel est effectué.

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Time and Temperature Effects in Prestressed Concrete Bridges

Performance dans le temps et effets de la température sur des ponts en béton précontraint

Zeit- und Temperatureinflüsse in vorgespannten Stahlbetonbrücken

Ken ANDREWS
Lecturer
King's College London
Strand, UK

Ken Andrews obtained his first degree and Masters degree at King's College London. After a period in industry he returned to King's to undertake research and teaching. His main interests are computer-aided design and numerical analysis of time-dependent and impact problems in structures.

Andrew S. BEARD
Divisional Director
Mott, Hay & Anderson
Hong Kong

Andrew Beard was awarded his B. Sc. from University of Surrey and, after post-graduate study, his M. Sc. from Imperial College London. He has specialised in the design of concrete structures and maintains a particular interest in their long-term performance. He currently heads the Structural Division of MHA's Hong Kong Office.

George ENGLAND
Professor
King's College London
Strand, UK

George England is Professor of Mechanics and Structures in the University of London at King's College. In 1975 he was awarded the D.Sc (Eng) degree for research and analysis relating to time- and temperature-dependent behaviour of materials and structures. He maintains an active interest in long-term serviceability and safety of structures.

SUMMARY
This paper describes the observed serviceability behaviour and the analysis used to predict long-term performance, for a series of double cantilever 'T' structures built in Hong Kong during the early 1970's. The analysis incorporates creep, shrinkage and thermal strains for concrete, and prestress, relaxation and thermal strains for the prestressing steel. Temperature dependence of concrete creep and the daily and seasonal variations in bridge temperature distributions in depth are accounted for. The computer program also has provision for analysing the simultaneous effects of adding unbonded prestressing tendons to reduce the present unacceptable cantilever tip deflections.

RÉSUMÉ
L'article décrit le comportement en service et l'analyse utilisée pour prédire les performances à long terme de séries de structures construites à Hong Kong durant les années 1970. L'analyse prend en compte le fluage, retrait et les contraintes thermique pour le béton, et aussi la précontrainte, relaxation et contraintes thermiques pour l'acier de précontrainte. Le fluage du béton dépendant de la température, et des variations journalières et saisonnières des distributions de température en profondeur dans le pont sont considérées. Le programme informatique prévoit également l'analyse des effets simultanés des câbles de précontrainte à adhérence, pour réduire les flèches inacceptables aux extrémités de l'encorbellement.

ZUSAMMENFASSUNG
Diese Studie beschreibt das beobachtete Gebrauchsfähigkeitsverhalten, sowie die Analyse, welche angewendet wurde, um die Langzeitleistung für eine Reihe von Doppel-Ausleger-Strukturen (T), vorauszusagen, die in den frühen siebziger Jahren in Hong Kong gebaut wurden. Die Analyse beinhaltet Kriechverhalten, Schwinden und thermische Verformungen für Beton, sowie Belastungen, Lockerung und thermische Verformungen für Vorspannstahl. Die Temperturabhängigkeit des Betonkriechens und die täglichen und saisonbedingten Schwankungen der Brückentemperaturverteilung wurden berücksichtigt. Das Computerprogramm beinhaltet weiter eine Analyse der Gleichzeitwirkungen von zusätzlichen Spanngliedern ohne Verbund zur Reduktion der bestehenden unannehmbaren Abweichungen der Spitze des Auslegers.

1. INTRODUCTION

Between 1983 and 1986 a study was undertaken of a six span prestressed concrete bridge which had been constructed in Hong Kong in the early 1970's. The bridge consists of five 'T' units in which the 60 metre long cantilevers are integral with the piers but have no structural continuity between units. Observations and very limited monitoring prior to the study suggested that the tips of the cantilevers had deflected downwards excessively and concern was expressed that these deflections appeared to be increasing with time; Figure 3.

In order to investigate the behaviour in detail, vibrating wire strain gauges were surface mounted throughout the structure and thermocouples installed both inside and adjacent to the superstructure. The thermocouples were distributed to monitor both temperatures of the concrete box at various levels and the adjacent air temperatures and in addition to check on heat dissipation from the five 132kV electricity circuits carried through the superstructure box.

Once a month for over two years readings were taken from the strain gauges and thermocouples at hourly intervals during a 24hr period. During these monitoring periods precise levelling of the superstructure was carried out with the bridge closed to traffic at three times. Records were also obtained of the electrical load in the high voltage circuits. From these sessions typical variations in temperature, deflection and electrical loading were produced both for daily and yearly cycles. Figure 1 illustrates a typical set of readings obtained from the monitoring while Figure 2 and Table 1 show the yearly, seasonal and daily variations of temperature adopted for analysis.

In addition, traffic counts were undertaken with simultaneous reading of the strain gauges. This led to the production of a typical daily loading cycle and to an assessment of the loading growth since completion. It was concluded that the analysis should incorporate the full designed loading; Figure 4.

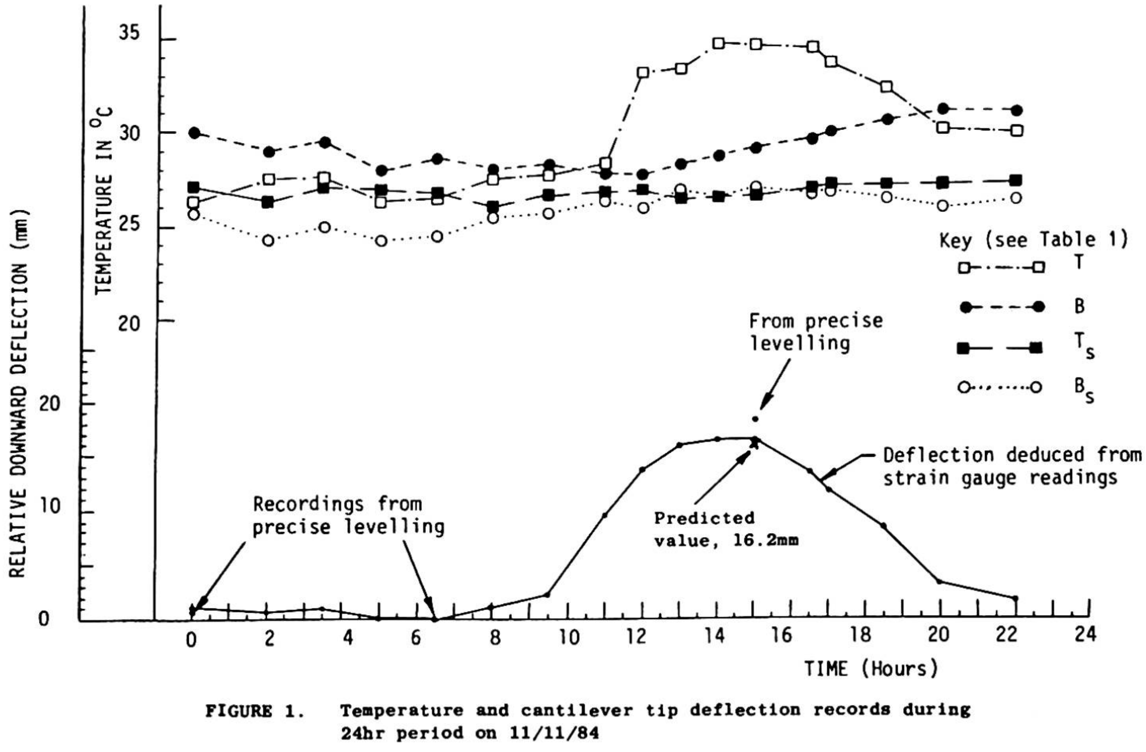


FIGURE 1. **Temperature and cantilever tip deflection records during 24hr period on 11/11/84**

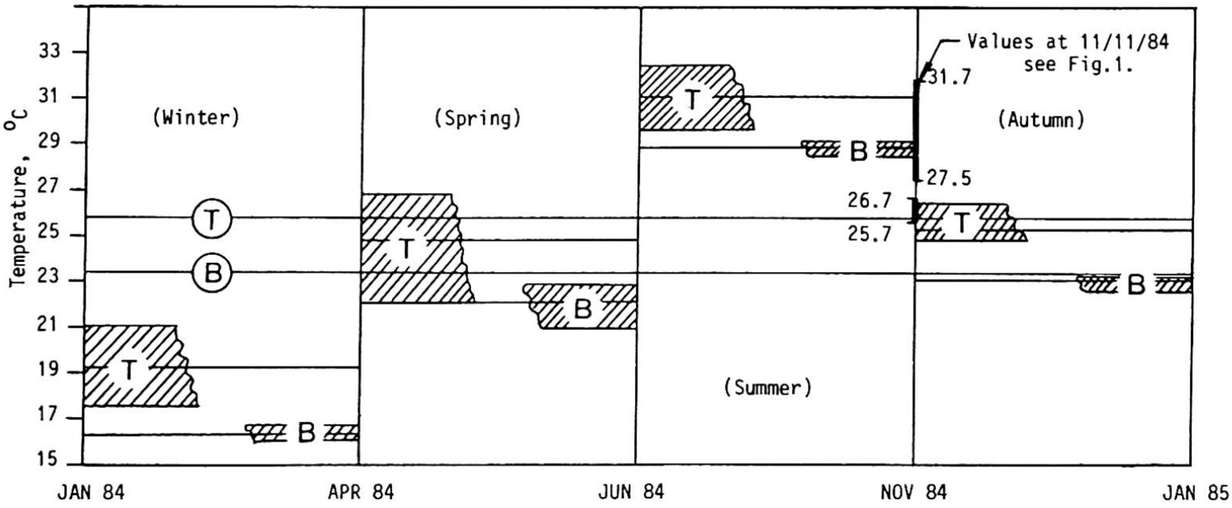


FIGURE 2. Mean upper and lower flange bridge temperatures: Yearly average; Seasonal average; and seasonally averaged Daily maximum and minimum variations.

2. ANALYSIS

A numerical step-by-step analysis in time was undertaken using a computer model of one of the cantilevers of a 'T' unit. The span was divided into 18 segments horizontally for which the section properties (e.g. dimensions, prestress, eccentricity) were known for each of the 19 bounding sections; Figure 3. Equilibrium and compatibility (plane section theory) equations for each segment were formulated such that creep, shrinkage and thermal strains of the concrete, and prestressing, relaxation and thermal strains of the steel could be incorpor -ated in a general 'initial' strain formulation. This permitted a 'standard' numerical calculation to be performed at every step of the analysis, to determine average values of centroidal strain, $\mathcal{E}$, and curvature, χ , for each segment. Numerical integration of these values along the span revealed the cantilever tip extensions and vertical deflections.

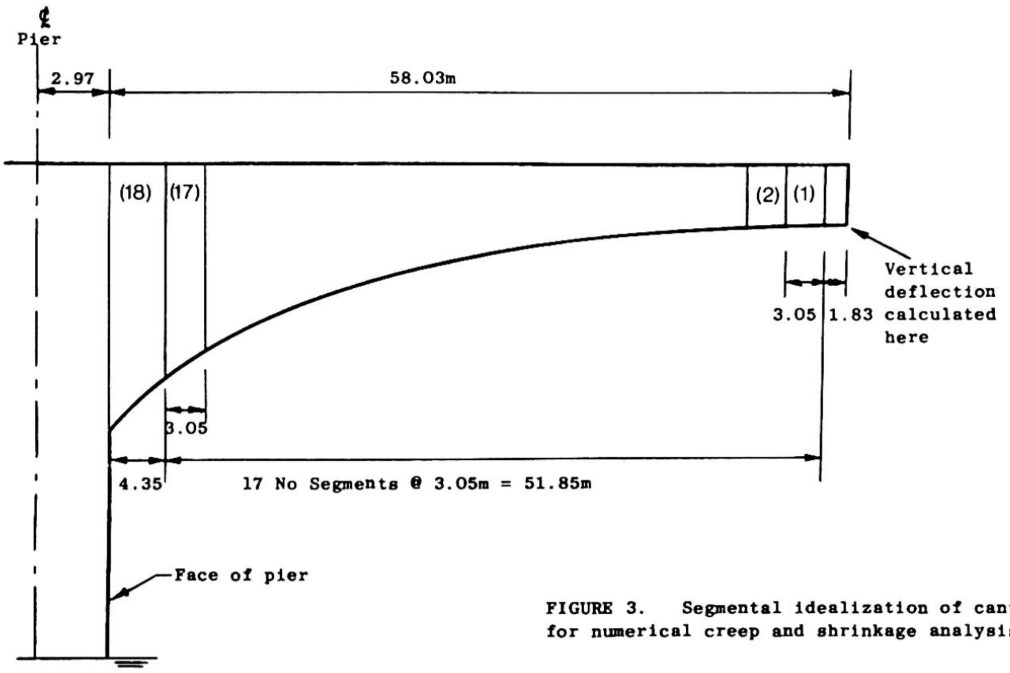


FIGURE 3. Segmental idealization of cantilever for numerical creep and shrinkage analysis.

Each segment was subdivided in depth and the stress associated with each slice (concrete) and prestressing steel (where appropriate) was derived from the calculated values of and together with the non-elastic strains appropriate to the step of the analysis being performed. The stresses were then used to determine the appropriate creep strains for the next step of the analysis; which then repeated but with changed values for the non-elastic strain components.

The prestress at the end of construction (November 1974) together with dead load, superimposed dead load, effective average live load and average yearly temperatures (Figure 2) defined the start of the time-dependent analysis. The effects of daily and seasonal temperature variations were computed from thermo-elastic analyses for temperature differences from average yearly values and were superimposed on the main time-dependent response as appropriate.

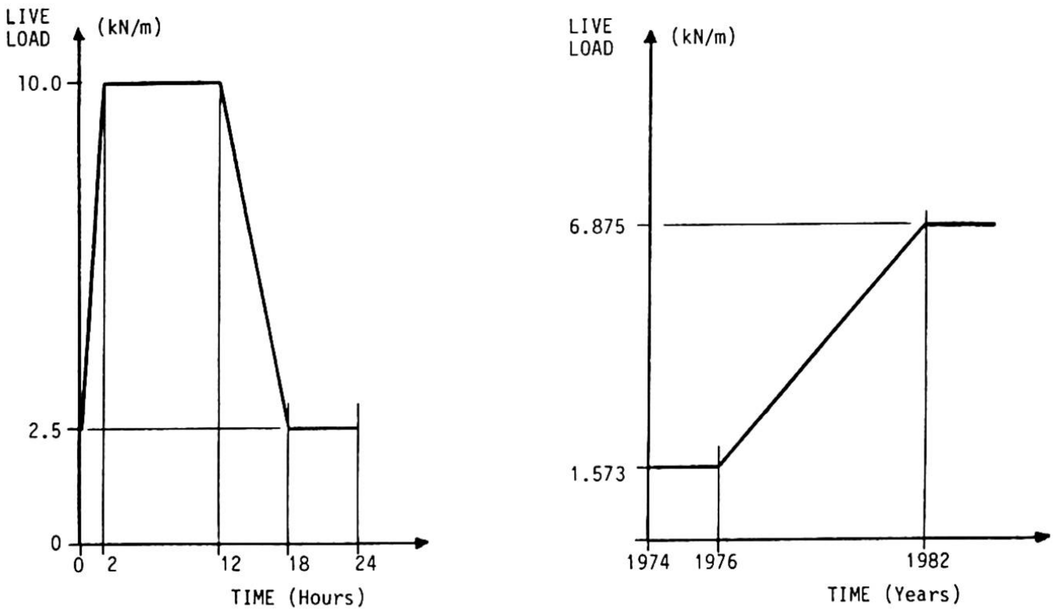


FIGURE 4. (a) Representative daily load cycle, 1982
(b) Mean daily live load variation since 1974. Design values.

3. MATERIAL DATA

The analyses were first performed using creep and shrinkage data extrapolated from tests carried out at the time of construction. Later, a survey was carried out and more recent data for Hong Kong concrete indicated that higher creep and shrinkage values should be adopted in design. The analyses were consequently repeated using these enhanced values. The temperature dependence of creep was taken into account by assuming a linear dependence on temperature in degrees Celcius. Table 2 shows the results of analysis and illustrates the separate significance of creep and shrinkage strains on tip displacements in the long term. The recorded tip displacement during 1984 is shown also for comparison.

4. DISCUSSION

The nature of the time-dependent analysis allows for the easy introduction of data which themselves change with time, e.g. live load (Figure 4), temperature and even the late introduction of additional prestress. A further benefit of the analysis is the ability to study the separate contributions to bridge deflections of prestress, temperature, loading (dead and live) and shrinkage; and thereby gain an appreciation of the sensitivity to particular parameters. Such knowledge is of importance for assessing the suitability of any remedial prestressing designed to reduce the long-term tip deflection.

Table 2 shows the recorded tip deflection in 1984 as being intermediate between the predictions based on 'extrapolated' creep and shrinkage test data and current 'design' strain values; the design values leading to a more pessimistic estimate. Table 3 shows calculated values for the average daily variations to tip deflect -ion due to temperature changes during the four seasons of the year. Additionally the maximum recorded daily deflection changes are shown for comparison. In order to predict the tip deflection for a particular day it is necessary to know the amount by which the maximum and minimum temperatures of that day differ from the seasonally averaged maximum daily values, and to then superimpose the deflection appropriate to these changes on the calculated values of Table 3. This exercise leads to good agreement with recorded values; a typical example comparison is given in Figure 1. These data highlight the need to recognise short term thermal displacement changes when taking on-site measurements for long-term performance. The predicted average daily change amounts to approximately 10mm, while the maximum daily change can exceed 20mm.

5. CONCLUSIONS

Within the limits of available data and the duration of the study, generally good agreement between predictions and on-site measurements has been obtained.

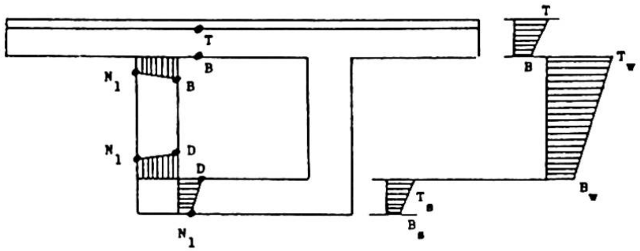
For the cantilever tip deflection calculations the incorporation of 'design' creep and shrinkage data has led to an overestimate, i.e. safe, whereas the use of 'extrapolated' test data led to underprediction.

The comparisons between measured and predicted deflections resulting from maximum daily temperature changes throughout the bridge depth were generally excellent.

In carrying out a site study of long-term bridge behaviour it is essential to give proper recognition to short-term (daily) temperature deflections, since these can mask completely the slow development of deflections due to concrete creep and shrinkage, and tendon relaxation.

SEASON		T	B	T_w	B_w	T_s	B_s	
Winter		19.55	18.86	17.46	16.35	16.63	16.06	Average seasonal temps.
Spring		25.30	24.32	23.15	22.13	22.28	21.98	
Summer		31.03	30.96	29.55	28.79	29.43	28.14	
Autumn		25.00	25.58	24.03	23.13	23.79	22.47	
Year Average		26.05	25.71	24.33	23.40	23.86	22.95	
Winter	Min	16.40	18.50	17.05	16.05	16.50	15.60	Seasonally averaged daily temp variations.
	Max	23.30	18.80	17.75	16.70	16.70	16.70	
Spring	Min	20.80	23.50	21.85	20.95	21.70	20.20	
	Max	29.30	24.30	23.85	22.90	22.40	23.40	
Summer	Min	28.50	30.70	29.15	28.50	29.40	27.60	
	Max	34.10	30.70	29.80	29.10	29.30	28.90	
Autumn	Min	22.90	25.70	23.65	22.65	23.70	21.60	
	Max	28.00	24.80	23.85	23.25	23.60	22.90	

TABLE 1. Yearly, seasonal, and daily temperature variations throughout bridge section. Temps in degrees Celcius



Data used in Analysis	Tip Deflection below Piers, mm			
	1974	1984	2016	2098
Extrapolated creep and shrinkage values	111.0	179.2	185.5	190.6
Extrapolated creep and design shrinkage values	-	197.3	206.5	213.9
Design creep and extrapolated shrinkage values	-	216.6	250.3	276.4
Design creep and design shrinkage values	-	249.3	288.5	320.3
Measured deflections	103.0	215.0		

TABLE 2. Comparison of Cantilever tip deflections as influenced by creep, shrinkage and temperature of concrete, and tendon relaxation. Ratios of 'design' to 'extrapolated values are; creep, 4·45; shrinkage, 1·60.

Season	Daily Change in Cantilever Tip Deflection (mm)		
	Calculated - averaged over 1984 season	Recorded	
		Max	Min
Winter	10.4	21.4	5.0
Spring	9.4	16.4	8.7
Summer	7.6	31.0	10.5
Autumn	5.7	20.0	6.8

TABLE 3. Daily changes in cantilever tip deflections.

Surveillance des enceintes nucléaires

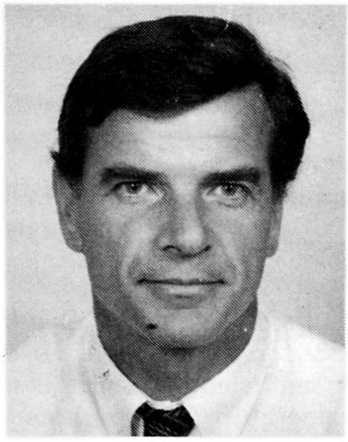
Überwachung von Sicherheitshüllen

Supervision of Nuclear Confinement Vessels

Henri ROUSSELLE
Ingénieur Auscultation
Electricité de France
Lyon, France

Henri Rousselle, né en 1945, est diplòmé de l'E.N.S.M. de Nantes. Après s'être occupé d'auscultation de barrages pendant une dizaine d'années. I est actuellement responsable du Centre d'Auscultation Nucléaire de Lyon qui a en charge l'ensemble du parc nucléaire d'Electricite de France.

Claude DUBS
Ingénieur étanchéité
Electricité de France
Lyon, France



Claude Dubs, né en 1943, obtient ses diplòmes à Strasbourg (T.S. Electrotechnique) et à Mulhouse (C.N.A.M.). Il s'occupe de conception et d'essais dans différents services, notamment pour la construction des centrales nucléaires de Bugey et Creys-Malville. Depuis 1983, il dirige la réalisation des essais d'étanchéité des enceintes nucléaires.

RÉSUMÉ
Electricité de France exploite un parc d'une quarantaine de tranches nucléaires à eau pressurisée; la protection de l'environnement en cas d'accident majeur est assurée notamment par une enceinte en béton précontraint: celle-ci doit conserver ses caractéristiques mécaniques et une bonne étanchéité sous l'effet de cette sollicitation. L'exposé montre les résultats généraux obtenus dans la surveillance de ces ouvrages lors des contrôles périodiques et permanents.

ZUSAMMENFASSUNG
Electricité de France betreibt circa vierzig Kernkraftwerksblöcke mit Druckwasserreaktoren. Der Umweltschutz im Fall eines Unfalls größeren Ausmaßes wird hauptsächlich durch eine Spannbetonhülle gewährleistet: diese Hülle muß, im Fall einer derartigen Beanspruchung, ihre mechanischen Eigenschaften und eine gute Undurchlässigkeit bewahren. Der Vortrag stellt die allgemeinen Ergebnisse dar, die aus der Überwachung dieser Vorrichtungen, mittels regelmäßiger und ständiger Kontrollen, hervorgegangen sind.

SUMMARY
Electricité de France operates around forty pressurized-water nuclear reactors; environmental protection in the event of a major accident is ensured most particularly by a prestressed concrete vessel: this vessel must be capable of retaining its mechanical characteristics and an efficient seal while subjected to these stresses. The paper illustrates the general results obtained in the supervision of these constructions during periodic and ongoing inspection processes.

1. INTRODUCTION

A la fin de l'année 1987, ELECTRICITE DE FRANCE exploite un parc de 44 tranches nucléaires à eau préssurisée -34 tranches de 900 MW et 10 tranches de 1300 MW- alors que 12 tranches de 1300 MW sont en construction. En cas d'accident majeur, la protection de l'environnement est assurée notamment par une enceinte en béton précontraint qui doit conserver ses caractéristiques mécaniques et une bonne étanchéité sous l'effet des sollicitations créées par la pression voisine de 5 bar absolus qui régnerait alors à l'intérieur de celle-ci. Elle constitue la 3ème barrière après la gaine du combustible et le circuit primaire principal.

Les tranches de 900 MW sont constituées d'une simple enceinte de 39 m de diamètre extérieur, 65 m de hauteur et 0,90 m d'épaisseur en partie courante du fût. Une peau métallique assure l'étanchéité. Celles du palier 1300 MW sont construites avec une double enceinte sans étanchéité métallique mais avec reprise des fuites indirectes dans l'espace entre enceinte. L'enceinte interne a un diamètre de 46 m, une hauteur de 66 m et l'épaisseur en partie courante du fût est selon le type considéré de 0,90 ou 1,20 m.

Des contrôles en fin de construction puis périodiquement et en permanence pour certains, sont donc réalisés afin de vérifier si l'enceinte répond aux critères d'étanchéité requis par la Sûreté Nucléaire et si elle conserve ses caractéristiques mécaniques sous l'effet des sollicitations dues à la pression d'une part et en fonction du temps d'autre part.

Le dispositif d'auscultation a donc été étudié de façon à pouvoir contrôler les déformations locales de l'enceinte, son état thermique, les déplacements d'ensemble de la structure et les variations de tension des câbles de précontrainte lors des essais de mise en pression ainsi que, pendant toute la durée d'exploitation de la centrale, pour suivre les évolutions à long terme. Il est constitué par :

- des extensomètres à corde vibrante,
- des thermocouples,
- des pendules qui mesurent les déplacements horizontaux selon quatre génératrices à trois niveaux différents,
- des fils invar horizontaux et verticaux,
- un système de nivellement hydraulique noyé dans le radier,
- un système de nivellement direct dans la galerie basse du radier,
- des dynamomètres équipant quatre câbles verticaux injectés à la graisse.

La méthode de mesure du taux de fuite (le taux de fuite est la variation relative de la masse de gaz contenue dans l'enceinte par unité de temps) est basée sur l'application pratique de l'équation des gaz parfaits qui établit une relation mathématique entre la masse d'une quantité de gaz, le volume qu'elle occupe, sa pression et sa température. Les résultats sont obtenus avec une précision tout à fait étonnante, de l'ordre de quelques Nm^3/h de débit de fuite sur un volume total de 250000 à 400000 Nm^3, compte tenu des difficultés inhérentes à la mesure avec précision et une bonne représentativité de toutes les grandeurs physiques entrant dans ces résultats. Ainsi, par exemple les variations de la température moyenne doivent être mesurées à quelques centièmes de °C près dans un bâtiment cloisonné de plus de 50000 m^3 où les différences de température entre le bas et le haut (60 m) peuvent atteindre plusieurs °C.

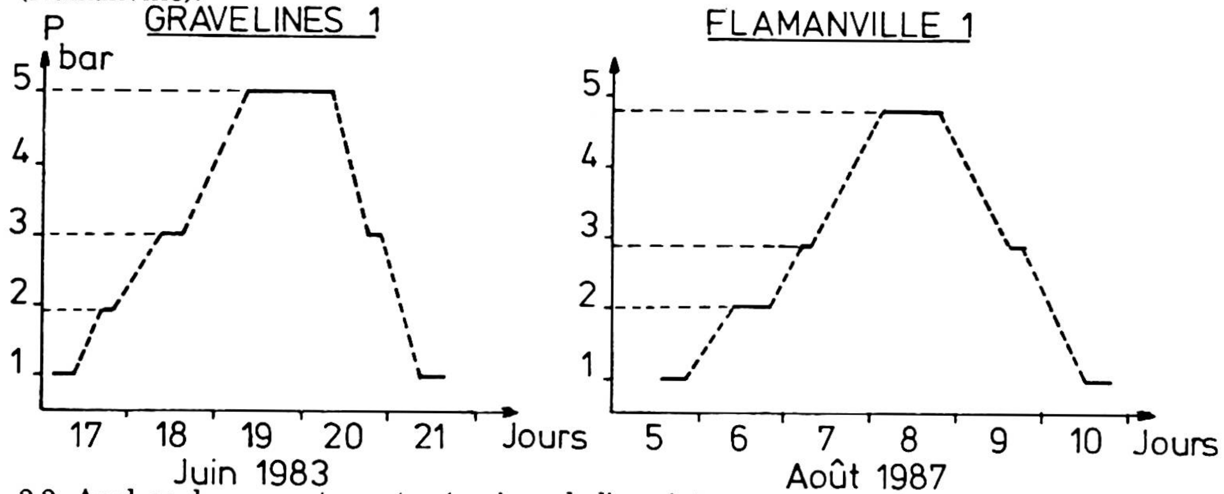
Deux systèmes complémentaires sont utilisés. Le premier permet de s'assurer, tous les dix ans, de la bonne qualité de l'étanchéité de l'enceinte dans les conditions équivalentes à l'accident de référence, le second vérifie uniquement, mais en permanence, la bonne configuration des "portes" de cette 3ème barrière.

2. CONTROLES PERIODIQUES PAR MISE EN PRESSION DE L'ENCEINTE

Un premier contrôle -essai préopérationnel- réalisé à la fin de la construction de l'enceinte avant sa mise en service constitue l'essai de reception de celle-ci. Le même type de contrôle est ensuite réalisé tous les dix ans, le premier d'entre eux intervenant au moment du premier rechargement du réacteur en combustible, soit approximativement deux à trois ans après l'essai préopérationnel. Fin 1987, 88 essais ont été réalisés, 66 dont 34 préopérationnels pour les tranches du palier 900 MW et 22 dont 16 préopérationnels pour les tranches du palier 1300 MW.

2.1 Déroulement d'un essai

A titre d'exemple, on présente ci-après les diagrammes de la variation de la pression interne en fonction du temps pour un essai d'une tranche de 900 MW (Gravelines) et d'une tranche de 1300 MW (Flamanville).



2.2 Analyse du comportement mécanique de l'enceinte

Les mesures d'auscultation réalisées lors des différents paliers de gonflage et de dégonflage permettent de vérifier la linéarité et la réversibilité des déformations sous l'effet des variations de pression. Lors de l'essai préopérationnel, on vérifie également la concordance des résultats des mesures avec ceux déterminés par les études. Les résultats des essais en exploitation sont analysés par comparaison avec ceux des essais précédents de la même tranche ainsi qu'avec ceux des tranches identiques du même site et ceux des tranches de même type des autres sites.

2.3 Contrôle de l'étanchéité de l'enceinte

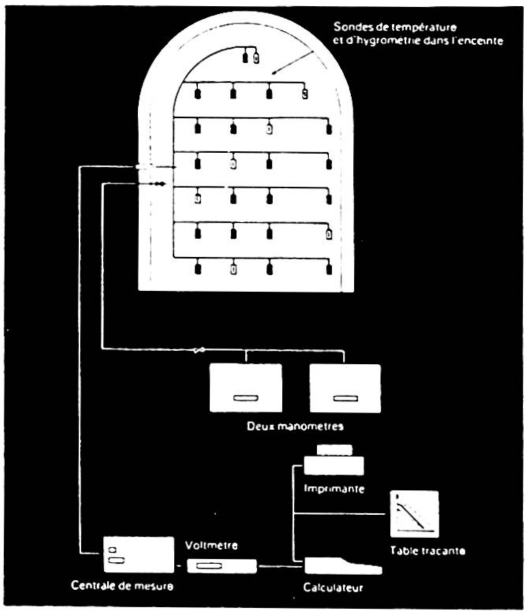


Schéma du dispositif de mesure du taux de fuite

Le système comporte plus de soixante dix capteurs de haute précision dont soixante capteurs de température protégés contre les effets de paroi (rayonnement), dix hygromètres au chlorure de lithium et deux capteurs de pression de très haute précision (10^{-4}) dont la cellule de mesure est maintenue à température constante. Pour réaliser régulièrement (tous les quarts d'heure) les mesures sur ces capteurs et effectuer les très nombreux calculs nécessaires, on fait appel à une chaîne d'acquisition automatique comprenant un voltmètre numérique de précision, un scanner et un calculateur pilotant cet ensemble et ses périphériques. La méthode de mesure, l'emplacement des capteurs et l'installation d'essai ont fait l'objet d'une étude spécifique et une procédure unique d'essai a ensuite été définie pour chaque type d'enceinte. Des mesures du taux de fuite sont réalisées aux différents paliers de pression précédant le palier nominal, permettant en cas de défauts graves, un retour rapide à la pression nominale. A la pression d'essai, le palier de mesure dure entre 15 heures (1300 MW) et 24 heures (900 MW) si les variations des paramètres sont faibles et stabilisées. En effet, on constate que les estimations du taux de fuite sont plus précises en fin de palier de mesure, lorsque les hétérogénéités de la température et de l'hygrométrie de l'air, dues au gonflage, se sont estompées.

Le critère d'étanchéité a été déterminé en prenant en compte notamment la température (très différente entre l'essai : quelques dizaines de °C, et l'accident : quelques centaines de °C)et le vieillissement du béton. Pour les enceintes simples à peau métallique type 900 MW, le taux de fuite doit être inférieur à 0,165%/j soit un débit de fuite de 16 Nm^3/h. Pour les enceintes doubles sans peau métallique type 1300 MW, le taux de fuite doit être inférieur à 1%/j, soit un débit de fuite de 150 Nm^3/h. A noter que cette dernière valeur est directement liée à la capacité de traitement du système de reprise des fuites dans l'entre-enceinte.

Par ailleurs, l'ensemble des traversées mécaniques et électriques de l'enceinte sont testées individuellement par mise sous pression avant l'épreuve globale, pour s'assurer de leur qualité individuelle et pour connaître la valeur du taux de fuite directe des enceintes doubles.

3. CONTROLES EN EXPLOITATION COURANTE

3.1 Contrôle du comportement mécanique

A partir de l'essai préopérationnel, les mesures d'auscultation sont faites une fois par trimestre sur l'ensemble du dispositif, sauf les mesures de nivellement direct qui sont réalisées en général une fois par an, de façon à contrôler les évolutions de l'ouvrage dans le temps.

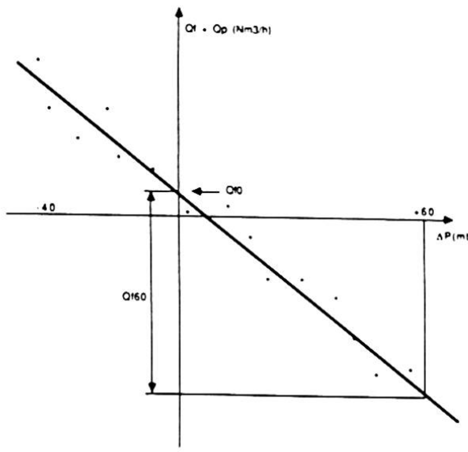
L'interprétation des résultats repose sur la comparaison du comportement actuel de l'ouvrage à celui du passé considéré comme normal; l'état de référence est celui observé lors de l'essai préopérationnel. L'expérience montre que les mesures brutes réalisées sur les enceintes résultent de la superposition de trois états principaux théoriquement indépendants :

- un état irréversible correspondant à une évolution du phénomène dans le temps, évolution qui peut avoir tendance à s'amortir (adaptation ou consolidation) ou à s'accélérer (dégradation),
- un état réversible correspondant à l'effet de la pression interne,
- un état réversible lié à la répartition des températures dans l'ouvrage,

auxquels il y a lieu cependant d'ajouter les erreurs expérimentales ainsi que les effets de toutes les autres causes secondaires que l'on néglige par simplification. Lorsqu'on dispose d'un échantillon de mesures suffisant (environ une trentaine), on réalise un traitement statistique qui consiste à rechercher le modèle de comportement permettant de séparer la part irréversible du phénomène observé de la part réversible.

3.2 Contrôle du débit de fuite en fonctionnement

Le système de contrôle en exploitation et en continu sous très faible pression (0 à quelques dizaines de mbar) utilise une version allégée du système de contrôle périodique (une dizaine de capteurs seulement) et un traitement mathématique des données différent, qui intègre des temps de mesures beaucoup plus longs. De telles mesures, déjà très difficiles à réaliser dans un bâtiment réacteur à l'arrêt, semblaient a priori pratiquement irréalisables dans une enceinte en exploitation, notamment en raison des variations importantes des températures et de l'hygrométrie induites par le fonctionnement du réacteur. Du fait des fuites d'air comprimé des actionneurs, la pression dans l'enceinte s'élève lentement jusqu'à sa valeur maximale d'exploitation. Il y a alors rejet, après contrôle, à l'atmosphère. La pression de l'enceinte varie selon une courbe en "dents de scie". La durée d'un cycle est d'environ 20 jours . L'analyse de la courbe d'évolution du débit de fuite en fonction de cette pression effective permet de donner un diagnostic complet de la configuration de l'enceinte.



Courbe $Qf_j = f(\Delta P_j)$(cycle de pression enceinte)

4. PRINCIPAUX RESULTATS

4.1 Essai de mise en pression de l'enceinte

4.1.1 Comportement mécanique

Pour un même type d'enceinte, l'ensemble des résultats obtenus présente une très bonne homogénéité et les écarts observés, pour une tranche donnée, entre les résultats de l'essai préopérationnel et ceux du premier essai en exploitation ne montrent pas de différence de comportement significative sous l'effet des variations de pression. Par ailleurs, la linéarité et la réversibilité des phénomènes ont toujours été mises en évidence.

On trouvera ci-dessous, pour chaque type d'ouvrage, les ordres de grandeur des variations des paramètres les plus caractéristiques pour la pression nominale :

	Enceinte 900 MW	Enceinte 1300 MW
- Augmentations de diamètre (mm)		
. en partie supérieure du fût	de 5 à 7	de 5 à 7
. au deux tiers de la hauteur	de 9 à 12	de 9 à 14
. au tiers de la hauteur	de 8 à 9	de 9 à 11
. du radier	négligeable	négligeable
- déformations locales (µm/m)		
. à mi-hauteur du fût : circonférencielles	de 200 à 220	de 220 à 270
verticales	de 60 à 80	de 60 à 100
- flèche du radier (mm)		
entre le centre et la périphérie	de 3 à 6	de 2 à 4

D'autre part, les variations de tension des câbles de précontrainte verticaux sont dans l'ensemble très modiques, inférieures à 50 kN pour chaque type d'enceinte.

4.1.2 Mesure du taux de fuite

L'incertitude obtenue sur le calcul de taux de fuite est de 0,005 %/j à 0,02 %/j soit de l'ordre du Nm^3/h. Le calcul des incertitude de mesure s'appuie sur une étude particulière qui révèle que la précision des mesures dépend non seulement des caractéristiques métrologiques des capteurs et appareils de mesure, mais surtout des conditions de température et d'hygrométrie régnant dans l'enceinte au moment de l'essai. Un phénomène parasite vient également perturber la mesure ; il s'agit de l'engazage, ou retard à la mise en pression, du béton et des structures qui se poursuit en se réduisant pendant le palier de mesure. La détermination de la durée des paliers tient compte de ce phénomène qui conduit à donner une valeur de fuite par excès. Les résultats annoncés sont donc conservatifs vis à vis de la sûreté.

Pour les enceintes simples à peau métallique, le critère est toujours respecté, mieux encore les taux de fuite sont en général au moins quatre à cinq fois plus faibles et souvent de l'ordre de grandeur de l'incertitude. Les seules exceptions ayant momentanément conduit à un résultat hors critère , provenaient d'un défaut de traversée mécanique. Dans ce cas, la précision de la méthode permet une localisation rapide du défaut et son élimination avant la fin de l'essai.

Pour les enceintes doubles, les résultats se situent généralement légèrement sous le critère. Il est remarquable de constater que l'on arrive à construire des enceintes sans peau d'étanchéité à ce niveau de qualité. Ce dernier exige une mise en oeuvre du béton tout à fait particulière faisant entre autre appel à un système de contrôle de l'étanchéité des reprises à l'avancement et à leur injection éventuelle. Néanmoins, sur certains sites la "porosité " trop importante des parois a nécessité la mise en place d'une étanchéité complémentaire sous forme d'un enduit résine sur le parement interne. Des investigations supplémentaires ont été réalisées pour améliorer la connaissance des chemins de fuites et se rapprocher plus encore des conditions accidentelles. Il s'agit de la mesure du taux de fuite après immersion totale du bas de l'enceinte (cas de rupture du circuit primaire). Les résultats sont sensiblement améliorés de l'ordre de 10 à 20 %.

4.2 Evolution des paramètres en fonction du temps

4.2.1 Comportement mécanique

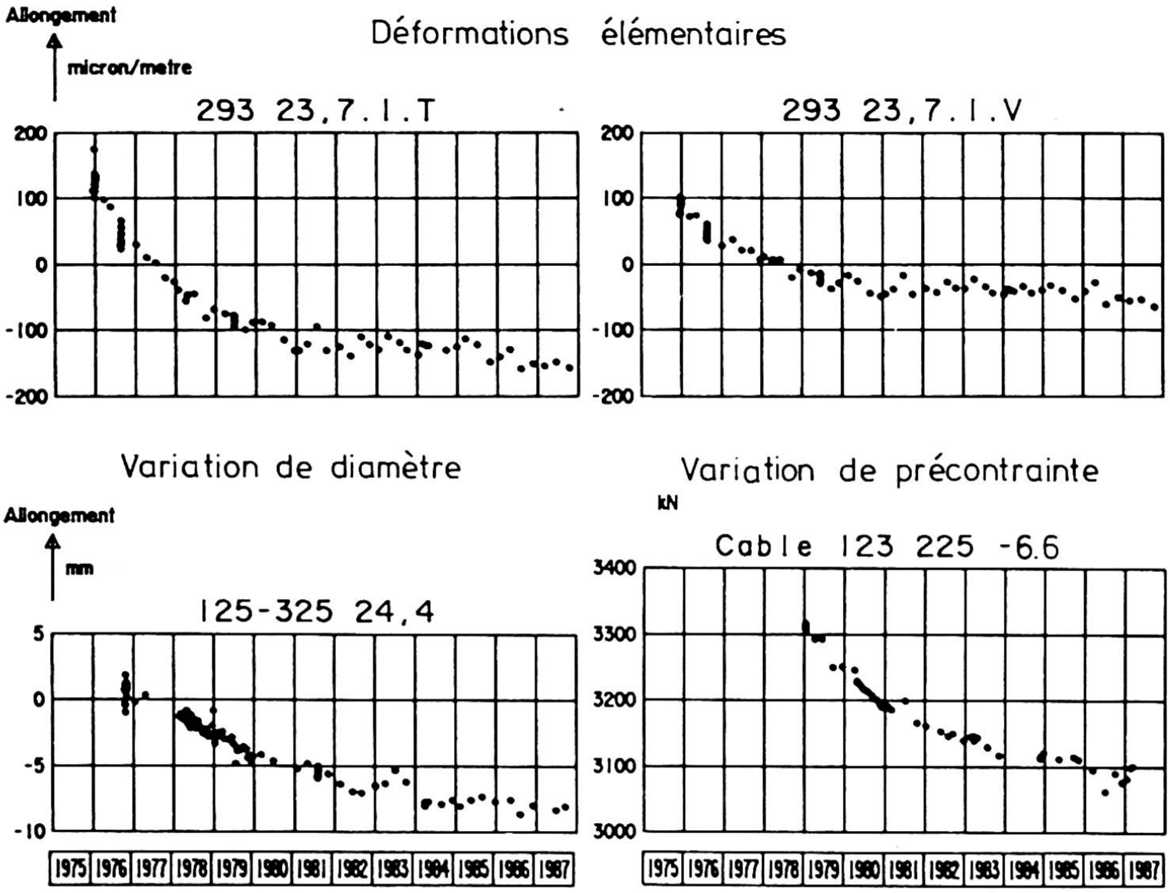
Le comportement des enceintes qui reste très directement comparable d'un ouvrage à l'autre est caractérisé par des raccourcissements de la plupart des grandeurs mesurées. Ils sont dus essentiellement aux phénomènes de retrait et fluage du béton qui se manifestent plus particulièrement au cours des cinq à six premières années de la vie de l'ouvrage. L'expérience acquise à ce jour montre qu'au-delà, ces évolutions sont nettement ralenties et tendent souvent vers un amortissement complet.

De la même façon, en ce qui concerne la précontrainte, on observe logiquement une diminution de la tension initiale due au retrait et au fluage du béton ainsi qu'au phénomène de relaxation de l'acier. Ces évolutions qui s'amortissent également après cinq à six ans d'exploitation sont toujours restées en-deçà des valeurs maximales admises par les notes de calcul.

Par ailleurs, les tassements de l'enceinte, dont la connaissance est importante pour juger des déplacements relatifs avec les autres bâtiments de l'ilôt nucléaire qui sont également contrôlés, varient évidemment d'un site à l'autre en fonction de la nature du sol de fondation et s'accompagnent parfois d'un léger basculement.

On donne ci-après les amplitudes des évolutions caractéristiques observées sur les premières enceintes de 900 MW dont les essais préopérationnels ont été réalisés de 1975 à 1979 ainsi que quelques exemples de graphiques de surveillance.

- diminutions de diamètre (mm)					
. en partie supérieure du fût		de	6	à	8
. au deux tiers de la hauteur		de	8	à	10
. au tiers de la hauteur		de	8	à	10
. du radier		de	2	à	3
- déformations locales (µm/m)					
. dans le radier : radiales et circonférencielles		de	30	à	50
. à mi-hauteur du fût	circonférencielles	de	200	à	300
	verticales	de	130	à	250
- diminution de la tension de précontrainte (kN)		de	150	à	200



4.2.2 Contrôle de l'étanchéité

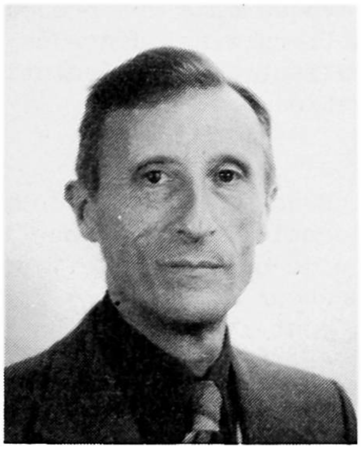
Il n'y a pas d'évolution notable des taux de fuite entre le premier et le second essai de mise en pression. (1ère épreuve décennale en 1989).
En ce qui concerne la Surveillance en continu en Exploitation du Taux de fuite des ENceintes, le système appelé SEXTEN est installé ou en cours d'installation sur toutes les centrales. Ce système est sollicité chaque fois qu'il détecte automatiquement une augmentation anormale des fuites. Un programme spécifique de recherche de fuite est alors utilisé qui permet, grâce à sa sensibilité, une recherche rapide du défaut de configuration et un retour à l'exploitation normale. Ce système double, avec plus d'efficacité, la méthode dite "manuelle" déjà en place depuis la divergence.

Fissures de fatigue dans les viaducs métalliques démontables

Ermüdungsrisse in demontierbaren Stahlbrücken

Fatigue Cracks in Demountable Steel Bridges

Pierre MEHUE
Ing. Div. TPE
SETRA
Bagneux, France



Affecté au Service Central d'Etudes Techniques puis au Service d'Etudes Techniques des Routes et Autoroutes où il a occupé les fonctions de Chef d'Arrondissement à la Division des Ouvrages en Métal du Département des ouvrages d'Art.

RÉSUMÉ
De très nombreux viaducs métalliques démontables comportant des platelages en dalle orthotrope ont été construits en France entre 1970 et 1980. Après la découverte en 1977 de fissures de fatigue dans les tôles de roulement de ces platelages, une surveillance accrue des ouvrages a permis de recenser diverses formes de fissures, ainsi que d'observer et quantifier leur développement sur plusieurs viaducs soumis à des conditions d'exploitation particulièrement sévères. A partir de cela ont pu être mises au point une méthodologie d'évaluation, une stratégie d'intervention et une politique de réparation adaptées aux différents cas.

SUMMARY
In the period 1970–1980 many demountable steel bridges with orthotropic decks have been constructed in France. In 1977 fatigue cracks were discovered in some decks. Detailed investigations enabled various types of crack to be identified and their development observed in several highly stressed bridges. Based on this work a procedure for determining the defects and a method of repair adjustable to the different types of damage could be developed.

ZUSAMMENFASSUNG
In der Jahren 1970 bis 1980 wurden in Frankreich zahlreiche demontierbare Stahlbrücken mit orthotropen Fahrbahnplatten erstellt. 1977 wurden in gewissen Fahrbahnplatten Ermüdungsrisse entdeckt. Ausgedehnte Untersuchungen ermöglichten die Feststellung verschiedener Rissformen sowie die Beobachtung ihrer Entwicklung an mehreren besonders stark beanspruchten Brücken. Davon ausgehend konnte ein Konzept zur Feststellung der Defekte und ein Vorgehen für die Reparaturen erstellt werden, abgestimmt auf die verschiedenartigen Schadenfälle.

1 - INTRODUCTION

Au cours de la précédente décennie ont été construits en France une centaine de viaducs métalliques démontables pour passages surélevés provisoires destinés à résoudre, temporairement mais rapidement, les problèmes de circulation posés par certains carrefours où l'aménagement définitif ne pouvait être entrepris avant plusieurs années.
Ces viaducs, dont la surface totale mise en oeuvre avoisinait 120.000 m^2 étaient tous constitués d'éléments de tablier métalliques comportant des platelages en dalle orthotrope avec des tôles de roulement de 10 mm d'épaisseur.
Sur plusieurs de ces ouvrages, qui supportaient un trafic abondant et comportant une forte proportion de véhicules lourds, a été constatée, quatre à cinq ans après leur mise en service, l'existence de nombreuses fissures de fatigue dans ces tôles de platelage, comme évoqué dans une précédente communication [1].
Une surveillance accrue des tabliers ainsi dégradés a permis de recueillir diverses informations sur la localisation et l'évolution de ces fissures, ainsi que sur l'efficacité des réparations effectuées.

2 - EMPLACEMENT DES FISSURES

Les éléments de tablier des viaducs métalliques démontables sont pratiquement de deux types qui ne diffèrent essentiellement en partie courante que par l'écartement des poutres et le nombre de nervures, le type I étant de beaucoup le plus répandu (Fig. 1). D'une longueur allant de 6 à 30 m pour une largeur unitaire de 3,50 m, ces éléments, dont la disposition longitudinale dépend des conditions de franchissement, peuvent être utilisés séparément pour les viaducs à voie unique ou assemblés transversalement pour constituer des viaducs à deux ou trois voies, à circulation unidirectionnelle ou bidirectionnelle.
Les fissures décelées jusqu'à ce jour, au nombre de trois cents environ, affectent surtout l'assemblage par soudure sur la tôle de platelage des nervures trapézoïdales qui sont situées à l'aplomb des zones de passage des roues des véhicules lourds. Elles sont donc relativement symétriques dans le cas d'ouvrages à voie unique, légèrement décalées vers l'axe de la chaussée dans le cas de viaducs à deux voies bidirectionnels en raison de l'effet de paroi dû aux dispositifs de retenue, et présentent un décalage encore plus marqué vers la gauche dans le cas de viaducs unidirectionnels à deux ou trois voies (Fig. 1).
Par rapport au cordon de soudure d'assemblage, réalisé sans préparation préalable du bord de nervure par chanfreinage partiel, la fissure peut se trouver :
- soit à la racine du cordon, pour traverser la tôle de platelage et déboucher sous le revêtement de chaussée,
- soit à la base du cordon, et traverser la joue de la nervure pour déboucher à l'intérieur,

selon les ouvrages, les éléments de tablier et les nervures concernées (Fig. 2).
Le premier cas, qui est le plus fréquent, est aussi le plus facile à repérer, car d'une part la détérioration locale du revêtement de chaussée met rapidement la fissure à nu, et d'autre part l'infiltration des eaux de ruissellement, qui

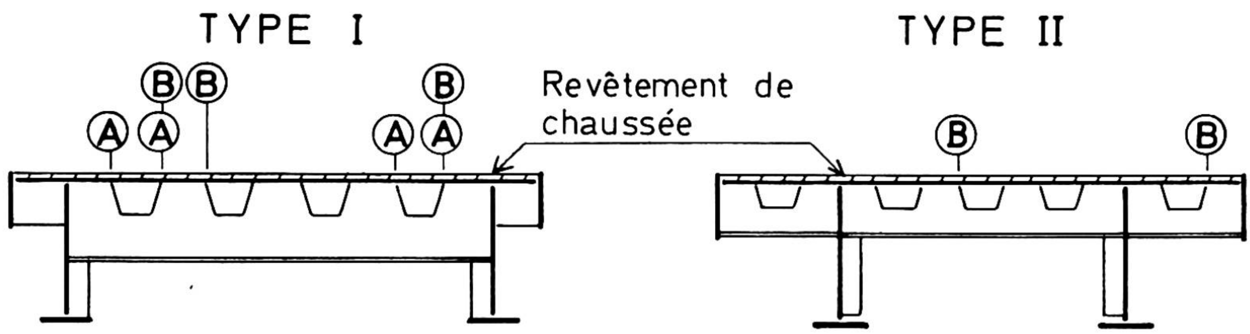


Fig. 1 - Coupes transversales types des tabliers et emplacements des fissures. Le repère A correspond aux viaducs à voies unique, le repère B aux viaducs à deux ou trois voies.

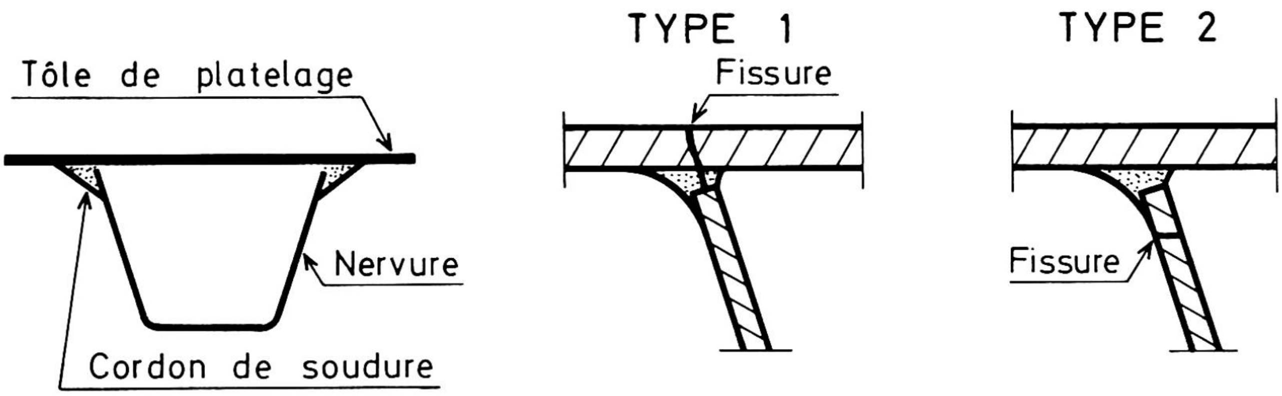


Fig. 2 - Emplacement et position des fissures

remplissent très vite la nervure, maintient entre les lèvres de la fissure une humidité permanente mise en évidence, même lors de fortes chaleurs, par un suintement constant de gouttes d'eau souvent accompagné d'un dégagement de bulles d'air.
Dans le second cas, qui ne peut s'observer que par dessous, le mauvais éclairage et l'état généralement médiocre des peintures de finition rendent souvent délicate la découverte des fissures, à moins que celles-ci ne quittent la base du cordon pour se prolonger dans la joue de la nervure.
Enfin la distribution des fissures sur les éléments de tablier est totalement aléatoire, car elles peuvent affecter les plus longs comme les plus courts, et apparaitre aussi bien vers la mi-portée qu'à proximité des appuis ; de même pour les panneaux de platelage où il est possible de découvrir des fissures en n'importe quel endroit, tant entre les pièces de pont qu'à leur aplomb.

3 - CARACTERISTIQUES DES FISSURES

3.1 - Longueur

La longueur des fissures est très variable et dépend évidemment du stade de développement qu'elles ont atteint au moment de leur découverte, leur repérage étant souvent tributaire des impératifs d'exploitation du viaduc (possibilité de fermer l'ouvrage à toute circulation durant quelques heures), des conditions d'inspection (facilité d'accès, moyens d'investigation, etc.) ainsi que de l'état du revêtement de chaussée pour les fissures de type 1 ou de la protection anticorrosion pour les fissures de type 2.
Difficilement décelables visuellement, en raison de leur finesse, avant qu'elles n'atteignent 25 mm, la plupart des fissures avaient une longueur comprise entre 120 et 650 mm, avec des lèvres assez nettement disjointes dans leur partie centrale. Si les exemples dépassant le mètre n'étaient pas rares, des longueurs supérieures à 2 m n'ont été mesurées qu'exceptionnellement, résultant alors de l'allongement puis de la jonction de deux ou trois fissures successives. Dans ces derniers cas, les fissures étaient très marquées, avec des lèvres épaufrées et parfois dénivelées l'une par rapport à l'autre (Fig. 4).

3.2 - Tracé

Alors que les fissures de type 2 ne diffèrent en général que par leur longueur, les fissures débouchant à la surface de la tôle de platelage offrent une assez grande variété de tracés parmi lesquels il est possible de recenser six figures élémentaires principales (Fig. 3) :
- fissure simple, approximativement rectiligne ou vaguement arquée,
- fissure sensiblement rectiligne, se terminant par des arcs de cercles de 30 à 50 mm de rayon,
- fissure légèrement sinueuse, présentant une extrémité bifide,
- fissure crénelée, dont les redans ont une profondeur de l'ordre de 3 à 4 millimètres,

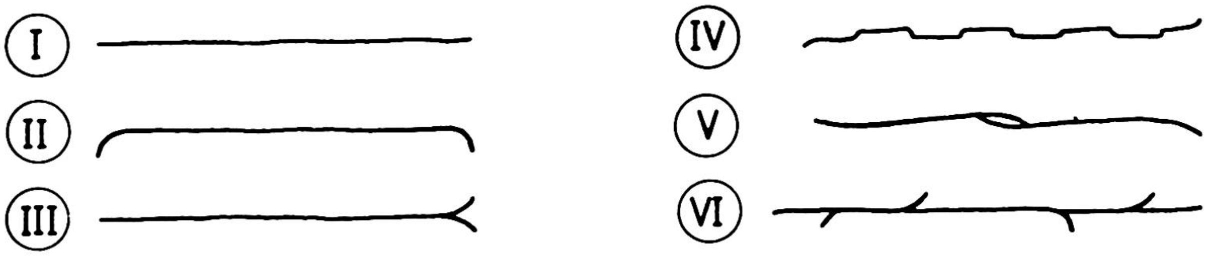


Fig. 3 - Principaux types de tracés de fissures

- fissure ondulante, isolant dans sa partie centrale une lentille de métal de 3 à 5 millimètres de longueur,
- fissure barbelée, comportant de nombreuses amorces biaises de 20 à 50 millimètres de longueur,

dont certaines peuvent bien entendu se retrouver dans des fissures de caractère plus complexe.
Il semblerait d'ailleurs, d'après les relevés effectués sur plusieurs ouvrages, que les figures I, II et III ne constituent que des étapes dans la réalisation de la figure VI.

3.3 - Evolution

La manière dont se développent les fissures, à partir du moment ou elles ont été decelées, est difficile à établir dans la mesure où :
1 - toute investigation précise exige la fermeture de l'ouvrage à la circulation, ce qui ne peut être fait trop souvent en raison des impératifs de service,
2 - de nombreux facteurs (présence du revêtement de chaussée, formation de rouille en plaque, irrégularités ou défauts des cordons de soudure, etc.) rendent délicate la détermination des extrémités,
ce qui a souvent conduit à accueillir avec prudence les valeurs données à l'issue de visites rapides ou d'inspections sommaires.
Il faut dire que, tout comme leur apparition, l'évolution des fissures est très aléatoire, et que si certaines d'entre elles ne cessent de s'allonger, d'autres toutes proches ne connaissent pratiquement pas de modifications.
Sur quelques ouvrages il a néanmoins été possible de procéder à quelques mesures précises dont le tableau 1 donne un exemple qui met en évidence des résultats très variés ainsi que des taux d'accroissement annuel pouvant atteindre 100 %.

4 - METHODOLOGIE D'EVALUATION

Si la découverte des premières fissures en 1977 a créé quelque émoi chez les maîtres d'ouvrage concernés, il est vite apparu que leur présence, pour ennuyeuse qu'elle fût vis-à-vis de la maintenance et de l'entretien, n'était, dans la majorité des cas, pas de nature à attenter à la sécurité des usagers. En effet les visites d'inspection, effectuées de façon aussi fréquente et régulière que le permettaient les servitudes d'exploitation, ont, à de rares exceptions près, permis de constater que les dégradations étaient relativement limitées et évoluaient de façon assez progressive. Il a donc en général été possible de procéder à un examen réfléchi des risques encourus ainsi que des réparations pouvant

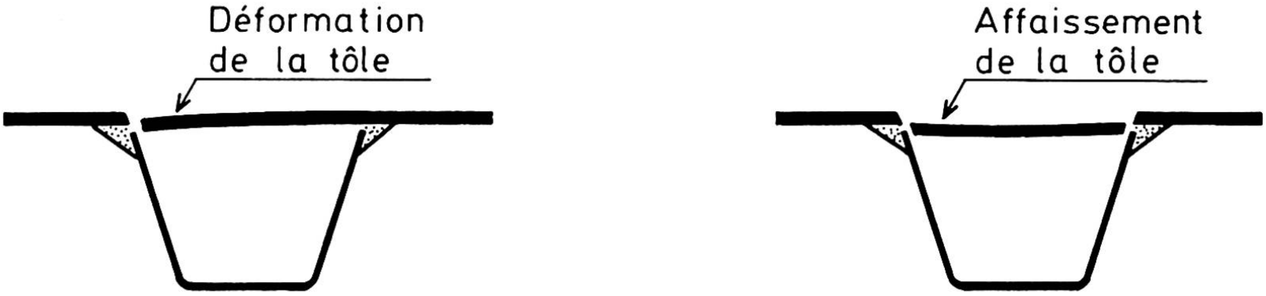


Fig. 4 - Déformation des lèvres des fissures et affaissement de la tôle

FISSURE	LONGUEUR (MM)		ALLONGEMENT (MM)			ACCROISSEMENT %
	LE 061083	LE 240584	EN TETE	EN PIED	TOTAL	
1	300	340	40	-	40	13
2	130	260	40	90	130	100
3	270	270	-	-	-	0
4	330	370	30	10	40	12
5	330	410	30	60	90	24
6	240	240	0	0	0	0
7	200	245	25	20	45	23
8	180	250	20	50	70	39
9	260	480	70	150	220	85
10	350	420	10	60	70	20
11	200	320	70	50	-	60

Tableau 1 - Evolution des fissures

être entreprises, puis de mettre au point un programme d'intervention et de réparation.
En pratique la situation n'est devenue préoccupante que dans les quelques cas où, par suite d'une évolution rapide, le développement de fissures parallèles voisines conduisait à la formation de lanières de tôle de platelage de 300 mm environ de largeur pour 1 m de longueur, qui étaient susceptibles de se déformer de façon importante au passage des véhicules lourds ou sous l'action de contraintes thermiques, et de provoquer ainsi des accidents (Fig. 4).

5 - MAINTENANCE DES OUVRAGES ET REPARATION DES FISSURES

La première mesure envisagée pour limiter la propagation de ces fissures de fatigue a bien entendu été d'en supprimer la cause essentielle, à savoir le passage des véhicules lourds. Mais dans de très nombreux cas, et en dépit de l'existence de voies latérales au sol, cette mesure n'a pu être appliquée, du moins immédiatement, à cause des difficultés qu'elle soulevait ; ce qui a conduit à essayer de mettre au point diverses méthodes de réparation en fonction de la gravité des dommages d'une part, et des possibilités d'intervention sur l'ouvrage d'autre part.
C'est ainsi qu'ont été effectuées :
1 - pour les fissures simples, des réparations par :
- ressuage, élimination du défaut par meulage, puis soudage et arasage,
- pose d'un couvre-joint fixé par cordon d'angle périphérique,

ne demandant que des interruptions de circulation de quelques heures,
2 - pour les fissures parallèles voisines ou les fissures de caractère complexe, des réparations par remplacement :
- du morceau de tôle de platelage dégradée,
- du morceau de tôle de platelage concernée et de nervure correspondante,

exigeant cette fois des interruptions de trafic de deux à trois semaines, avec des fortunes diverses.
Si l'on excepte l'utilisation des couvre-joints dont les cordons de fixation se sont rapidement fissurés, les réparations ont dans l'ensemble plutôt donné satisfaction dans la mesure où elles étaient exécutées par du personnel qualifié travaillant dans les conditions requises. Pour les réparations simples, les échecs constatés (réapparition de la fissure au même emplacement ou apparition d'une nouvelle fissure en bordure de la zone affectée thermiquement) paraissent essentiellement dus à des interventions rapides, réalisées sans préparation ni contrôle. Les meilleurs résultats semblent avoir été obtenus dans les cas, très peu nombreux il est vrai, de remplacement de morceaux de platelage, mais il s'agit là d'une méthode très compliquée, très contraignante et très coûteuse.

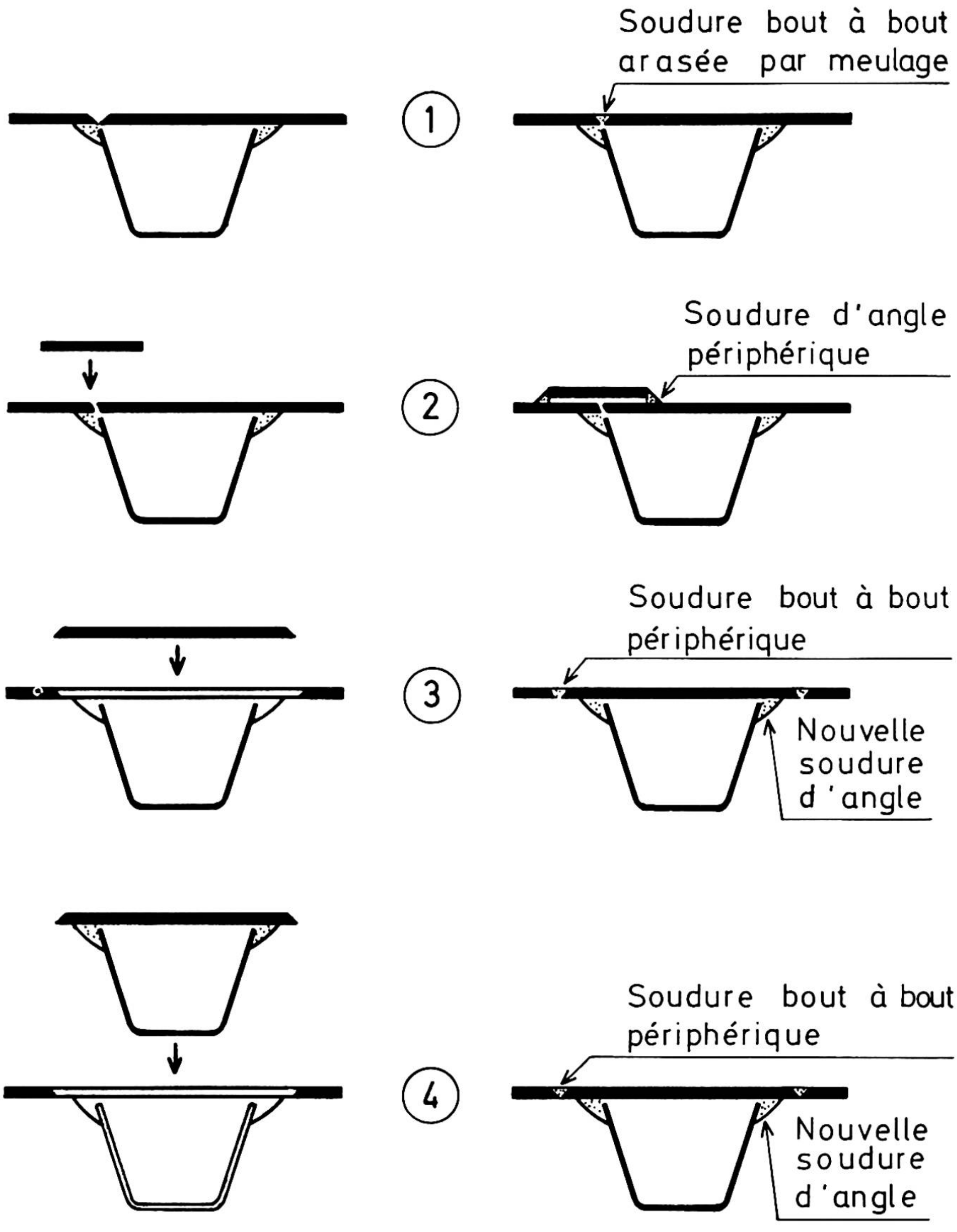


Fig. 5 - Procédés de réparation des fissures

6 - CONCLUSION

Le suivi des viaducs métalliques démontables sur une quinzaine d'années a ainsi permis de recueillir un certain nombre de données intéressantes sur le comportement de platelages métalliques orthotropes sous sollicitations réelles.

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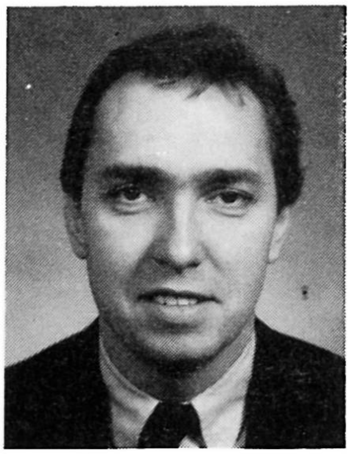
Zustandsbewertung von Holzbauteilen in Wohnbauten des 19. Jahrhunderts

Evaluation of Wooden Constructions in Residential Buildings of the 19th Century

Etat des éléments en bois d'habitations du XIX^{e} siècle

Andreas KOLBITSCH
Dr. techn.
Ö.G. Erhaltung von Bauten
Wien, Österreich



Andreas Kolbitsch promovierte als Bauingenieur an der Technischen Universität Wien. Bis 1986 war er als Assistent am Institut für Hochbau und Industriebau der Technischen Universität Wien tätig; diesem Institut gehört er als Lehrbeauftragter weiterhin an. Seit 1987 beschäftigt er sich als Geschäftsführer der ÖGEB mit konstruktiven Fragen der Althaussanierung.

ZUSAMMENFASSUNG
Der Untersuchung und Zustandsbewertung von Holzbauteilen kommt aufgrund der festgestellten Schäden immer größere Bedeutung zu. Der vorliegende Bericht befaßt sich vor allem mit der Untersuchung von verdeckten Holzkonstruktionen.

SUMMARY
The increasing importance of analysis and evaluation of wooden constructions is related to the considerable amount of damages. This report is concerning with the problems of the investigation of imbedded wooden structures such as ceiling constructions.

RÉSUMÉ
En raison des dommages constatés, il faut accorder de l'importance à l'expertise et à l'évaluation de l'état des éléments de construction en bois. La présente étude traite essentiellement de l'évaluation des constructions couvertes en bois.

1. GRUNDSÄTZLICHE PROBLEMATIK

In der zweiten Hälfte des 19. Jahrhunderts kam es, bedingt durch das rasante Wachstum der städtischen Bevölkerung, zu einer regen Wohnbautätigkeit. In Österreich wird diese städtebauliche Phase, deren Ende mit dem des Ersten Weltkrieg zusammenfällt, unter dem Begriff "Gründerzeit" zusammengefaßt.

Trotz der Zerstörungen während des Zweiten Weltkrieges sind noch heute große, zusammenhängende Gebiete - speziell in Wien - geschlossen mit Wohnhäusern dieser Entstehungszeit bebaut.

Da in den letzten Jahren der Trend von der Stadterweiterung zur Revitalisierung von Altbauten umschlug, konzentrieren sich die Überlegungen auf Möglichkeiten der Wohnstandardanhebung in derartigen Gebäuden. Dabei fällt der Zustandsbewertung und Untersuchung tragender Bauteile als Grundlage für Sanierungsentscheidungen besonderes Gewicht zu.

Während die massiven Teile der Tragkonstruktion in den meisten Fällen kaum Schäden aufweisen, mußten im Zuge zahlreicher Gebäudeanalysen an tragenden Holzbauteilen Schäden - die in den meisten Fällen auf das Einwirken von Feuchtigkeit zurückzuführen sind - festgestellt werden.

2. SCHEMATISCHE ZUSTANDSBEWERTUNG

Um die Untersuchung dieser Bauteile, die in der Regel einen relativ hohen Aufwand erfordert, zu vereinheitlichen und zu standardisieren, wurden Erhebungsbögen entworfen, die in der praktischen Anwendung eine erhebliche Zeiteinsparung, sowohl bei der Vorbereitung, als auch bei der Auswertung von Untersuchungen als Grundlage der Konzeption von Sanierungsmaßnahmen brachten.

Die bei der Erstellung dieser Erhebungsbögen wichtigsten Kriterien wurden folgendermaßen definiert:

- Vollständige Erfassung der zu untersuchenden Bauteile;
- Berücksichtigung vorhandener Unterlagen;
- standardisierte Schadensbewertung.

Einen nicht zu vernachlässigenden zusätzlichen Vorteil derartiger Erhebungsbögen stellt die Möglichkeit dar, die Vollständigkeit und Qualität der ausgeführten Sanierungsarbeiten zu überprüfen.

3. METHODEN ZUR ÜBERPRÜFUNG VON HOLZTRAGWERKEN

3.1 Untersuchung zugänglicher und verdeckter hölzerner Konstruktionsteile

Zur Untersuchung von Holztragwerken wurden in den letzten Jahren, ausgehend von Entwicklungen zur Gütefeststellung von frischem Bauholz in der Forstwirtschaft und von verfeinerten Laboruntersuchungen, eine Reihe von Methoden entwickelt, die sich jedoch bei der Überprüfung eingebauter Holztragwerke und unzugänglicher Konstruktionen nur zum geringen Teil eignen.

Die folgende Tabelle versucht, einige der zahlreichen Methoden hinsichtlich ihrer Eignung zur Überprüfung verdeckt liegender Konstruktionen (im speziellen zur Untersuchung von Deckenkonstruktionen) zu bewerten:

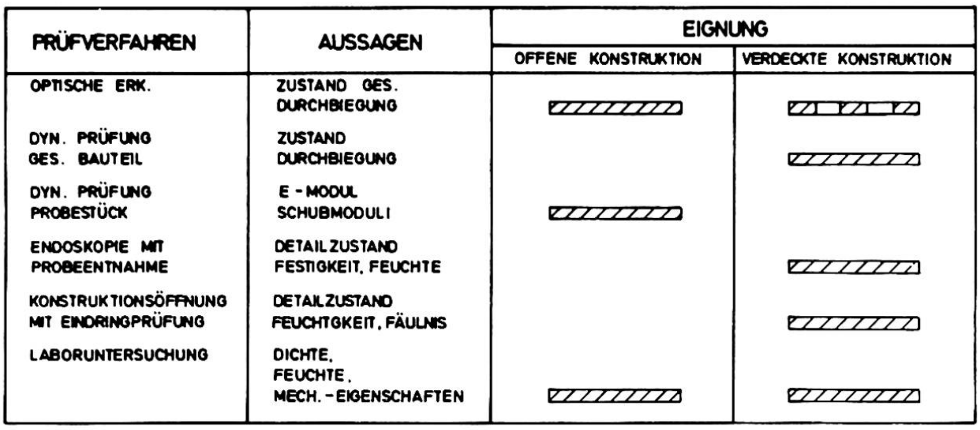


Tabelle 1 Möglichkeiten der Untersuchung von Konstruktionsteilen aus Holz

3.2 Methoden zur Untersuchung hölzerner Deckenkonstruktionen

Von den in Tabelle 1 angeführten Untersuchungsmethoden eignen sich nur einige für die Beurteilung hölzerner Deckenkonstruktionen, da bei derartigen Bauteilen die Träme nicht direkt zu erreichen sind und speziell die Auflagerbedingungen der Holzbalken in den wenigsten Fällen von vorneherein bekannt sind.

Daher hat sich - aufgrund der in praktischen Untersuchungen gewonnenen Erfahrungen - folgende, nach dem Grad der mit der Untersuchung verbundenen Eingriffe gereihte Palette von Methoden als günstig erwiesen:

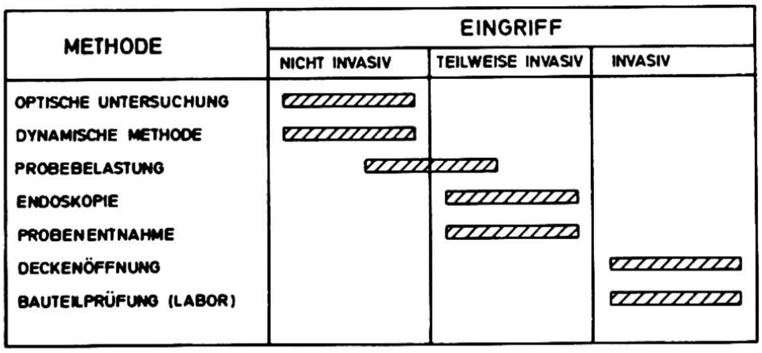


Fig.2 Mögliche Untersuchungsmethoden für hölzerne Deckentragwerke, gereiht nach dem Grad der Zerstörungen des Eingriffes

3.2.1 Dynamische Methoden

Im Gegensatz zu den bei Laboruntersuchungen verwendeten dynamischen Methoden, wie Laufzeitmessung, Schwingungszeitmessung und Bestimmung von Dämpfungseigenschaften an unbelasteten, ideal gelagerten Prüfkörpern zur Bestimmung der mechanischen Werkstoffeigenschaften mit relativ hoher Präzision, werden die im folgenden behandelten dynamischen Untersuchungsmethoden in situ zur raschen Beurteilung des eingebauten und belasteten Bauteiles eingesetzt.

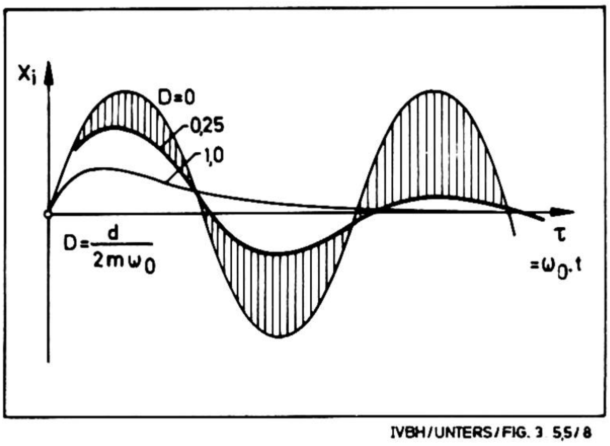


Fig.3 Impulsübergangsfunktion (Dmax=0.25 für gesunde Decken)

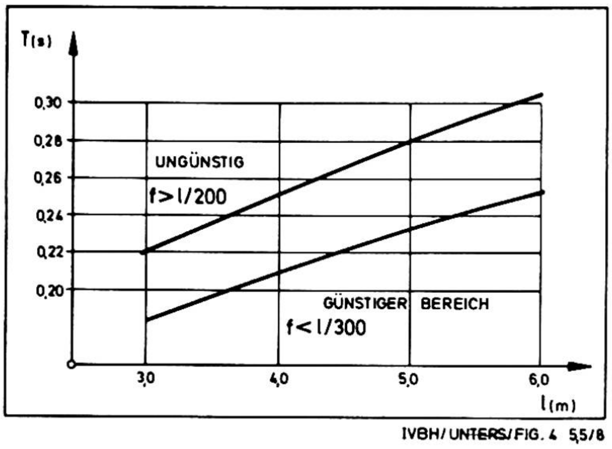


Fig.4 Zusammenhang zwischen Schwingungsdauer und Deckendurchbiegung

Die beiden zur Beurteilung der Decken herangezogenen Kennwerte sind einerseits das Dämpfungsmaß (Fig.3) und andererseits die Schwingungsdauer, die bei der gleichen Messung ermittelt werden können. Die Dämpfungszahl erwies sich dabei als wertvoller Parameter zur Beurteilung des Erhaltungszustandes der Träme, während die Schwingungsdauer einen direkten Rückschluß auf die Durchbiegung des untersuchten Deckenfeldes erlaubt.

Versuche zur genaueren Eingrenzung der Vergleichswerte werden zur Zeit noch durchgeführt, die im Moment vorliegenden Richtgrößen geben im Voruntersuchungsstadium bereits gute Anhaltspunkte für die generelle Zustandsbeurteilung.

3.2.2 Endoskopische Untersuchung

Stellen sich aufgrund der optischen Begutachtung oder der dynamischen Untersuchung Zweifel über den guten Erhaltungszustand bzw. die ausreichende Tragfähigkeit einzelner Deckenbereiche ein, so stellt die Anwendung der ursprünglich für medizinische Untersuchungen entwickelten Endoskopie eine geeignete Maßnahme dar, mit minimalem Eingriff die kritischen Bereiche (meist die Auflager der Deckenträme) direkt zu begutachten. Dabei können auch Kernbohrungen der Deckenbalken durchgeführt werden, um die entnommenen Proben einer Laboruntersuchung zuzuführen.

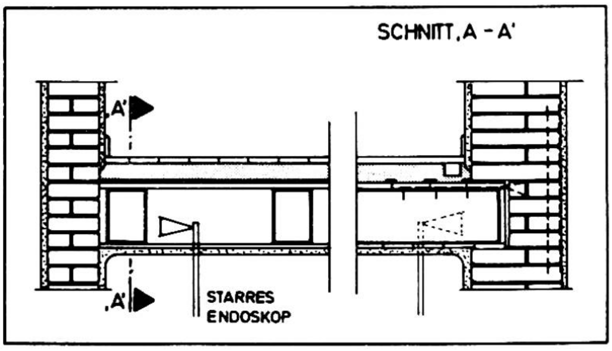


Fig.5 Begutachtung eines Deckenauflagers von der Unterseite der Decke aus. (Bei dieser Vorgangsweise werden Probleme mit dem Beschüttungsmaterial vermieden.)

Diese Methode stellt hohe Anforderungen an den Untersuchenden, da die sofort durchzuführende Beurteilung ein großes Maß an Erfahrung voraussetzt. Da im Rahmen einer endoskopischen Deckenuntersuchung praktisch jedes Deckenauflager erfaßt werden muß, ergeben sich relativ hohe Kosten, die die Notwendigkeit einer Eingrenzung "kritischer Bereiche" im Rahmen von Voruntersuchungen verdeutlichen.

3.2.3 Deckenöffnung

Bei der Untersuchung von Dippelbaumdecken oder unzugänglichen Bereichen, die keine endoskopische Untersuchung erlauben, ergibt sich zur Begutachtung kritischer Bereiche nur die Möglichkeit der Deckenöffnung, um eindeutigen Aufschluß über den Zustand der Balkenlagen (speziell im gefährdeten Auflagerbereich) zu erhalten.

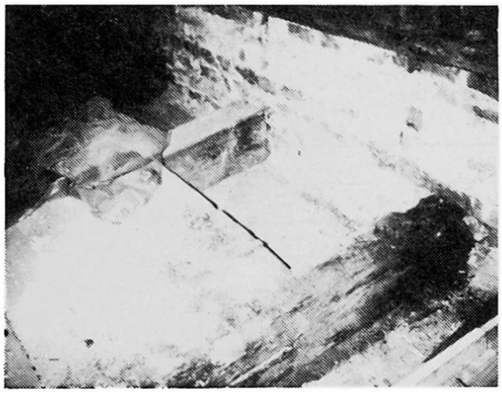


Fig.6 Deckenöffnung im Auflagerbereich einer Tramdecke (mit Fäulnisbefall)

3.2.4 Laborprüfung

Wird an einzelnen Deckenbalken im Rahmen der in situ-Untersuchungen eine Holzschädigung festgestellt, sind zur Identifikation der Schädlinge (Pilzbefall, Käferbefall) Laboruntersuchungen an entnommenen Holzproben durchzuführen, um die weitere Vorgangsweise festlegen zu können. Die dabei angewandten Methoden sind aus den verschiedensten Bereichen der Holzuntersuchung bekannt und werden daher nicht eingehend behandelt.

3.3 Ablaufschema für eine Deckenuntersuchung

Aufgrund der zuvor skizzierten Untersuchungsmethoden wurde ein einfaches Ablaufschema für die Untersuchung hölzerner Deckenkonstruktionen entwickelt, das eine rasche Entscheidung über den Deckenzustand bei möglichst geringen Eingriffen in die Konstruktionen (besonders wichtig bei bewohnten Objekten) und bei niedrigem Kostenaufwand erlaubt.

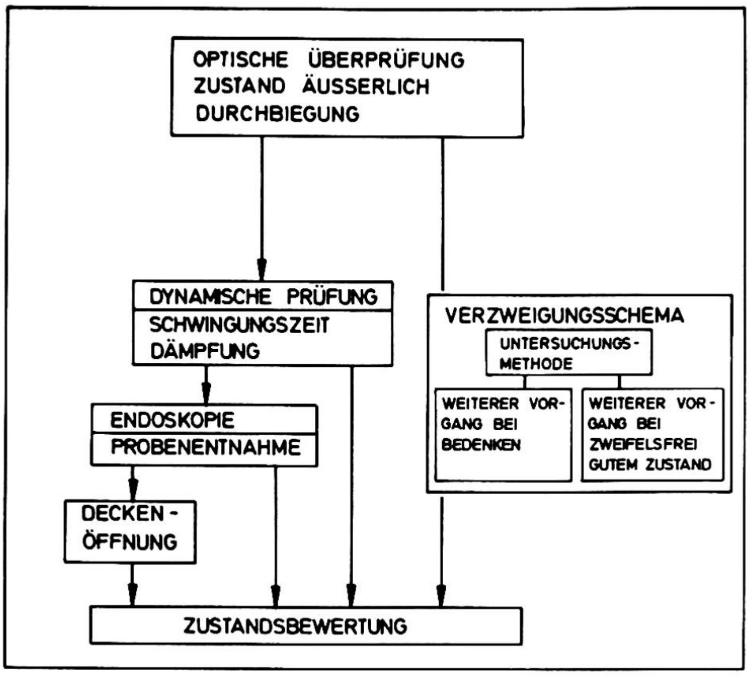


Fig.7 Ablaufschema für die Untersuchung von hölzernen Deckentragwerken

4. ERHEBUNGSBÖGEN FÜR DIE AUSWERTUNG DER UNTERSUCHUNGEN

Da die im Rahmen der Untersuchung der Deckenkonstruktionen gewonnenen Erkenntnisse möglichst rasch einer Auswertung als Grundlage für die Festlegung weiterer Maßnahmen zugeführt werden sollen, wurde ein standardisierter Erhebungsbogen entworfen, der einen raschen Überblick über die Untersuchungsergebnisse erlaubt. Dieser Übersicht werden die detaillierten Protokolle beigelegt.

GEBÄUDE:		
ERHEBUNG	AM:	DURCHGEFÜHRT VON:
GESCHOSS		WOHNUNGSEINHEIT (EN)

BAUTEIL / TYP	KURZZEICHEN	FLÄCHE (m²)	BEILAGEN: BESTANDSPLAN	MÜNDL. AUSKUNFT	OPT. BEGUTACHTG.	DYN. SCHWINGUNG	ENDOSKOPIE	DECKENÖFFNUNG			BEURTEILUNG: Ⓐ ABBRUCH U. NEUBAU %	Ⓐ m²	Ⓢ SANIERUNG %	Ⓢ m²	Ⓔ IN ORDNUNG %	Ⓔ m²
DECKEN HOLZDECKE																
TRAM	T	75	1	/	2	3	5	/			0	0	75	50	25	25
DIPPEL	D	0														
TRAMTR.	TT	40	1	/	2	3	6	/			0	0	100	50	0	0

SKIZZE:

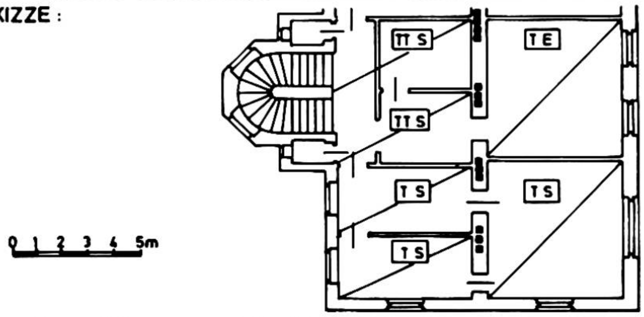


Fig.8 Erhebungsbogen zur Untersuchung von hölzernen Deckenkonstruktionen

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Stabilization of the Leaning Tower of Pisa

Stabilisation de la tour penchée de Pise

Die Stabilisierung des schiefes Turmes von Pisa

Raffaello BARTELLETTI
Prof. of Struct. Eng.
University of Pisa
Pisa, Italy

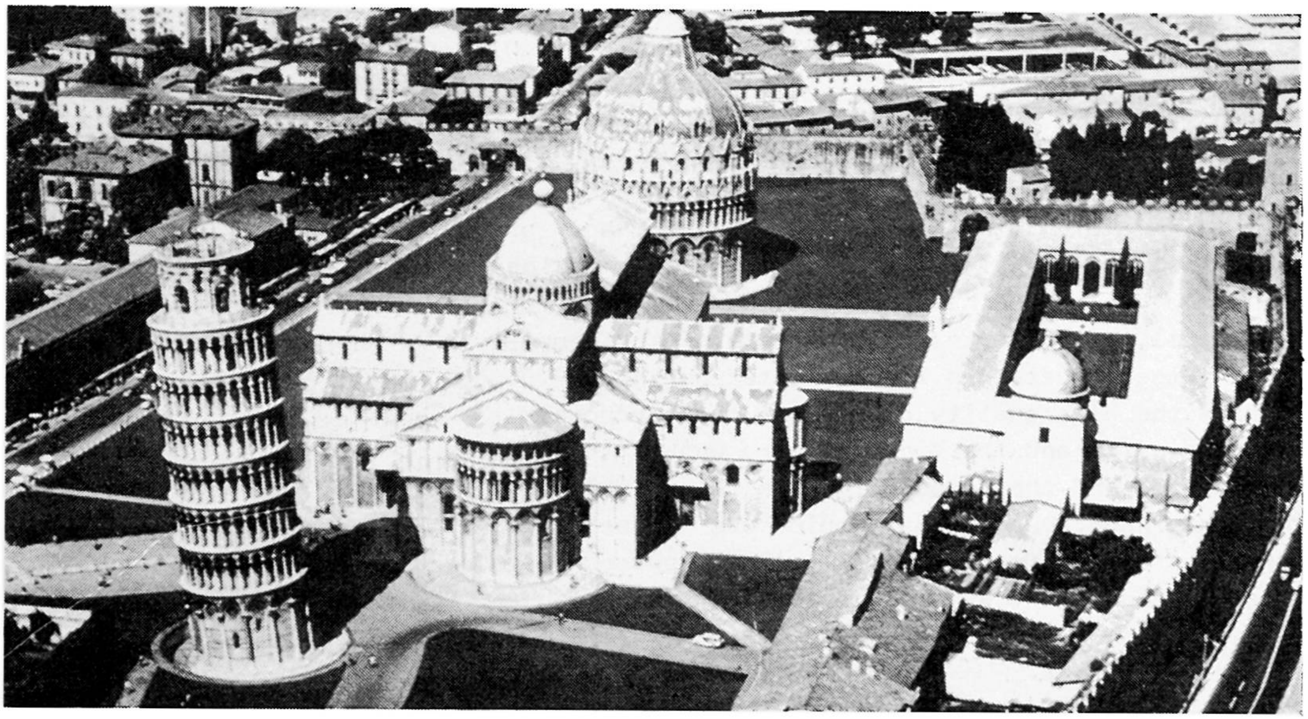
Giorgio BERARDI
Prof. of Geotechn. Eng.
University of Genova
Genova, Italy

Luciano CAROTI
Prof. of Civil Eng.
University of Pisa,
Pisa, Italy

Leo FINZI
Prof. of Theory of Struckt.
Politecnico of Milan,
Milan, Italy

Michele JAMIOLKOWSKI
Prof. of Geotechn. Eng.
Politecnico of Torino,
Torino, Italy

Luca SANPAOLESI
Prof. of Struct. Eng.
University of Pisa,
Pisa, Italy



SUMMARY
After a brief historical outline and a concise description of the structure and soil foundation of the Leaning Tower of Pisa, conceptual trends for proposed stabilization design are explained.

RÉSUMÉ
Après un aperçu historique et une description de la structure et du sol de fondation de la Tour Penchée de Pise, on expose les principes du projet proposé pour la consolidation du monument.

ZUSAMMENFASSUNG
Nach einem Ausblick auf die Geschichte des schiefen Turmes von Pisa und einer kurzen Beschreibung des Bauwerks und der Bodeneigenschaften werden die Prinzipen des Projektes zur Stabilisierung des Turmes dargelegt.

1. HISTORICAL OUTLINE

The construction of the Tower of Pisa started in 1173. According to the tradition, its designer was the architect Bonanno Pisano, but more recent studies indicate its designer was Diotisalvi (Ragghianti, 1985). The works were interrupted in 1178, probably because the Tower was already leaning when the construction was at the fourth floor and its net weight approached 95 MN. Construction was resumed in 1272 by Giovanni di Simone, and after six years was completed up to the bell cell. At that time the net weight applied to soil reached the 138 MN and the leaning of the Tower became evident due to a differenzial settlement of about 90 cm. The construction of the bell cell started in 1360, by Tommaso d'Andrea and was completed in 1370.

The present situation of the Pisa Tower is summarized in Fig. 1.

The systematic measurement of the Tower movement started in 1911: at that time the rigid tilt approached 5° 14' 46" . During the 1900's the Tower continued to set and to tilt occurring in a North to South direction. The movement of the Tower is clearly some kind of self-driving instability phenomenon, causing the Italian Ministry of Public Works to establish, in the sixties, an International Commission which aim was to study the structural and soil mechanics aspects of the problem. In 1971 the Commission completed its work leaving in the hand of the Ministry a technical documentation suggesting the promotion of an international competition for the stabilization works. The analysis of the proposals by the competitors was completed during 1975. Despite the high quality of some of the proposals, the Commission and the Ministry were unable to take a decision, and the competition was then closed without a winner. During that period was stopped the pumping of water from deep wells in the surrounding areas, because they were found to be responsible for a general subsidence leading to an acceleration of the Tower tilt.

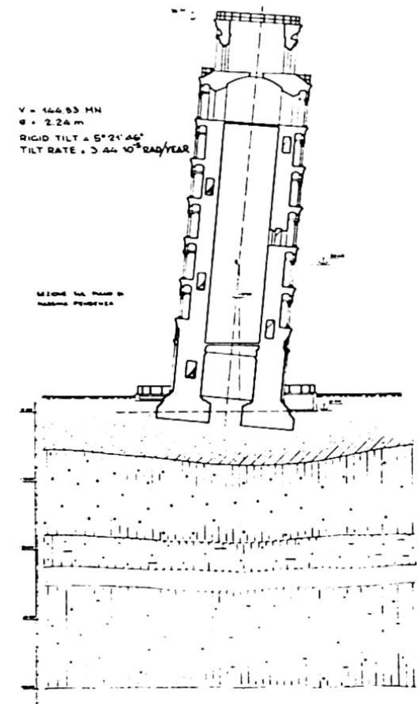


Fig. 1 Pisa Tower present situation

In 1984 the Ministry nominated the Authors as a design team whose duty was to establish a design for the stabilization work. They have supplemented the large amount of data already gathered by the Ministry by means of preliminary extensive experimental investigations both on the structure and on the foundation.

2. STRUCTURE OF THE TOWER

The cylindric wall is made of two square blocks, shells filled with mixed masonry of cobbles and very good hydraulic lime mortar. The foundation, made of stone masonry and hydraulic lime mortar has been extensively consolidated in 1935 by Rodio using cement grouting. The masonry max. stress, inferred by a theoretical analysis and supported by sophisticated measurements on the structure (doorstopper, flat jacks, dilatometer) are of about 7 N/mm^2 on the outer freestone with an ascertained resistance of 80 to 100 N/mm^2 and Young's modulus (E') of about 70.000

N/mm^2. In the inner masonry which has an E' about 6–8 times lower we can presume values around 1 N/mm^2 vs an ascertained resistance between 6 and 10 N/mm^2.

3. SOIL FOUNDATION

The soil profile and the geotechnical characterization of the soil foundation are based on the above set of investigation.

Fig. 2 shows the soil profile and the stress history of the deposit. The lower clay extends to an elevation of -37 m below m.s.l., where a thick bearing layer of very dense sand is encountered. The results of oedometer tests performed on good quality undisturbed samples taken far from the Tower (representing free field

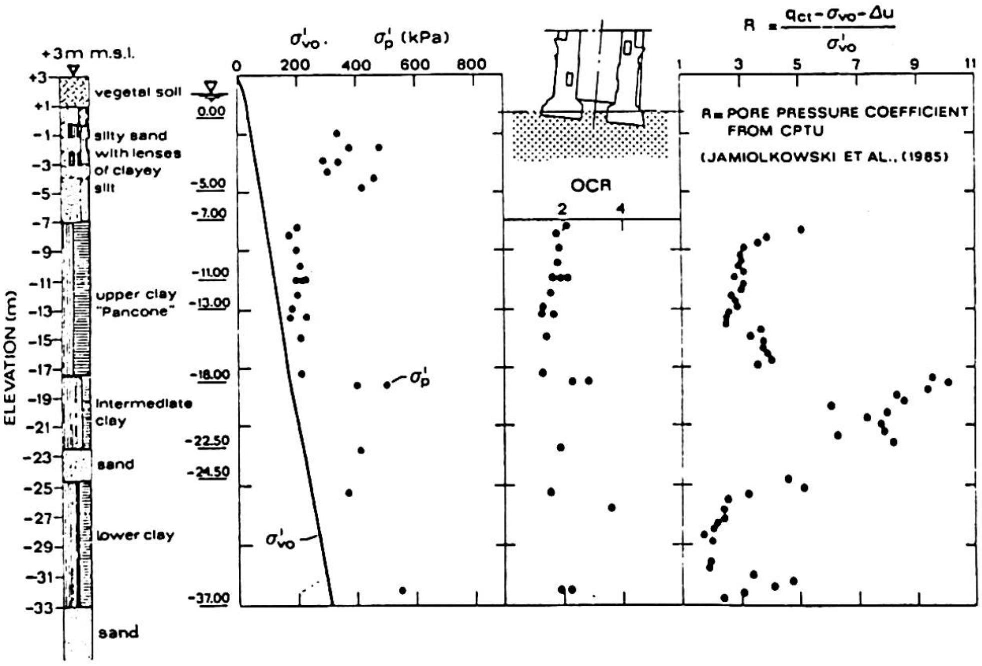


Fig. 2 Profile of soil underlaying Pisa Tower

values, hence not influenced by the stress induced by the Tower) indicated consistently that $\sigma'_p > \sigma'_{vo}$, leading to the OCR profile shown in the Figure. The preconsolidation mechanisms responsible for this are not known; probably they are linked with aging, ground water level oscillations and other environmental changes.

Fig. 3 presents the effective stress shear strength envelope of undisturbed Pisa clay, as obtained from drained compression tests performed on both isotropically and anisotropically consolidated specimens. The plot presents the mean effective stress (s') on the horizontal axis and one half of the deviatoric stress (t) on the vertical axis, both normalized with respect to the σ'_p.

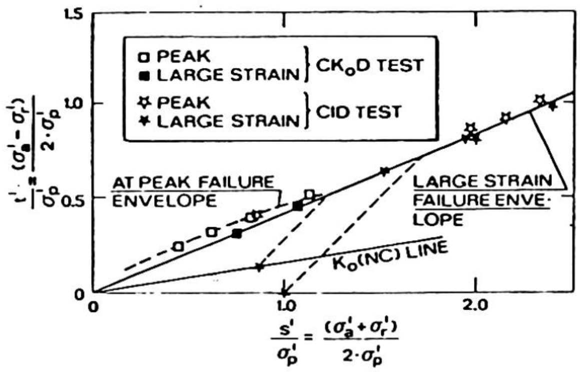


Fig. 3 Strength envelope of Pisa clay from drained tests

Fig. 4 takes us back to the Cambridge p' - q plane, in which the results of undrained triaxial compression and extension tests, performed on specimens reconsolidated to the in-situ initial stresses (σ'_{vo}, σ'_{ho}) in K_o conditions are summarized. Also in this case both p' and q have been normalized to in-situ σ'_p.

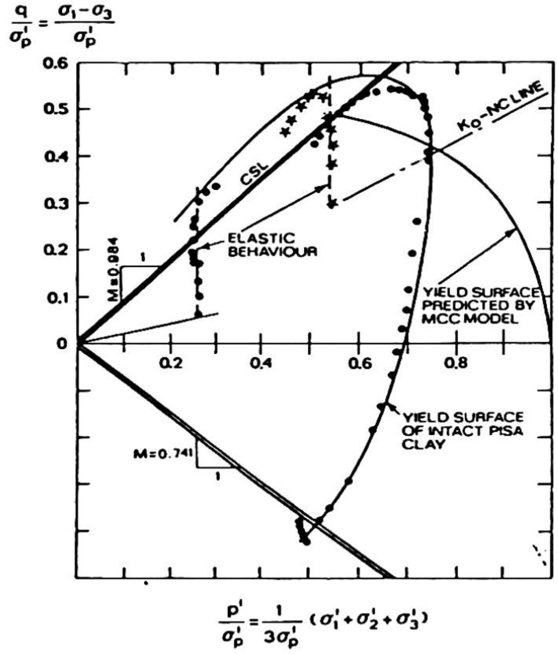


Fig. 4 Yield locus of Pisa Clay

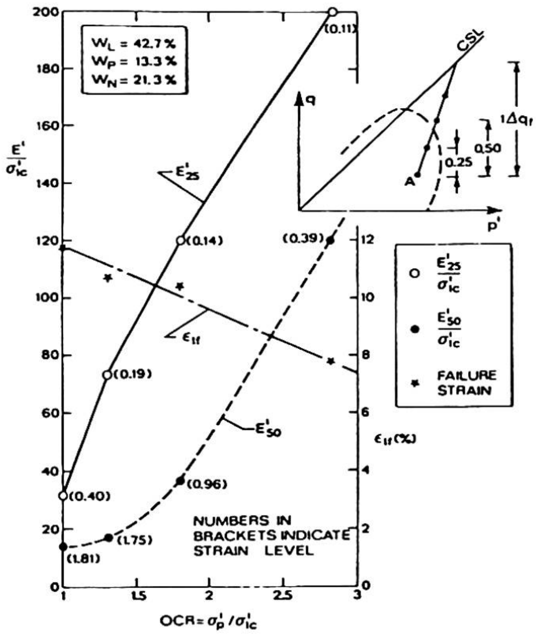


Fig. 5 Stifness of natural Pisa clay from TX-CK_oD compression tests

Fig. 5 shows the variation of normalized drained Young's modulus (E') of Pisa Clay versus OCR. The E'_{25} and E'_{50} correspond to the secant moduli evaluated at one fourth and one half the increment of the deviator stress at failure respectively.

4. CONCEPTUAL TRENDS IN STABILIZATION OF PISA TOWER

The conceptual trends of the design proposed by the Authors in 1985 with the aim to stop the movements of the Tower are shown in Fig. 6.

The basic idea of the geotechnical solution proposed by the design team in 1985 consists of the application of a stabilizing moment to the Tower and is linked qualitatively to the Critical State Soil Mechanics (CSSM) concept as visualized in Fig. 7. The effect of the application of the stabilizing moment will be to change the existing stress distribution below the Tower's foundation; unloading the soil under the Southern part and reloading the soil under the Northern part. This, at least in principle, will lead to a situation as shown in Fig. 7, in wich all relevant soil elements will be subjected to the stress state falling inside the stress space delineated by the current yield locus. In order to quantify

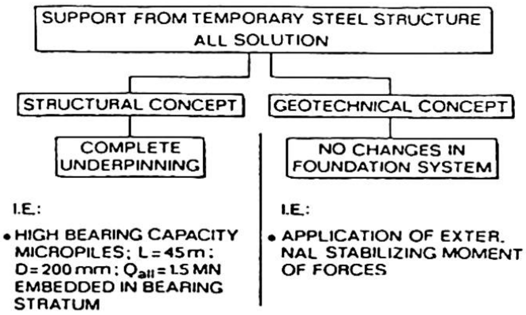


Fig. 6 Stabilization of Pisa Tower: conceptual trends

this concept, the application of the stabilizing moment to the Tower has been modelled by FEM analyses incorporating an elasto-plastic model belonging to the family of modified Cam Clay Models (CCM's).

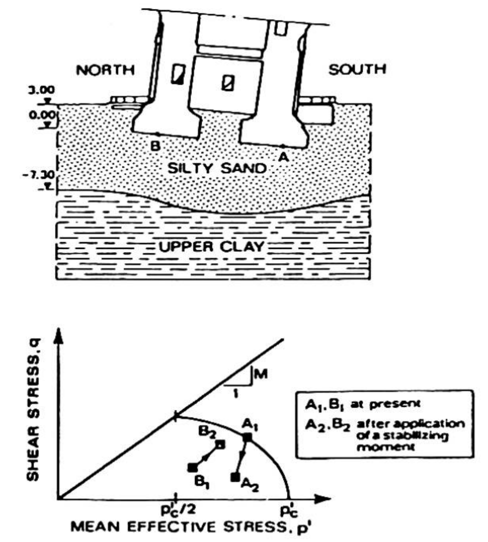


Fig. 7 Stabilization of Pisa Tower: geotechnical solution

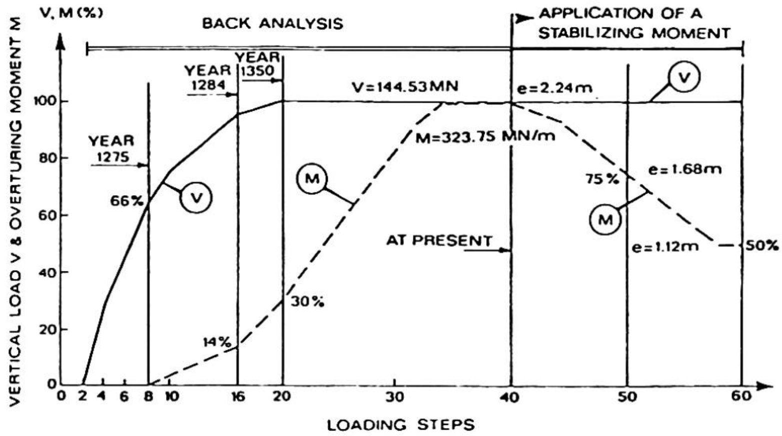


Fig. 8 Loading history of Pisa Tower

Fig. 8 shows the numerical simulation of the loading history. The loading step 20 corresponds to the end of construction. From this point, in the time the vertical load remains constant and the overturning moment increases as a consequence of the increase of the load eccentricity (e) caused by the Tower tilt in time. The loading step 40 coincides with the beginning of the application of the stabilizing moment of force, wich assumes that the final moment reduces by one half the existing value. According to the theory, the application of the stabilizing moment to the Tower would reduce its tilt within the limits allowed by the Ministry.

5. PROPOSED DESIGN

The proposed final solution for the stabilization of the Tower of Pisa consists in two distinct steps. The first providing the application of a stabilizing moment of force to the Monument, and the second one consisting in the complete underpinning of the existing foundation. The final acceptance of the first solution or the move to the second one is subordinated to the observed behaviour of the Tower after the application of the stabilizing moment of force.

A temporary steel structure will be the preliminary step. It will be placed on a prestressed concrete annular beam located far enough from the Tower and founded on high bearing capacity micropiles. The steel structure has been designed in order to: a) assure stability of the Tower during the works; b) apply the stabilizing moment to the Tower up to 320 MNm; c) support a vertical load of 150 MN. The above load on the masonry structure imply stresses compatible with the masonry resistance, previously consolidated by grouting. The schematic cross-section of the temporary steel structure is shown in Fig. 9. The structure will be used to apply the temporary stabilizing moment to the Tower (160 MNm). This will be followed by an adequate period of monitoring and possible adjustment of the moment according to the principle of the observational method. During this period of time, if the observed behaviour is satisfactory, proving the effectiveness of the soil model and of input data utilized, then a permanent structure embedded in the ground will be built, as shown in Fig. 10, and subsequently the temporary steel structure dismantelled. The permanent structure will incorporate a number of hydraulic jacks to permit adjustment to the stabilizing moment with time if necessary. By means of this solution, the Tower would still be laying on the foundation ground with a 8 centuries applied load but with a reduced eccentricity and therefore with a state of stress much more favourable than the present one.

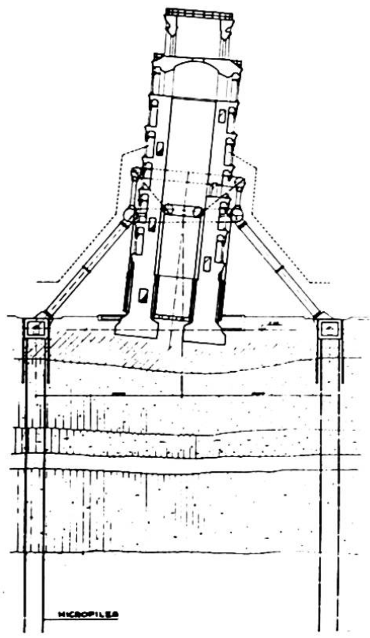


Fig. 9 Temporary support

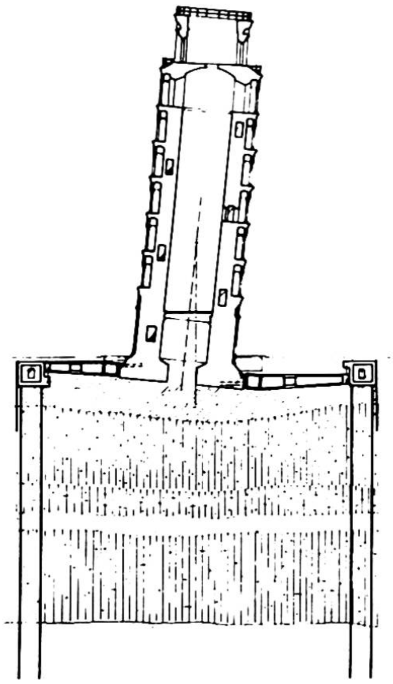


Fig. 10 Permanent structure for stabilizing moment

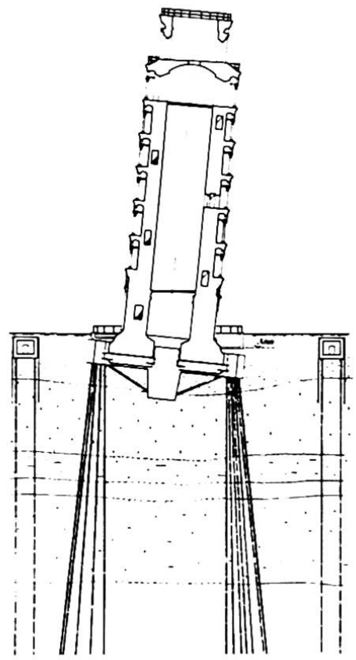


Fig. 11 Structure for complete underpinning

A complete underpinning has been exhaustively designed and will be applied shouldn't the application of the corrective moment give the predicted effects. Such an alternative method consists in the construction of a group of micropiles 45 m long, D 200 mm which penetrate the dense sand layer and with a bearing capacity of 1,5 MN, all located outside the foundation block (Fig. 11). Steel radial connected beams will transfer the load from the present foundation to the micropiles group. The beams are supported externally by the prestressed reinforced concrete ring overlaying the piles, while internally they lay on a concrete block sustained by 8 prestressed anchors. During construction before the operating of such anchors, the provisional structure will sustain the internal ends of the radial beams.

The two methods have been studied in order to maintain the historical-artistic integrity of the Monument, since after the completion of the construction no piece of work will be visible nor any super-structure of the Tower will have experienced any kind of modifications. Both designs have had the outstanding approval by late Prof. C. Ludovico Ragghianti, a great historical art expert and member of the Commission established by the Ministry of Public Works.

Untersuchungen zum Rißfortschritt an einer Stahlbrücke

Investigations of crack growth on a steel bridge

Etude de la fissuration d'un pont métallique

Klaus BRANDES
Dr.-Ing.
BAM
Berlin

Klaus Brandes, geb. 1936 promovierte 1968 an der Technischen Universität Berlin. Nach vier Jahren Tätigkeit an der TU Berlin und Mitarbeit in Ingenieurbüros ist er seit 1968 in der Bundesanstalt für Materialforschung und -prüfung (BAM) in unterschiedlichen Gebieten tätig.

ZUSAMMENFASSUNG
Risse in Fahrbahnträgern einer älteren genieteten Straßenbrücke waren Anlaß für experimentelle Untersuchungen. Diese umfaßten sowohl Dehnungs- und Verformungsmessungen an der Brücke als auch Materialuntersuchungen, wobei Rißfortschrittsversuche einbezogen wurden, um die Gefährdung der Brücke realistisch beurteilen zu können.

SUMMARY
Cracks in the structural members of a riveted road-bridge gave rise to experimental investigations. They included measurement of strains and deformations of the bridge as well as tests on the material to evaluate the characteristic technological properties. In addition, we performed crack growth tests in order to come to an appropriate and realistic estimation of the damage risk of the bridge.

RÉSUMÉ
Une étude expérimentale a été conduite sur les fissures d'éléments structuraux d'un pont route rivé. Elle comprend la mesure des tensions et déformations du pont de même que des essais de matériaux afin de déterminer les propriétés technologiques caractéristiques. Des essais de fissuration ont été conduits afin d'estimer de façon réaliste le danger de ruine du pont.

1. EINFÜHRUNG

Nicht nur Stahlbeton- oder Spannbetonbrücken haben Risse, sondern auch Stahlbrücken, die allerdings nicht immer eine Gefährdung der Brücke mit sich bringen müssen [1] [2].
Im Rahmen der Ermittlung der Restnutzungsdauer alter Stahlbrücken geht man in aller Regel davon aus, daß noch keine sichtbaren Risse vorhanden sind und man bei der Beurteilung auf die bekannten Wöhler-Diagramme [3] zurückgreifen kann.
Werden allerdings größere Risse entdeckt, so sind zwei Fragen zu beantworten, um das von ihnen ausgehende Risiko beurteilen zu können. Zum einen muß der Spannungszustand in der Umgebung des Risses zutreffend bestimmt werden, zum anderen ist zu erkunden, wie schnell ein Riß bei bekanntem Spannungszustand in dem Material wächst. Fast immer sind rechnerische Analysen allein zur Lösung der Aufgabe nicht ausreichend, vielmehr sollten sie von experimentellen Untersuchungen begleitet werden.

Für eine Brücke, an deren Querträgern längere Risse aufgetreten waren, ist eine derartige Untersuchung durchgeführt worden, über die im folgenden berichtet wird.

2. PROBLEMSTELLUNG

An einer genieteten Fachwerk-Straßenbrücke (Fig. 1) waren bei den routinemäßigen Untersuchungen an Querträgern Risse entdeckt worden, die in der Nähe des Anschlusses an die Hauptträger ent-

Fig. 1 Untersuchte genietete Fachwerk-Straßenbrücke

standen waren. Sie hatten ihren Ausgang von konstruktiven Kerben aus genommen, die insbesondere wegen des Anschneidens von Seigerungszonen in den als Querträger verwendeten Walzträgern aus Thomas-Stahl gefärlich waren, siehe Fig. 2.

Die Brücke war zu Beginn der fünfziger Jahre gebaut worden, das Material stammte aus den dreißiger Jahren und war z. T. aus anderen, zerstörten Bauten entnommen worden. Die Werkstoffeigenschaften waren nicht bekannt.

Bis zu einer Klärung der Frage, aus welchen Gründen die Risse an diesen Stellen aufgetreten waren, mußten Beschränkungen für den Verkehr über die Brücke angeordnet werden, um jede Gefährdung zu vermeiden.

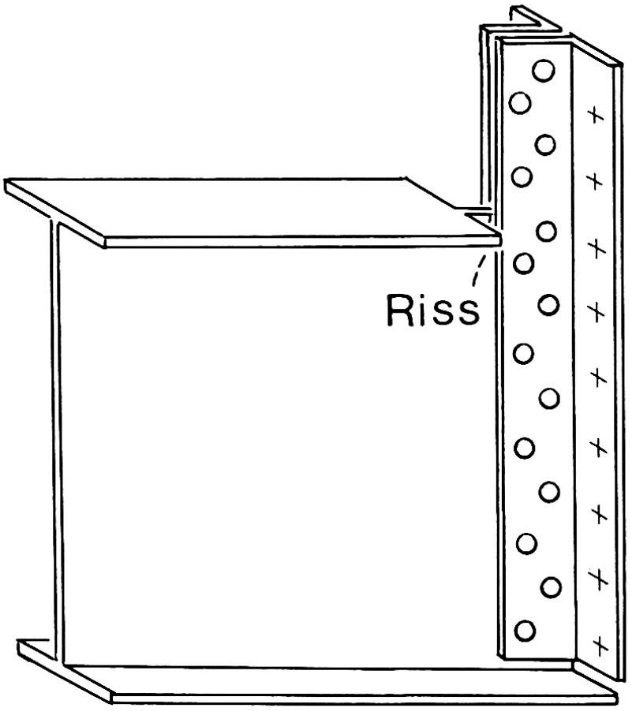


Fig. 2
Konstruktive Details im Bereich der Riß-ausgangsstelle

3. UNTERSUCHUNGEN

Zur Klärung der aufgeworfenen Fragen wurden folgende Untersuchungen vorgesehen:

- Materialuntersuchungen zur Bestimmung der Materialeigenschaften von Quer- und Längsträgern. (Über das Material der Hauptträger lagen ausreichende Informationen vor.)
- Rißfortschrittsuntersuchungen an Proben aus demjenigen Querträger, der die niedrigsten Kennwerte bezüglich der technologischen mechanischen Kennwerte aufwies, sowie an Proben aus einem Längsträger.
- Dehnungsmessungen an den Fahrbahnträgern unter definierter statischer Belastung und unter laufendem Verkehr.
- Rechnerische Verfolgung des Lastabtrags von den Fahrbahnträgern an die Hauptträger unter Zugrundelegung eines Trägerrostmodells.

An Stelle der traditionellen Dauerschwingfestigkeitsuntersuchungen wurden Rißfortschrittsuntersuchungen gewählt, da bereits Risse vorhanden waren und das Material eventuell zu schnellem Rißwachstum hätte neigen können.
Mit den Dehnungsmessungen konnte einerseits das der Rechnung zugrunde gelegte statische Modell überprüft werden (statische Belastung), während andererseits aus den Messungen unter laufendem Verkehr die Schwingungsempfindlichkeit der gesamten Konstruktion festgestellt werden konnte. Daneben ergaben sich Daten zur Überprüfung der Annahmen über die Beanspruchungskollektive.

Bei der Übertragung von Rißfortschrittsuntersuchungen an standardisierten Proben (Compact Tension Specimen) nach der amerikanischen Regel ASTM E 647-83 [4] (Fig. 3) auf die Verhältnisse im realen Bauteil treten in aller Regel erhebliche Schwierigkeiten auf; denn als Kennwert der Beanspruchung wird der Spannungsintensitätsfaktor K (Dimension: $N/mm^{3/2}$) an der Rißspitze verwendet, eine Größe, die nur bei einfachsten geometrischen Verhältnissen einigermaßen zutreffend bestimmt werden kann. Seine Ermittlung im Bauteil ist meist nur näherungsweise und oft nur in weiten Grenzen möglich.

Die für eine endgültige Beurteilung wesentliche Frage, wie von den Beanspruchungskollektiven der Brücke auf Versuche mit konstanter Spannungsamplitude umgerechnet werden kann, sei hier nur zur Vervollständigung erwähnt.

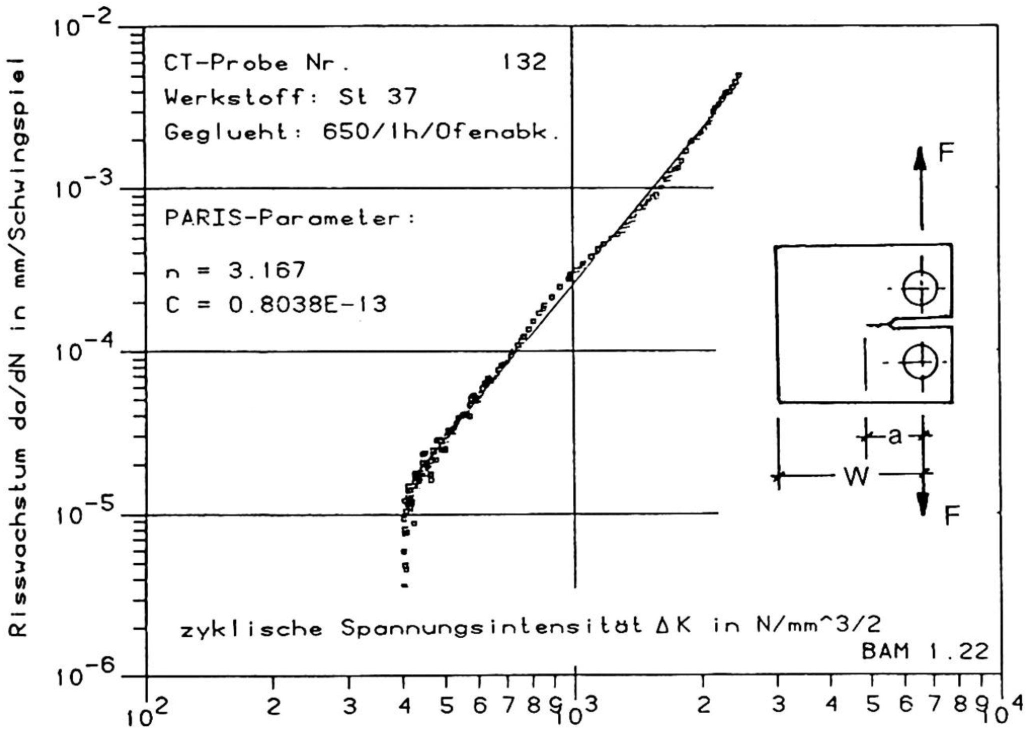


Fig. 3
Darstellung der Rißwachstumsrate über der Zyklenzahl

4. ERGEBNISSE

Aus den Materialuntersuchungen ergab sich, daß sowohl Querträger wie Längsträger aus Thomas-Stahl bestanden, der stark gealtert war. Die Fließspannung und Zugfestigkeit entsprachen im wesentlichen denen eines Baustahls St 37. Die chemische Zusammensetzung zeigt Werte, die nicht ganz den heutigen Anforderungen (DIN 17 100) genügen. Die Werte für die Kerbschlagarbeit zeigten bei 0 °C geringe Werte, was für Thomas-Stahl erfahrungsgemäß fast immer auftritt. Wie makroskopische Untersuchungen zeigten, ist das Gefüge über die Materialdicke sehr unterschiedlich (Seigerungszonen mit hohem Schwefelgehalt im Innern).

Aus den Rißfortschrittsuntersuchungen ergab sich eine Wachstumsgeschwindigkeit, die den aus der Literatur bekannten Variationsbereich etwas überschreitet. Der in Fig. 3 erkennbare lineare Zusammenhang (Paris-Gleichung [5]) wird in der Form

$$da/dN = C \Delta K^n$$

angegeben, wobei der Exponent n für Baustahl etwa n = 3 ist. Für das untersuchte Material wurden Werte zwischen

2,83 und 4,55 (Querträger) und
3,88 und 4,75 (Längsträger)

ermittelt, die sich bei wärmebehandelten Proben etwas nach unten verschoben.

Als ganz wesentliche Untersuchungen stellten sich die Dehnungsmessungen an den Quer- und Längsträgern heraus. Zum einen erwies sich das der Berechnung zugrunde gelegte idealisierte Modell für statische Lasten als zutreffend. (Die Berechnungen wurden vom Ingenieurbüro HRA, Bochum, durchgeführt.) Zum anderen ergab sich, daß bei fließendem Verkehr die Brücke durch Unebenheiten in der Fahrbahn zu Schwingungen angeregt wurde, die sich vor allem an den Stellen, an denen die Rise aufgetreten waren, bemerkbar machten. Die Fahrbahn, etwa in Höhe des Untergurtes der Fachwerkhauptträger angeordnet, wurde über die Querträger zum Mittragen im Haupttragsystem gezwungen. Die dabei in den Querträgern in der Ebene von deren Obergurten entstehenden Beanspruchungen sind rechnerisch kaum zu erfassen, erreichen aber, wie die Dehnungsmessungen zeigten, erhebliche Werte. Aus Fig. 5 ist zu erkennen, wie die Dehnungen an zwei sich gegenüberliegenden Stellen am Querträgersteg (1. Fig. 2) beim Überfahren eines schweren Lastkraftwagens verlaufen. Im wesentlichen werden Querbiegungen des Querträgersteges hervorgerufen.
Die Risse sind auf diese, nicht planmäßige Beanspruchung zurückzuführen, die je Überfahrt eines schweren Fahrzeuges eine Reihe von Beanspruchungszyklen erzeugt (s. Fig. 4).

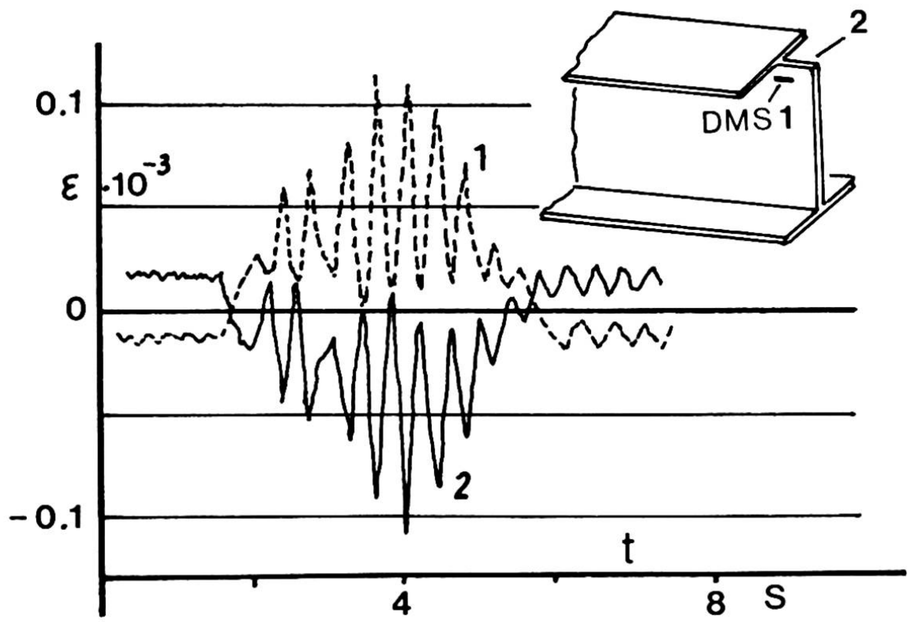


Fig. 4 Zeitlicher Verlauf der Dehnungen an zwei einander gegenüberliegenden Punkten am Steg des Querträgers im Bereich des Anschlusses an den Hauptträger

5. FOLGERUNGEN

Nach Abschluß aller Untersuchungen konnten zwei Feststellungen getroffen werden:

- Die beobachteten Risse an den Querträgern sind auf nicht planmäßige Beanspruchungen zurückzuführen. Erreichen diese Risse eine bestimmte Länge, dann wird das Zusammenwirken von Fahrbahn und Fachwerk-Untergurt zunehmend abgebaut, der Riß wächst kaum noch weiter.
- Für andere hochbeanspruchte Stellen ergaben die Untersuchungen, daß die Spannungen meist unterhalb der rechnerisch ermittelten liegen, die Anzahl der Beanspruchungszyklen mit großer Spannungsdifferenz niedriger liegt als zunächst errechnet. Da das Material nicht zu einem erxtrem schnellen Rißwachstum neigt, kann eine Gefährdung der Brücke ausgeschlossen werden, ein Ergebnis der Rißfortschrittsuntersuchungen.

Auf Grund der Untersuchungsergebnisse kann auf eine Sanierung von Teilen der Brücke verzichtet werden. An den Rißstellen werden an den Rißspitzen Bohrlöcher gesetzt, um ein weiteres Fortschreiten zu unterbinden. Auch an den übrigen Querträgern werden diese Bohrlöcher vorgesehen, da auch dort mit dem Entstehen von Rissen zu rechnen ist.

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Dynamic Response and Subsequent Retrofit of a Tied-Arch Bridge

Dynamisches Verhalten und Verstärkung einer Bogenhängebrücke

Comportement dynamique et réhabilitation d'un pont arc à tirants

John M. KULICKI
Partner
Modjeski and Masters
Harrisburg, PA, USA

John M. Kulicki received his bachelor's degree from Lafayette College, Easton, PA and his masters and doctorate from Lehigh University, Bethlehem, PA. He joined Modjeski and Masters in 1974 and was named a Partner in 1980. Dr. Kulicki is the Partner-in-Charge and Principal Investigator on the I-24 Tied Arches Study.

Dennis R. MERTZ
Engineer
Modjeski and Masters
Harrisburg, PA, USA

Dennis R. Mertz was awarded his doctorate from Lehigh University, Bethlehem, PA in 1984. While at Lehigh, he was involved in the study of fatigue and fracture of steel bridges. Dr. Mertz is currently Project Manager of Modjeski and Masters' study of the I-24 Tied Arches.

Richard E. MURPHY
Engineer
Modjeski and Masters
Harrisburg, PA, USA

Richard E. Murphy received his engineering degree from Drexel University, Philadelphia, PA. In six years with Modjeski and Masters, he has been involved in design, development of in-house software and field inspections. Mr. Murphy's responsibilities on the current I-24 project include field investigation, finite element modelling and analysis.

SUMMARY
Two tied-arch spans across the Ohio River are experiencing excessive vertical dynamic displacement under normal traffic. The distortion, in addition to being intolerable due to public reaction, has resulted in fatigue cracking of the welded-floorbeam webs. The problem was studied through a combination of field studies and finite element modelling. These investigations suggest that the distortion problem can be solved through the addition of a longitudinal stiffening truss near the roadway level, and bolted connections spanning existing web gaps.

RÉSUMÉ
Deux ponts arcs à tirants sur l'Ohio présentent des mouvements verticaux excessifs sous des charges normales de trafic. Cette situation anormale n'est pas acceptée par les utilisateurs et résulte aussi en l'apparition de fissures de fatigue dans les poutres soudées du tablier. Le problème a été étudié sur place et au moyen d'un modèle par éléments finis. Ces recherches montrent que le problème de distorsion peut être résolu par l'addition d'une poutre de rigidité longitudinale près du tablier et par des assemblages boulonnés entre les poutres existantes.

ZUSAMMENFASSUNG
Zwei über den Ohio führende Bogenhängebrücken weisen unter normalen Verkehrslasten übermässige vertikale dynamische Verformungen auf. Diese Verformungen haben schon zu öffentlichen Reaktionen und in den geschweissten Fahrbahnträgern zu Rissen geführt. Das Problem wurde mit Feldmessungen und Finite-Elemente-Modellen untersucht. Die Abklärungen führen zum Schluss, dass das Verformungsproblem mit einem versteifenden Längsträger nahe der Fahrbahn und mit geschraubten Verbindungen über die vorhandenen Oeffnungen im Flanschbereich gelöst werden kann.

1. BACKGROUND

Two tied-arch spans carrying I-24 across the Ohio River near its confluence with the Mississippi River are experiencing severe distortional problems. These spans are 192 m and 223 m in length and of the bowstring-type tied arch in which the tie girder contributes insignificant bending stiffness to the span. The longitudinal bending stiffness of the spans is concentrated in the very significant arch rib. The spans are connected together and to the riverbanks by continuous plate-girder spans forming a river crossing totalling over 1,700 m in length. A photograph of one of these spans is shown in Figure 1. The resultant problems are twofold. The global vertical displacement field of the tied-arch spans is excessive and, due to the public's loss of confidence in the bridge, intolerable. Secondly, out-of-plane distortions of the floorbeams, which are related to the global displacement, have resulted in ever-increasing fatigue cracking in the webs of the welded floorbeams at locations shown in Figure 2.

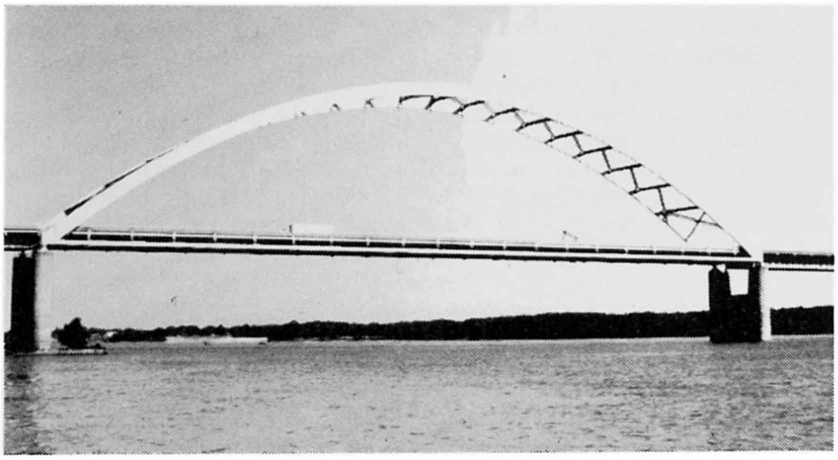


Figure 1 - View of 223 m tied-arch span.

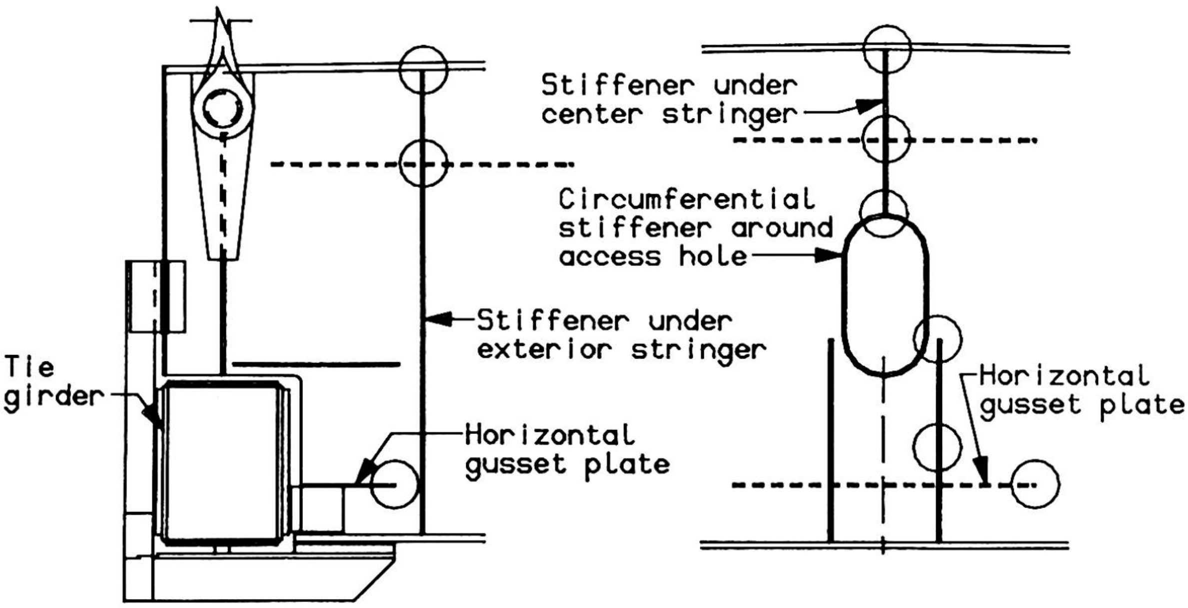


Figure 2 - Locations of typical fatigue cracks on welded floorbeams.

The distorted shape of the tied arches under vehicular traffic is dominated by the classic first bending mode for an arch, i.e., a 360 degree sine wave. The maximum vertical displacement at the quarter points of the longer span have been observed by the owner to be approximately +0.25 m and -0.25 m, resulting in a total excursion of one-half of a meter. The most significant distortions are produced by single trucks (or the chance occurrence of trucks traversing the tied-arch span side-by-side for the entire span), since trucks on other portions of the span tend to counteract the displacement and dampen the motion.

2. CORRELATIONS BETWEEN FIELD AND ANALYTIC STUDIES

Investigations began with field studies using accelerometers and strain gages. Analytic studies utilized extensive finite element dynamic modelling. The finite element models used to study the global dynamics were three-dimensional assemblages of plate bending, beam and truss elements as shown in Figure 3. A comparison of field-measured and calculated frequencies and mode shapes are shown in Figure 4. In this figure, the closed circles represent measured normalized amplitudes at the discrete accelerometer locations, while the continuous solid lines represent calculated mode shapes. Note that the mode numbers indicate that not all of the theoretical modes are significantly excited by the controlled truck traffic used in the field studies. During the course of the study of the tied-arch spans in their original configuration, it became evident that only the lower frequencies were significant. This corroborates the field observation of displacement-induced crack locations that suggests that the first bending mode is dominant. The comparison of field and analytic results indicates that the computer modelling accurately represents the physical situation. This step provided a confident base onto which various retrofits could be superimposed.

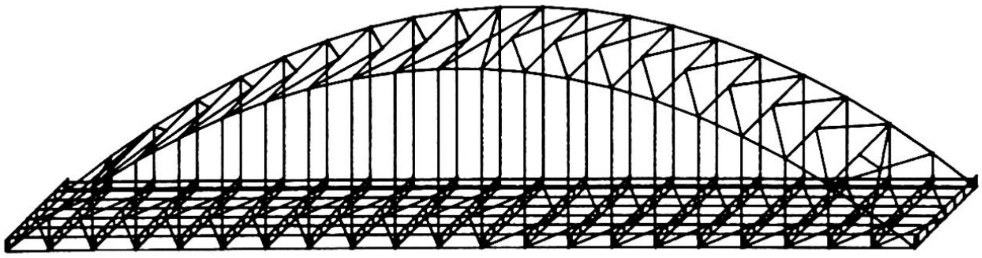
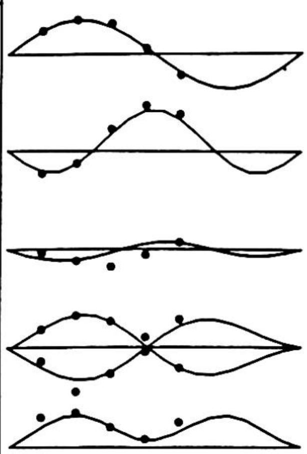


Figure 3 - Isometric view of three-dimensional finite element model.

192 m SPAN

CALCULATED		MEASURED	
MODE NO.	FREQUENCY	MODE NO.	FREQUENCY
1	0.317 HZ	1	0.316 HZ
2	0.654 HZ	2	0.679 HZ
3	0.684 HZ	3	0.953 HZ
4	0.894 HZ	4	1.050 HZ
8	1.276 HZ	5	1.165 HZ



223 m SPAN

CALCULATED		MEASURED	
MODE NO.	FREQUENCY	MODE NO.	FREQUENCY
1	0.271 HZ	1	0.273 HZ
2	0.558 HZ	2	0.566 HZ
6	1.012 HZ	3	1.023 HZ
8	1.165 HZ	4	1.250 HZ
9	1.488 HZ	5	1.550 HZ

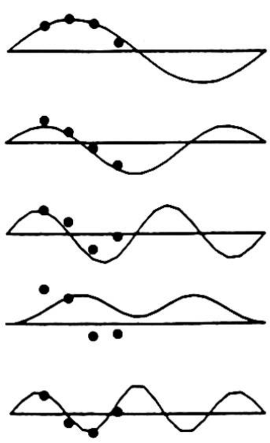


Figure 4 - Comparison of field-measured vs. calculated frequencies and mode shapes.

3. PARAMETRIC STUDIES OF POTENTIAL GLOBAL RETROFITS

3.1 Structural Retrofits

The effects of varying certain bridge stiffness parameters were considered in an attempt to determine an appropriate corrective course of action. The parameters singled out as potential sources of decreasing the magnitudes of the displacement field of the tied arches were the addition of several inclined hangers; and increases in tie girder axial stiffness, tie girder flexural stiffness, hanger axial stiffness, hanger flexural stiffness and arch rib flexural stiffness.

The dynamic properties of the original tied-arch spans and their potential retrofitted versions were studied through more efficient two-dimensional finite element modelling. Fundamental frequencies and their accompanying mode shapes were determined through modal analysis utilizing the SAPIV finite element library. The effects of increasing the original span parameters, indicated above, are illustrated in Figure 5.

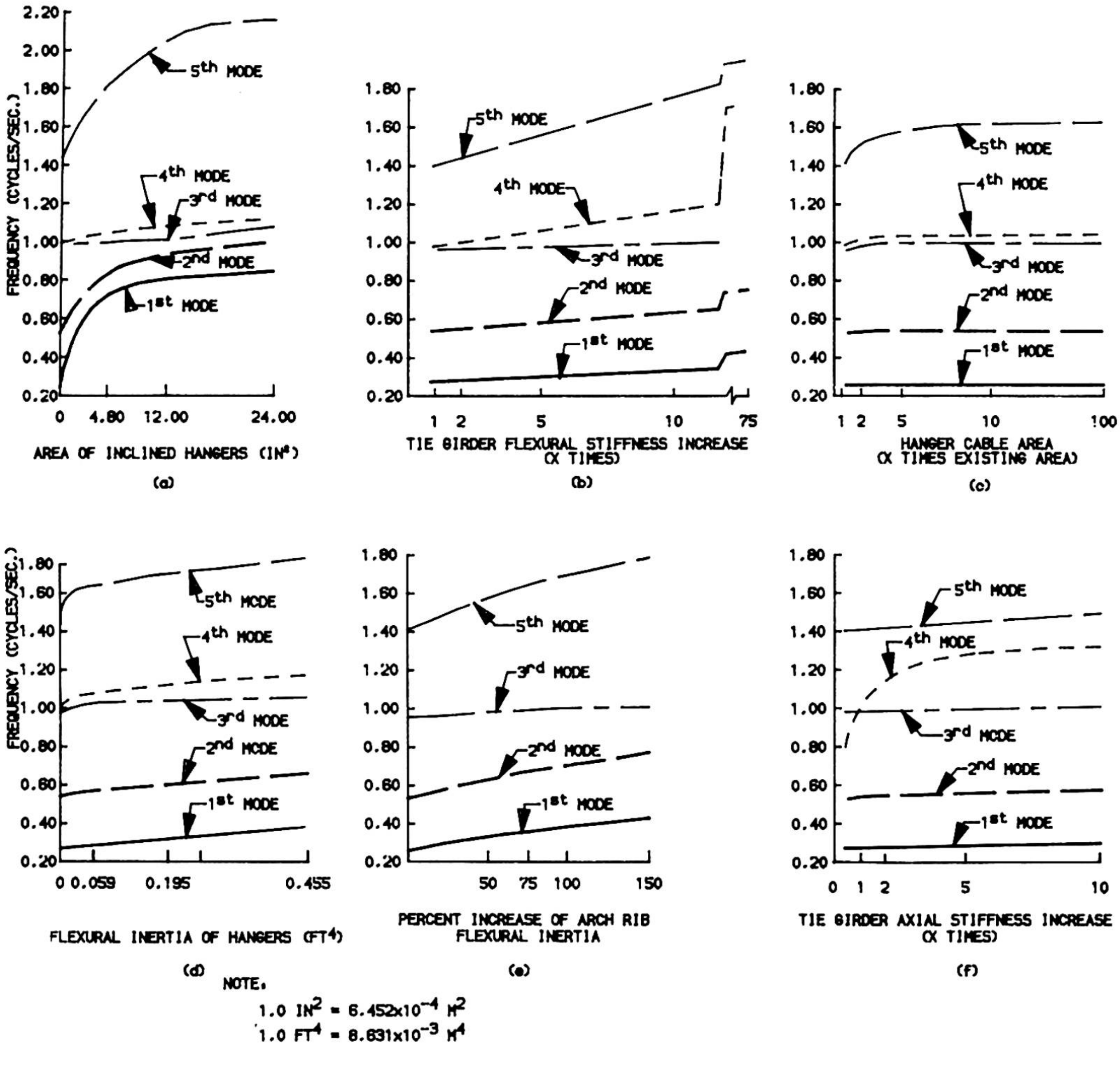


Figure 5 - Structural parameters vs. frequency.

Of the structural changes considered the increase in the flexural stiffness of the tie girder (Figure 5b) and the addition of inclined hangers (Figure 5a) held the most promise. Increasing the axial stiffness of the tie girder or one order of magnitude or more (Figure 5a) is not practical. The increase of hanger axial stiffness by two orders of magnitude (Figure 5c) is totally ineffective. Conversion of the hangers into flexural members (Figure 5d) would be very costly and only marginally effective. Finally, adding flexural stiffness to the already significant arch rib (Figure 5e) is less than practical, both in terms of material used and required height of construction, and is only marginally effective.

The required increase in tie girder flexural stiffness of almost two orders of magnitude indicated in Figure 5 can be achieved relatively easily by converting the tie into the bottom chord of a full-length longitudinal stiffening truss. Inherent in this proposed retrofit is a redundant load path for the originally fracture-critical tie girder. The hypothesized addition of several inclined hangers to supplement the existing hangers has significant effects on the dynamic response of the tied-arch spans. This retrofit basically ties the point of zero vertical deflection at mid-span to the points of maximum vertical deflection at the quarter points. Practically speaking, this alternative converts the tied-arch into a hybrid structure of unknown experience, which is troublesome.

The effect of these two most promising retrofit concepts on the dynamic response under vehicular traffic were studied using the three-dimensional finite element models. A typical comparison of the two concepts with the original response is shown in Figure 6. This comparison shows the time history of quarter point displacements under truck passage calculated for the original configuration and hypothetical retrofits based on (1) adding inclined hangers and (2) conversion of the tie into a stiffening truss. The results of these studies indicate that while both are quite effective in reducing the unacceptable vertical displacement, the stiffening truss retrofit concept is more effective in reducing the crack-inducing distortion of the welded floorbeams. Specifically, the stiffening truss would result in live load dynamic displacements of about 30% of the original movements while the inclined hangers would reduce it to only 15% of original. Stress ranges at details near the center of the floorbeam would be reduced to 40% of original by the stiffening truss, or to 70% of original by the inclined hangers.

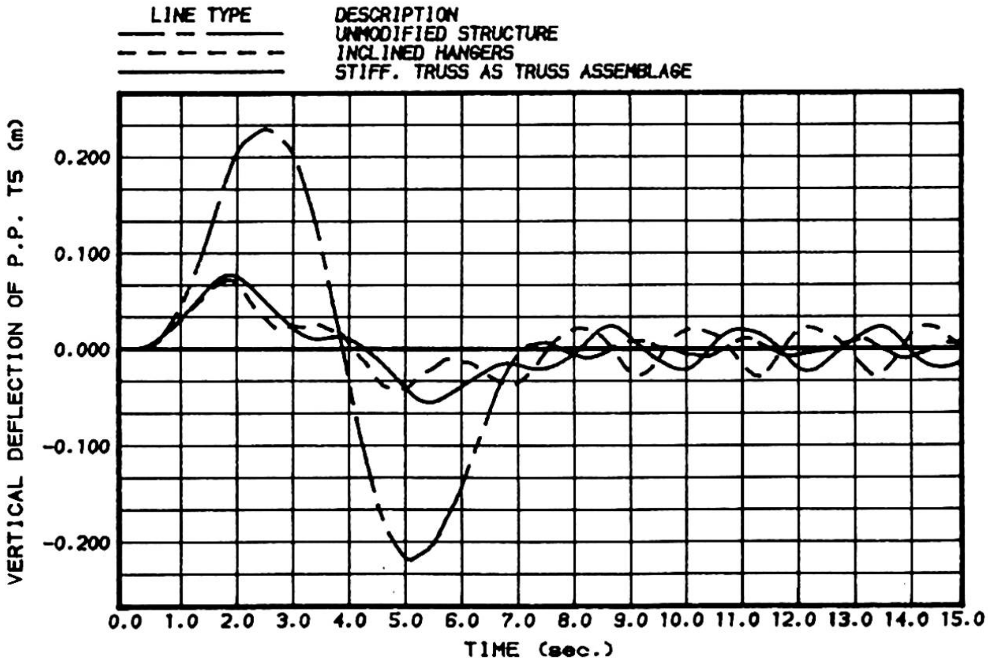


Figure 6 - Quarter point displacements for unmodified and retrofitted structure (223 m span), two trucks at 100 km/h.

3.2 Damping Retrofits

Hydraulic and friction dampers were also considered to reduce the global response of the structure. Since the damping force in hydraulic dampers is proportional to the square of the velocity of the mass, they are primarily used to remove high frequency, low amplitude vibration or flutter and, consequently, are not applicable to this situation. In contrast, friction dampers work independent of velocity and can be used in low frequency applications. However, a threshold displacement is necessary to actuate a friction damper. Although the vertical displacement of the structure is large, the relative displacement between adjacent floorbeams where dampers could be placed did not exceed this threshold.

Tuned mass dampers were also investigated through dynamic analysis. Various numbers and masses of dampers were considered and found to be ineffective. The reason for this observation probably lies in the nature of the excitation, which is basically impulsive, rather than resonant.

4. RECOMMENDATIONS

4.1 Global Retrofit

At present, the owner is considering the recommendation to convert the existing tie girder into the bottom chord of a stiffening truss as an attempt to solve the dynamic response problems of these tied-arch spans. The stiffening truss, while reducing the stress ranges at the details which are cracking, was not considered sufficient to prevent continued fatigue crack development.

4.2 Local Retrofits

The underlying causes of the cracking at the locations shown in Figure 2 were also isolated by three-dimensional finite element substructure analyses. Cracking at the connection of the floorbeam to the tie was found to be caused by the difference in the period of the extension of the tie girder and the out-of-plane rotation of the floorbeam. The period of the extension of the tie girder is equal to the time required for a vehicle to cross the span. The period of the floorbeam out-of-plane movement is approximately one-third of the period of tie girder extension for a vehicle traveling at 100 km/hr. This difference in response results in relative distortions of web gaps in the connection and stiffening details. At the centerline of the floorbeam, the lateral (wind) bracing is connected to the bottom flange of the floorbeam and locally interferes with the out-of-plane movement of the bottom flange resulting from floorbeam rotation caused by the vertical movement of the bridge. The addition of bolted connecting angles to these fatigue-prone details have been suggested to further reduce the small relative displacements occurring at locations of large stiffness changes.

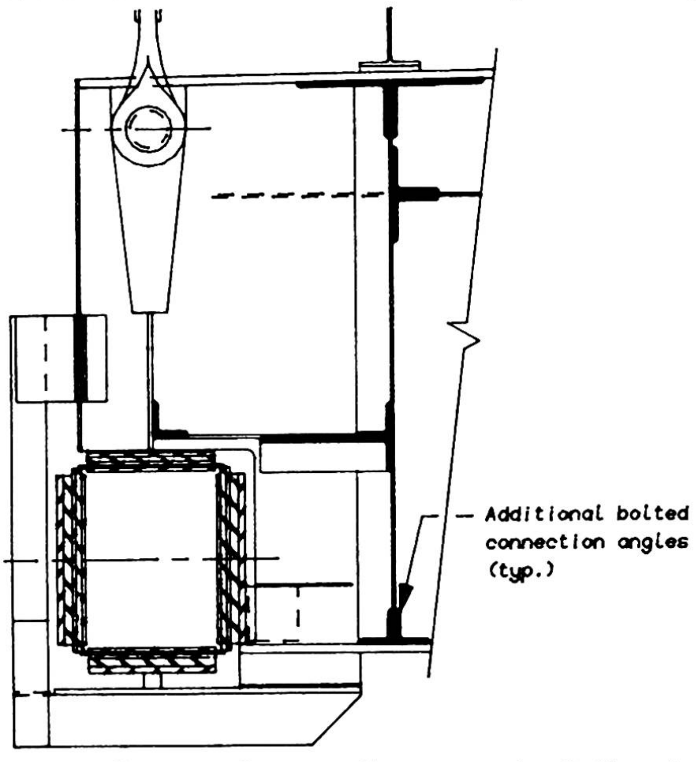


Figure 7 - Local retrofits at end of floorbeam.

Assessment of Condition and Future Service Life of a Railway Bridge

Appréciation de l'état et de la durée de vie résiduelle d'un pont de chemin de fer

Beurteilung von Zustand un Restnutzungsdauer einer Eisenbahnbrücke

Knut ENGESVIK
Senior Research Engineer
SINTEF
Trondheim, Norway

Knut Engesvik, born 1946, received his civil engineering degree at the Norwegian Institute of Technology in Trondheim. He has been involved in research on fatigue of welded structures. Within this area he holds a doctors degree. He has also been practicing structural engineering in a consulting firm.

SUMMARY
The condition of an old steel railway bridge is investigated. The work comprises material testing, strain recordings, condition assessment with respect to fatigue and fracture, and a proposed inspection plan for extended service life.

RÉSUMÉ
L'état d'un ancien pont-rail en acier est examiné. Le travail comprend des essais de matériaux, des mesures de déformations relatives, l'appréciation de l'état de fatigue et de rupture. L'auteur propose un plan d'inspection pour une utilisation prolongée de l'ouvrage.

ZUSAMMENFASSUNG
Der Zustand einer alten Eisenbahnbrücke wurde untersucht. Die Arbeit umfasst Materialprüfung, Dehnungsmessungen, Schätzung des Zustandes hinsichtlich Ermüdung und Bruch, und ein Inspektionsplan zur Verlängerung der Restnutzungsdauer.

1. INTRODUCTION

This paper describes part of an investigation performed by the author under a contract with the Norwegian State Railways.

The structure is a riveted steel bridge built in the year 1900. The bridge was slightly modified in 1934. This modification involved welding, and these details attracted particular interest in the condition assessment, which is limited to fatigue and fracture considerations. In this paper only the most fatigue-prone detail will be treated, while the investigation encompassed various selected details.

2. MATERIAL DATA

Material for testing purposes was provided by replacement of a diagonal member in the horizontal stiffening truss. This was made of Siemens-Martin "fluss-iron". Results from the chemical analysis are given in Table 1. The metallographic investigation showed that the steel microstructure consists of ferrite and pearlite (7%), evenly distributed. The ferrite grain size was 25μm. Tensile testing gave the following result:

σ_y (R_{eH}) = 269 MPa, σ_u (R_m) = 410 MPa, Elongation (A) = 32.4%

Results from Charpy testing are given in Fig. 1. The fractured sections showed that brittle fracture dominated at all test temperatures, although at +20 ^{0}C some ductile fracture was present. The minimum temperature in the bridge is likely to be in the range -20 - -25 ^{0}C. At these temperatures the notch toughness is 6-7 J, which is lower than requirements to ensure a ductile failure mode [1]. CTOD-values were found by 3-point bending tests at -25 ^{0}C. Results are given in Table 2. All tests gave brittle fracture, although with some degree of ductility. The results are seen to demonstrate considerable scatter which is also common in modern steel [2, 3]. Crack growth testing yielded a slope of 2.39 and an intercept equal to -7.807.

3. LOADEFFECT

Strains were recorded at selected points in the structure, in order to establish the distribution of stress ranges. At these points the paint was removed by grinding, and strain gauges were glued on. The strain gauges were connected to data recording and processing units, which perform "rainflow" cycle counting and store the results in histogram form. The recordings were done continuously, with periodic inspection and "dumping" of stored data on a printer. In this way it was possible to decide whether statistical parameters and distributions had stabilized. Stabilization was used as an indication that sampled data represented the train-traffic of today.

4. NDI

Nondestructive inspection was carried out in a part of the structure. For the welded and riveted joints magnetic particle and ultrasonics were used respectively. No cracks were detected.

5. ASSESSMENT OF CONDITION AND REMAINING LIFE

The assessment pertains to the most fatigue prone type of detail, i.e. stringers with welded-on brackets (Fig. 2).

5.1 Stress analyses

Two finite element stress analyses were done. A 3D plate/shell-model yielded the stress concentrating effect at the bracket end, while a 2D plane strain model provided the stress concentration at the weld toe, and the stress distribution through the thickness of the stringer flange.

The weld was modelled with a throat size equal to the nominal size. The local geometry at the weld toe was described by an angle $\theta = 68^0$ and a radius $\rho = 1.8$ mm (Fig. 3). This local geometry was based on recordings of weld geometry. The geometry modelled is the "worst case" of those recorded.

The crack was assumed to initiate at the weld toe (A), and propagate through the flange. For simplicity the crack path was assumed to grow in mode I. Thus, stresses normal to the crack plane (A-A') was employed. This is thought to be a reasonable approximation [4].

5.2 Fracture mechanics analysis

Computations were based on LEFM. The crack propagation life was computed according to the expression:

$$N_p = \frac{1}{C\Delta\sigma_{eq}^m} \int_{a_i}^{a_f} \frac{da}{G(a)(\sqrt{\pi a}\cdot F(a))^m} \qquad (1)$$

where: m = crack growth exponent (Paris eqn.)
C = crack growth coefficient (Paris eqn.)
$\Delta\sigma_{eq}$ = equivalent CA stress range, based on the stress range histogram
$G(\cdot)$ = threshold factor [5]
$F(\cdot) = F_E F_S F_T F_G$ [6]

The crack growth parameters were obtained from the test results. A conservative value of C was obtained by adding 2 standard deviations (log) to the mean value from the regression analysis. Thus, $m = 2.39$ and $C = 2.84\cdot10^{-11}$ (MPa, m, m/cycle).

A semielliptic surface crack at the toe of the weld at the bracket end was considered. The crack shape, a/2c varies during crack growth. This was taken care of by employing the empirical relation [7]:

$$2c = \begin{cases} 9.29a & ;\ a < 1 \text{ mm} \\ 6.71+2.58a; & a > 1 \text{ mm} \end{cases} \qquad (2)$$

The initial crack depth was taken to be $a_i = 0.25$ mm. This is ten times the material grain size, which is a suggested lower limit for the applicability of LEFM to describe crack growth [8].

The critical crack size was estimated at -25^0C. Welding residual stresses equal to σ_y were included, and no account was taken of stress gradients. A calculation based on the CTOD-criterion in PD 6493 [9] yielded a max allowable crack depth, $a_{max} \approx 12$ mm. The minimum value of 5 test values was used for δ_c.

5.3 Condition and remaining life

The fatigue life given by SN-curves is approximately equivalent with the number of cycles required to initiate a crack and propagate it until penetration.

The ECCS recommendations [10] classify the actual detail as category 50. The actual stress range histogram then gives a fatigue life, $T_{SN} = 67$ yrs.

Fracture mechanics computations with $\Delta K_{th} = 3.0\ MNm^{-3/2}$ and $a_f = 20$ mm (=flange thickness) yielded $T_p' = 41$ yrs. Hence, the initiation period was estimated to be

$$T_I = T_{SN} - T_p' = 67-41 = 26 \text{ yrs}$$

The propagation period for the actual case, with $a_f = 12$ mm was found to be $T_p = 34$ yrs. The fatigue life is then given as:

$$T_f = T_I + T_p = 26 + 34 = 60 \text{ yrs}$$

The structural detail under consideration was made in 1934. Thus, in 1987 it has been in service for 53 yrs, i.e. the residual life being 7 yrs.

6. INSPECTION PLAN FOR EXTENDED SERVICE LIFE

6.1 Safety requirements

A safety index $\beta = 3.5$ is recommended if general failure may be triggered by a local failure [10]. This yields the following approximate failure probability:

$$P_f \approx \Phi\ (-\beta) = 1 - \Phi(3.5) = 2.3 \cdot 10^{-4} \qquad (3)$$

where $\Phi(\cdot)$ is the standard normal distribution.

6.2 Probability of failure

In order to satisfy safety requirements an expression for the failure probability must be established which accounts for inspection, e.g. [11]:

$$P_f = 1-[1- \prod_{j=1}^{k} (1-P_D(l_j) \cdot P(I))]^m \qquad (4)$$

where: P_f = probability of failure
$P(I)$ = probability of inspection
$P_D(l_j)$ = probability of detecting a crack with length l_j
l_j = crack length at j'th inspection
k = number of inspections
m = number of cracks

This formula expresses the probability of (during k inspections) not detecting all cracks in the structure before they reach critical size. This pertains to cracks that grow from a tolerable to a critical size during the service period.

6.3 NDI plan

Stringers with welded-on brackets should be inspected with magnetic particle. The detection probability curve given in Fig. 4 is a lower bound to data in [12]. All details of the actual type should be inspected, i.e. P(I) = 1.0. The 1st inspection should be performed no later than 1994.

Details with crack detection should be repaired/strengthened by e.g. HSFG bolts and tension flange splicing plates [13].

At each detail with no crack detection at 1st inspection, one crack with surface length 6 mm is assumed present. This yields a crack growth period of about 28 yrs until fracture, Fig. 5. Thus, after the 1st inspection the bridge can remain in service for about 28 yrs without compromising safety, if the proposed inspection plan is adopted.

The number of inspections during the extended service life period was determined from Eq. (4), which for the actual case yields

$$P_f = 1 - [1 - \prod_{j=1}^{k} (1-0.99\cdot 1.0)]^{40} = 1 - [1-(0.01)^k]^{40}$$

as the number of details is 40, i.e. m = 40. To satisfy the requirement $P_f < 2.3\cdot 10^{-4}$ three inspections are necessary. The crack growth period T_p ($\approx$ 28 yrs) is divided into intervals in such a way that the crack growth increment in the depth direction is the same in all intervals. In this way inspection intervals are determined, Fig. 6.

7. SUMMARY

Data on material, local geometry and load effect for an existing bridge have been obtained and used in a fatigue and fracture condition assessment. In this assessment the traditional SN-Miner approach was combined with the LEFM approach. Simple probabilistic concepts have been employed in order to control reliability during an extension of service life.

8. ACKNOWLEDGEMENTS

Chief engineer P. Hektoen at the Bridge Division of the Norwegian State Railways is gratefully acknowledged for the permission to publish this paper.

% C	0.10	% Al	0.001
" Si	0.01	" Cu	0.17
" Mn	0.53	" Mo	0.01
" P	0.044	" Nb	0.001
" S	0.072	" V	0.01
" N	0.009	" Ti	0.001
" Cr	0.01	" Sn	0.015
" Ni	0.03		

Table 1 Chemical analysis

Specimen no.	CTOD (mm)
1	0.56
2	0.19
3	0.74
4	0.25
5	0.39

Table 2 CTOD test results at -25^{0}C

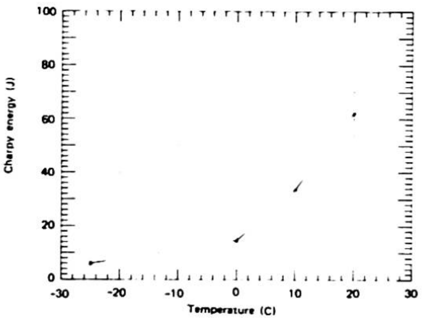


Fig 1 Charpy energy vs temperature

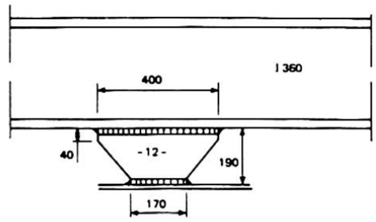


Fig 2 Stringer with welded on bracket

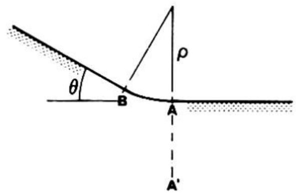


Fig 3 Weld toe geometry

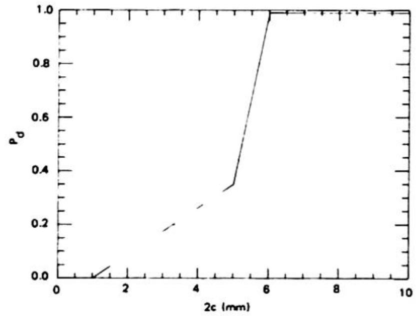


Fig 4 Detection probability (Magnetic particle)

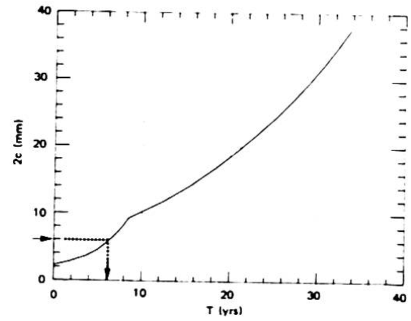


Fig 5 Crack length vs time

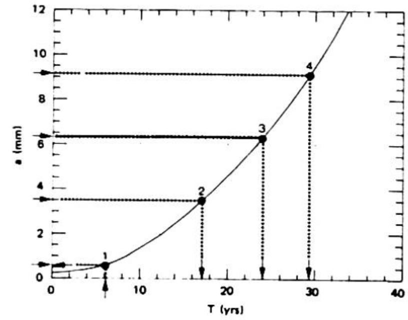


Fig 6 Crack depth vs time

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The Use of Microcores in Structural Assessment

Utilisation de microcarottes dans l'évaluation de l'état des structures

Betondruckfestigkeits – Bestimmung mit Mikrobohrkernen

Pietro BOCCA
Associate Professor
Ist. Univ. di Archit.
Venice, Italy

Pietro Bocca received his degree in engineering from Politecnico di Torino in 1973. He has published more than 40 papers on mechanical behaviour of structures and materials. He is Director of the Non Destructive Experimental Laboratory of the University of Venice.

SUMMARY
The problem of evaluating concrete structures' strength by means of microcores is analyzed. We consider concrete structures both of good and poor quality. For the latter we formulate a combination of test methods and apply Bayes theorem to process experimental data. The highly degraded concrete of a factory building is used as an application example.

RÉSUMÉ
Le problème de l'évaluation de la résistance des structures en béton armé est abordé à l'aide des microcarottes. Différentes structures dont le béton était de bonne ou mauvaise qualité ont été pris en compte. Lorsque le béton est de mauvaise qualité on a proposé la combinaison de différentes méthodes et l'utilisation du théorème de Bayes afin d'élaborer les données. Un bâtiment industriel vieux et délabré a été testé de cette façon.

ZUSAMMENFASSUNG
Das Problem der Druckfestigkeitbestimmung des Betons von Stahlbetonbauten wird anhand von Mikrobohrkernen angegangen. Es werden Bauten guter und schlechter Betonqualität geprüft. Bei letzteren wird eine Kombination der Prüfverfahren empfohlen, bei welchen die Prüfergebnisse mit Hilfe des Lehrsatzes von Bayes ermittelt werden. Als Beispiel wird der degradierte Beton eines alten Fabrikgebäudes geprüft.

1. INTRODUCTION.

Recent papers [1] [2] [3] confirm the possibility of evaluating concrete structure strength by means of microcores analyses. No problems arise if the core diameter for a good quality concretes is reduced to values far below those usually employed [4] [5] [6]. Corresponding to size effects [2] [7] (almost zero) and to acceptable dispersion values [1] there is associated a strength reduction, due to damage and specimen dimension errors, quantifiable within precise limits. With inhomogeneous concrete structures the use of microcores is possible only if combined with other test methods [8] [9]. If this is the case the approach is always of a statistical kind. In the present article we describe a routine, based on the analyses of microcores and cores altogether or, alternatively, of microcores and ultrasonic waves. The fundamental aspect is the application of Bayes theorem [10] to the experimental data. Finally, we apply the routine to a factory building sited in Torino whose structures are quite degraded.

2. MICROCORES AND THE STRUCTURAL PROBLEM.

Microcores' geometrical characteristics are optimized for a good evaluation of the material strength. We chose a ratio H/D = 1 (H is the height and D is the diameter) in order to avoid problems of form effects when comparing the thrust values of standard cubes. For normally employed concretes in the structural field and with a maximum inert diameter up to 16 mm one can sample microcores with a diameter D = 2.8 cm [1].

As already said, the procedures of evaluating the strength are functions of the type of structure.

Microcore tests furnish sufficient information for a detailed picture of the strength if the material is a homogeneous concrete (*).

Poor quality concrete structures degraded or damaged by external factors, for example fire, the mentioned test should be combined with other methods: quasi-destructive or non-destructive [8] [9] [11]. Among these, the following are particularly useful:

- Generalized microcore sampling for the total structure and a core sampling only in important points;
- Microcores sampling and ultrasonic tests on the structure;
- Statistical data processing by means of Bayes theorem.

3. GOOD QUALITY CONCRETE STRUCTURES.

The cubic compression strength R of homogeneous concrete structures is obtainable from the crushing strength of microcores R_{mic} by means of the following expression:

$$R = k \cdot R_{mic} \tag{1}$$

where $k = k_o \cdot k_1$ - the factor k_o accounts corrections due to size effects - the factor k_1 accounts corrections due to microcores damage wich occur during sampling and to geometrical errors.

Fig. 1 is the plot, in bilogarithmic scale, of the compression strength of concretes with different homogeneity as function of core diameter [2]. To zero slope on the curve corresponds zero size effect and k_o is equal to 1.

Correction factors are sketched in fig.2 as functions of the concrete homogeneity (represented by the variation parameter C_v) based on performed experiments. For $C_v < 5\%$, from figure 2, one computes the following mean values:

$k_o = 0.95$; $k_1 = 1.20$ (for accurate microcore sampling).

And finally we have from (1):

(*) The homogeneity index can be assumed as the coefficient of variation values C_v. Some authors assume $C_v < 5\%$ for homogeneous concretes and $C_v > 5\%$ for inhomogeneous one.

(2) R = 1.14 R_{mic}

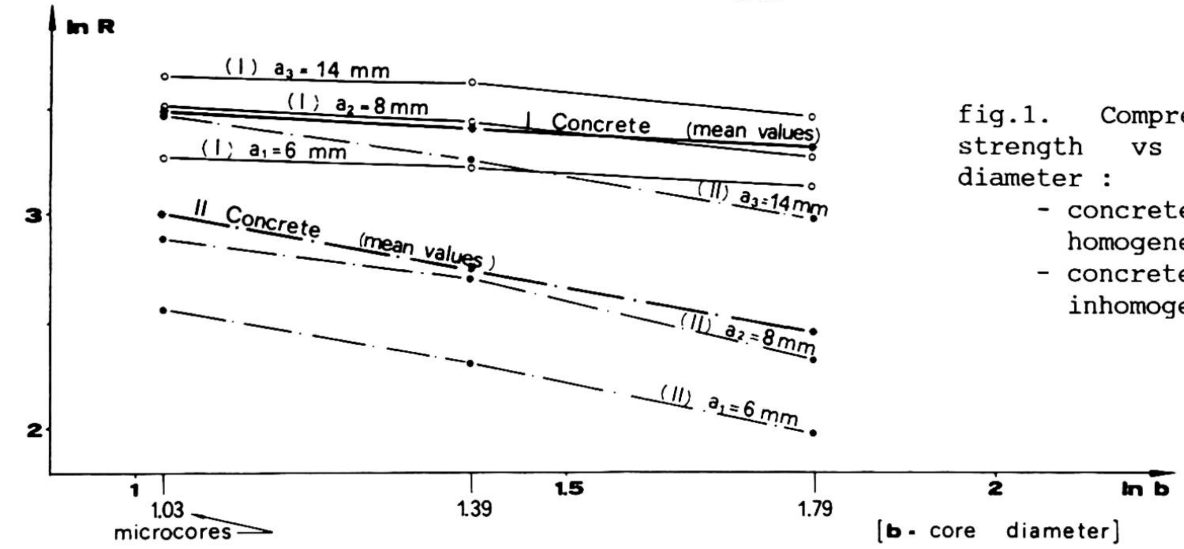

fig.1. Compression strength vs core diameter :
- concrete I homogeneous
- concrete II inhomogeneous

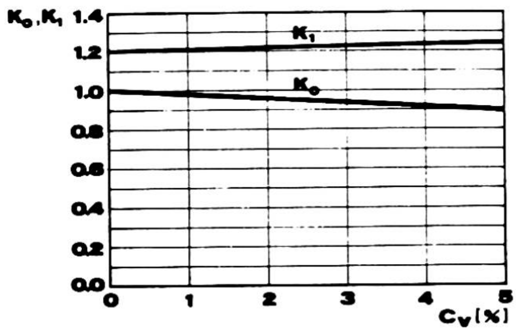

fig.2. Correction factors k_o, k_1 vs concrete variation parameter c_v.

4. POOR QUALITY CONCRETE STRUCTURES

Due to size effects, sample damage and dispersion values the use of only microcore tests, as a rule, should be avoided when estimating the strength of inhomogeneous concrete structures.

Better results are obtained with one of the following methods:
- 1. Microcore and core sampling;
- 2. Microcore sampling and ultrasonic tests.

We illustrate now how Bayes theorem works when data are treated statistically.

In the Bayesian approach the unknown parameters are considered random variables. They are, for example, the mean concrete strength R = M and the standard deviation $\delta = \Sigma$. The estimation of the true value of such parameters is obtainable from the posterior probability density. The scheme is sketched in fig. 3.

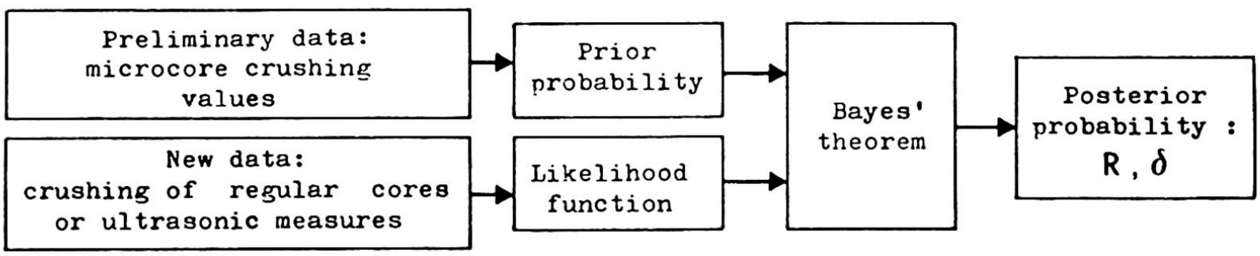

fig.3.

Here we hypothesyze that the distribution of R , the concrete strength, is the same as the one determined by means of microcores, of cores and ultrasonic tests. With such a condition one writes [10]:

$$(3) \qquad f''_{M,\Sigma}(m,\sigma) = f'_{M,\Sigma}(m,\sigma)\ L(m,\sigma/x_1,\dots,x_n)$$

We can use, in virtue of the above equation, instead of the prior estimate $f'_{M,\Sigma}(m,\sigma)$ of R with a posterior estimate $f''_{M,\Sigma}(m,\sigma)$ with the introduction of new parameters x_j, i.e. core crushing or ultrasonic measure.

From (3) the posterior density $f''_{M,\Sigma}(m,\sigma)$ is given by the product of the prior distribution $f'_{M,\Sigma}(m,\sigma)$ and the likelihood function $L(m,\sigma/x_1,\dots,x_n)$, apart from the normalizing constant N.

The l.h.s. of eq. (3) written more explicitily looks like:

$$L(m,\sigma/x_1,\dots,x_n) = \{(\sigma)^{-1}\cdot\exp(A)\}\cdot\{(\sigma)^{1-n}\cdot\exp(B)\}$$

$$A = -\tfrac{1}{2}[(m-\bar{x})/(\sigma\cdot\sqrt{m})]^2 \qquad B = -\tfrac{1}{2}(n-1)\cdot(s/\sigma)^2$$

n, $\bar{x}$, s^2 represent size, mean and variance of the cores or of the ultrasound values; $\bar{x} = \Sigma x/n, \quad s^2 = \frac{1}{n-1}\Sigma(x_i - \bar{x})^2$

$$f'_{M,\Sigma}(m,\sigma) = \left\{\frac{1}{\sqrt{2\pi}\,\sigma/\sqrt{n'}}\exp\left[-\frac{1}{2}\left(\frac{m-\bar{x}'}{\sigma/\sqrt{n'}}\right)^2\right]\right\} \times \left\{\frac{\left(\frac{n'-1}{2}\right)^{\frac{n'-2}{2}}}{\Gamma\left(\frac{n'-2}{2}\right)}\frac{2}{s'}\left(\frac{s'^2}{\sigma^2}\right)^{\frac{n'-1}{2}}\exp\left(-\frac{n'-1}{2}\frac{s'^2}{\sigma^2}\right)\right\}$$

where n',x' and s'^2 are the parameters of the prior joint density function of the microcores values.

- N is the normalizing constant - $f''_{M,\Sigma}(m,\sigma)$ has the same form as the prior joint density function, with different parameters obtainable from the following relations:

$$(4) \qquad n'' = n + n'$$
$$(5) \qquad \bar{x}'' = (n\bar{x} + n'\bar{x}')/(n + n')$$
$$(6) \qquad s''^2 = [(n-1)s^2 + (n'-1)s'^2 + n\bar{x}^2 + n'\bar{x}'^2 - n''\bar{x}''^2]/(n''-1)$$

The mean M and the variance Σ are obtained by integration of equation 1., we have then:

$$(7) \qquad \mu_M = \bar{x}''$$
$$(8) \qquad s^2_M = s''^2(n''-1)/[n''(n''-2)]$$

the mean and variance of the standard deviation are:

$$(9) \qquad \mu_\Sigma = s''[(n''-1)/2]^{\frac{1}{2}}\Gamma[(n''-3)/2]/\Gamma[(n''-2)/2] \qquad n'' > 3$$
$$(10) \qquad s^2_\Sigma = s''^2(n''-1)/(n''-4) - (\mu_\Sigma)^2 \qquad n'' > 4$$

5. EXAMPLE

In order to illustrate the routine the following example is useful. to establish the recoverability or not of a factory building it is necessary to evaluate the strength of the horizontal carrying parts. The three floor factory was built at the turn of the century (see fig.4)

One easely observes that the eastern part is highly degraded (double floors 7 ÷14 in fig.4). Here we realize a first sounding with microcores. The sampling is uniformly distributed between chords and secondary beams. While 15 cm diameter cores where sampled after and limited in number. We summarize the results in table 1. The mean strength, computed from eqs. (5) (7), is reported in the last column.

In tab. 2. are reported the initial distribution parameters of all values obtained with microcores ($n', s'^2, \bar{x}^2$) together with those (n, s_t^2, x_t) obtained from cores relative to the east part.

FLOOR	STRUCTURAL ELEMENT	MICROCORES		CORES 15 CM		
		n'	$\bar{x}'$ [MPa]	n'	x' [MPa]	
1°	CHORDS	8	14.6	3	11.7	13.8
1°	SECONDARY BEAMS	8	15.1	3	17.8	15.8
2°	CHORDS	10	14.2	4	13.6	14.0
2°	SECONDARY BEAMS	08	16.6	3	12.9	15.6
3°	CHORDS	08	08.7	3	09.0	08.8
3°	SECONDARY BEAMS	08	10.1	3	09.3	09.9

tab.1.

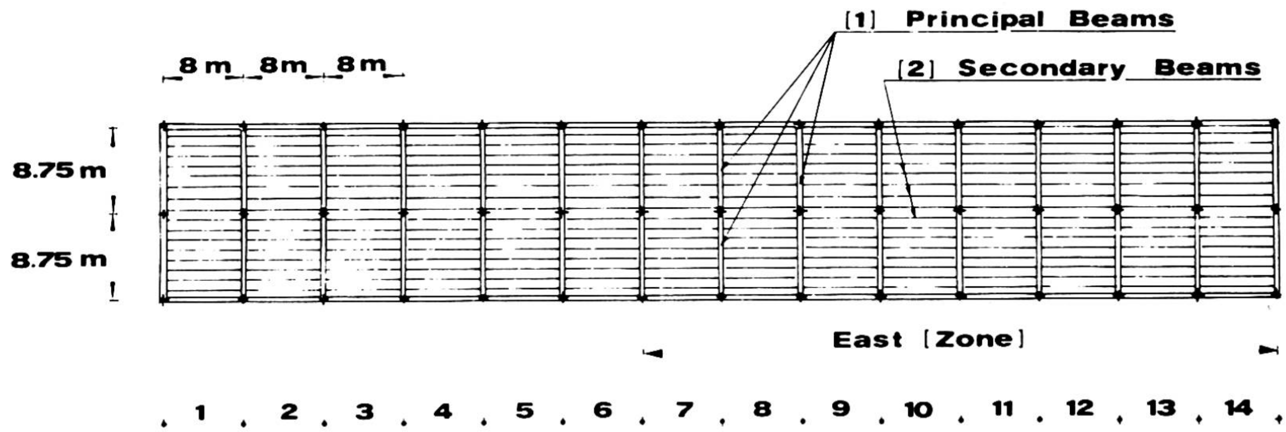

fig.4. Industrial building map.

Prior distr. microc.			New data -15cm Cores			Mean Strength	
n'	x'_t [MPa]	s'_t [MPa]	n	x_t [MPa]	s^2_t [MPa]	x''_t [MPa]	s^2_M [MPa]
50	13.2	$(6.8)^2$	19	12.4	$(6.0)^2$	13.0	6.2

tab.2.

Worth noting is the small difference between the value x' (i.e. mean strength of microcores) and x_t. Probably this is due to a combination of size effect with sample execution errors. It is interesting to note that the values s and s are quite similar. In such a case one can reduce the variance of the mean strengths computed s_M^2 (last column of table 2) obtained from eq. (6) (8), i.e.

$$(11)\quad s_M^2 = (n''-1)[n''(n''-2)(n''-1)]^{-1}[(n-1)s_\xi^2 + (n'-1)s'^2_\xi + n\overline{x}_\xi^2 + n'x'^2_\xi - n''\overline{x''}_\xi^2$$

increasing indifferently the number n of microcore samples or the number of core samples. In a different fashion, while keeping s_M^2 constant one can decrease the number n' (cores) and increase n (microcores) according to (11). One can easily imagine the practical advantages.

NOTATION:

M,Σ = mean value, standard deviation of strength
m,σ = values of M and Σ
μ_Σ = mean value of standard deviation
s^2_M = variance of mean value
n,x,s^2 = size, mean and variance of new values
μ_M = mean value of M
R,δ = mean value, standard deviation of compressive strength
n,x,s'^2,n'',x'',s''^2 = parameters of prior and posterior joint density functions respectively
x'' = R

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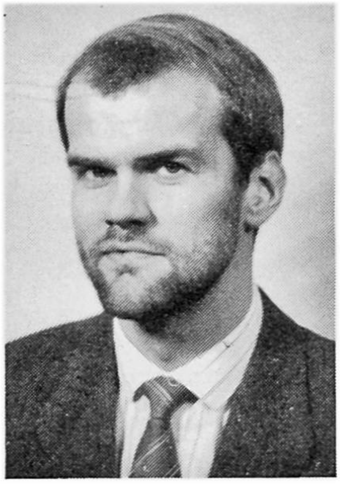
Fracture Mechanics Analysis of Fatigue in Plate Girders

Analyse de durée de fatigue relative aux poutres en acier

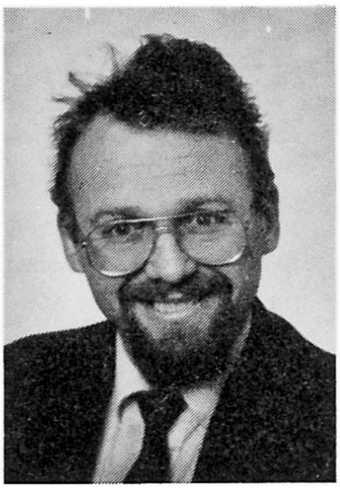
Bruchmechanische Analyse der Ermüdung von Plattenträgern

Heinz-Joachim WESSEL
Dr. Ing.
SALZGITTER AG
Berlin



Wessel, born 1953, received his Dr. Ing. degree from the Norwegian Institute of Technology (NTH), Trondheim. During a four-year research work at the NTH he was involved in fatigue problems. He is now working for a large industrial group in the FRG, involved in R&D concerning offshore technology.

Torgeir MOAN
Professor
Norwegian Inst. of Techn.
Trondheim, Norway



Torgeir Moan, born in 1944, received his M.Sc and Dr. Ing. degrees from NTH in 1968 and 1975, respectively. Since 1978 he is a professor of marine structures. His main research interests are structural analysis and safety, including the development of design criteria.

SUMMARY
A theoretical study of the fatigue life of large scale plate girders subjected to variable amplitude loading is presented in this paper. Particular attention was paid to the remaining fatigue capacity after a surface crack had penetrated the plate material. Although the fatigue capacity subsequent to the penetration of the web plate is relative small it represents a valuable information for the planning of inspection and repair of the component.

RÉSUMÉ
Ce compte rendu présente une étude théorique sur le comportement à la fatigue de poutres sous des charges variées. Une attention particulière est portée sur la capacité résiduelle de résistance à la fatigue après qu'une fissure en surface ait pénétré le matériau. Bien que cette période soit relativement courte, elle représente une information très importante pour la planification des contrôles et des réparations.

ZUSAMMENFASSUNG
Dieser Bericht präsentiert eine theoretische Studie zur Ermittlung der Dauerschwingfestigkeit von Plattenträgern unter variierender Belastung. Besondere Beachtung wurde der zusätzlichen Lebensdauer geschenkt, die verbleibt, nachdem ein Oberflächenriß das Plattenmaterial durchdrungen hat. Wenn dieser Zeitraum auch relativ kurz ist, so stellt dies doch eine wertvolle Information zur Beurteilung von Inspektionsintervallen und Reparaturvorhaben dar.

1. INTRODUCTION

The overall purpose of this study was to investigate fatigue crack growth in large scale plate girders with special emphasis on the stage where a through crack approaches the girder flange. A general fracture mechanics approach was developed and particulary verified against test results for large-scale test girders of the type shown in Fig. 1.

2. FRACTURE MECHANICS ANALYSIS

2.1 Fracture mechanics approach to fatigue

With the existence of an initial flaw fracture mechanics methods can immediately be seen to offer a powerful and realistic means for analysing the fatigue strength of welded joints. It was shown by Irwin [1] that the elastic stresses in the region adjacent to a crack tip depend on the stress intensity factor K. The general form of K is

$$K = \sigma \sqrt{\pi a} \cdot F \qquad (1)$$

where: σ = in-plane uniform stress applied remotely from the crack; a = half-crack length in the case of a through-crack or crack depth in the case of a surface crack; F = correction factor to account for the actual configuration of the body.

2.2 Fatigue crack propagation

An appropriate expression for the crack propagation rate is the increase of the crack length "a" per load cycle, viz. da/dN. This rate again is dependent on the alternating stress intensity ΔK.

An empirical relationship that adequately characterizes the crack growth rate in most types of welded steel materials has been suggested by Paris and Erdogan:

$$\frac{da}{dN} = C \cdot (\Delta K)^m \qquad (2)$$

where C and m are generally taken as material constants.

2.3 Stress intensity factor

For a wide range of standard situations closed-form expressions for the stress intensity factor K exist. However, because of the complexities of most practical problems, exact solutions are not available. Two different approaches to estimate K have been applied in this work and will be presented in the following.

2.3.1 Empirical solution

The first and most convenient approach is the use of an empirical equation for the stress intensity factor for a surface crack, presented by Newman and Raju [2]. This equation reads

$$K = (\sigma_t + H \cdot \sigma_b) \cdot \sqrt{\pi a/Q} \cdot F\ (a/t,\ a/c,\ c/b,\ \Theta) \qquad (3)$$

where: σ_t, σ_b = remote tension and bending stresses respectively; H = function, dependent on a/t, a/c, Θ; Q = shape factor for elliptical crack; F = boundary-correction factor; t = plate thickness; c = half length of surface crack; b = half-width of cracked plate; Θ = parametric angle of the ellipse.

2.3.2 Hybrid method

The stress distribution and hence the value of the stress intensity factor at the tip of a crack in a structure or component will be influenced by edges, surfaces etc., called "boundaries" in the following. The present method may be used to estimate the value of the stress intensity factor for a crack in a structure containing several boundaries by separating a complex configuration into a number of simpler auxiliary configurations each containing the crack and a single boundary. For this specific boundary solutions are available (e.g. [3] and [4]) for two- or three-dimensional crack problems in finite sheets and bodies. Many of these solutions define the stress intensity factor as

$$K = F \cdot \sigma \cdot \sqrt{\pi a} \tag{4}$$

with F being a correction factor for this configuration. This simplified, cost-effective technique is described more comprehensively in [5]. The total correction factor for the detail under consideration is the product of the single correction factors for the auxiliary configurations.

3. STRESS ANALYSES

It is obvious that the knowledge of the state of stress in the vicinity of the potential crack site is a prerequisite to any fatigue life calculation. At the Marine Structures Laboratory of the Norwegian Institute of Technology in Trondheim large-scale structural models have been tested under fatigue loadings. The type of specimen that is analyzed in the present work is a welded, built-up plate girder with transverse stiffeners on one side (cfr. Fig. 1). Weld-induced misalignments were present in the entire structure. Stress analyses that were conducted by means of the Finite Element Method provided the stresses in the structure in the length and width dimension as well as in the thickness direction at the end of one specific stiffener.

4. FATIGUE CRACK GROWTH CALCULATIONS

4.1 Computation of fatigue life

The fatigue crack growth calculations were based on a computer program "LIFE" [5]. A modified version of "LIFE" allows for the fatigue calculation of the actual structural detail, the welded plate girder.

In an I-girder the extension of a crack due to cyclic loading takes place in four stages:

- From an initial defect to the penetration of the girder web. (Stage 1)
- Along the stiffener-to-web weld to the tension flange. (Stage 2)
- Through the flange until this is penetrated. (Stage 3)
- As a through crack towards the ends of the flange until the total remaining intact cross section of the flange fails due to yielding. (Stage 4)

4.1.1 Surface crack in the web (Stage 1)

The early crack growth was calculated by means of Eq. 3. In the actual case of a plate girder the through-thickness stress distributions were far from linear. Hence, Newman-Raju's equation was corrected with regard to stress gradients.

The number of load cycles from an initial crack depth "a_i" to a certain depth "a" is according to Eq. 2

$$N = \frac{1}{C} \cdot \int_{a_i}^{a} \frac{da}{(\Delta K)^m} \tag{5}$$

The Stage 1 life spans calculated lay well within the μ and $\mu+2$ scatterband in the S-N approach.

4.1.2 Through crack in the web (Stage 2)

After the crack had penetrated the web plate thickness, it propagated both upwards and downwards along the stiffener weld. The further crack growth was determind by means of the hybrid method mentioned earlier. Here the global membrane stress distribution governed the growth. The magnitude of stresses and consequently the stress intensity factors were different at the two tips of the crack. The correction factors that account for the actual configuration of the body allowed for the effect of non-uniform stresses and the presence of the girder flange ahead of one of the crack tips.

In this second stage of crack growth the growth rate was different at the two crack tips. Hence the crack extension was controlled alternating at the two tips with the crack length "a" increasing with small increments.

4.1.3 Crack within the flange

The further considerations apply for the lower crack tip only. It was this tip that was decisive for the fatigue life of the detail.

Fisher [6] examined a fatigue crack failure in a steel bridge where the cracked detail was very similar to the actual plate girder. He based his calculations on the assumption that the crack extended into the flange as a semi-circular crack, the centre being the point of crack initiation. Both the welds and the embraced air split between the plates were disregarded. This idealization was adapted in the present investigation since Fisher`s results were found to be in good agreement with the actual field behaviour of those details. Several correction factors account for the different boundaries. Once the crack had penetrated the flange the further crack growth in a plain steel plate (the flange plate) could be considered to occur under uniform tension load.

4.2 Comparison between theoretical and experimental results

The validity of the fracture mechanics model was checked by comparing the computed fatigue lives with a-N-results from the laboratory tests. The subjects of this comparison were two quite similar plate girders with web plate thicknesses t=20 mm and t=40 mm.

A regression analysis concerning the crack shape development (a/2c) had been performed. This analysis was based on crack surface recordings from the laboratory tests. The resulting regression lines were different for the two web plate thicknesses.

As for Stage 1 of crack growth, the crack being a surface crack, the a-N calculations agreed well with the experimental data when initial crack depths in the range from 0.1 to 0.2 mm were assumed.

For Stage 2, the approach of the lower crack front to the girder flange is demonstrated in Fig. 2. As the crack front in the real case generally does not proceed as a straight line two curves for the maximum and minimum distance from the flange are given. In the case of Fig. 2 the crack did not reach the flange in the course of the tests. Therefore the curves from the test results cease at a distance from the flange larger than zero.

Laboratory test results for the final two stages were not available since those cracks which had extended into the flange had been repair welded.

The calculated relative fatigue lives for the respective stages turned out to be the same for both of the web plate thicknesses:

Stage 1: 92 %
Stage 2: 6 %
Stage 3: 2 %

4.3 Sensitivity studies

In the following the sensitivity of the fatigue life to changes in the most important quantities will be demonstrated.

The difference in Stage 1 fatigue life for the two plate thicknesses corresponds approximately to a factor 2.0, the thinner plate having the longer life.

Increasing the assumed initial crack depth within a reasonable range by a factor of 3.0 causes a decrease in fatigue life by 26 % for the 20 mm case and 20 % for the 40 mm case.

The dependence of the shape of the crack front (a/2c) on the crack depth had been expressed in terms of results from several regression analyses. The conclusions with respect to these circumstances are twofold. Firstly, a crack growth law obtained from a thin plate analysis should not be applied to larger thicknesses, and secondly the assumption of a crack growth law from a large plate regression analysis may cause an error, but this error is in the conservative direction.

With a stress gradient as in the plate girder the main crack will not initiate very far from the end of the stiffener since the nominal stress along the stiffener is highest at this place. It is obvious that, the shorter the stiffener, the lower the stress field at its end and the longer the fatigue life in Stage 1 and, not least in Stage 2. A thorough evaluation of the two phenomena buckling and fatigue may lead to an optimum solution of this problem. A possible gain in endurance of several 10 % for a shorter stiffener demonstrates that this can be as advantagous as e.g. time consuming post-weld improvement methods.

In order to find the significance of the weld toe radius this parameter has been altered within the range from 0.67 to 2.0 mm. The difference between the lowest and the highest of the calculated endurances is 10 %, what is not very much seen in relation to other influencing parameters. An expensive weld surface treatment to increase the toe radius, however, would also reduce the size of a present initial surface defect, or even remove this totally. Thus, a twofold improvement of the fatigue capacity might be achieved by grinding, shot peening etc.

In design the statistical nature of fatigue behaviour must be taken care of by applying a crack growth parameter C that provides 97.7 % probability of survival, what signifies that the C-value from the upper bound 95 % confidence limit has to be applied. In the course of the calculations it turned out that the endurance estimated on the basis of the 95 % confidence limit is only about 35 % of the endurance based on the mean C value.

5. CONCLUSIONS

A fracture mechanics approach has been developed and used in the analysis of plate girders which have been tested in the Marine Structures Laboratory of the Norwegian Institute of Technology. The trend in crack growth is well predicted. However, the endurance in the first stage from an initial defect to a through-thickness crack is uncertain because of the uncertain size of the initial defect. The fracture mechanics approach is particularly useful in tracing the later stages in crack growth which are not covered by S-N data and which are useful in planning inspection and repair [7].

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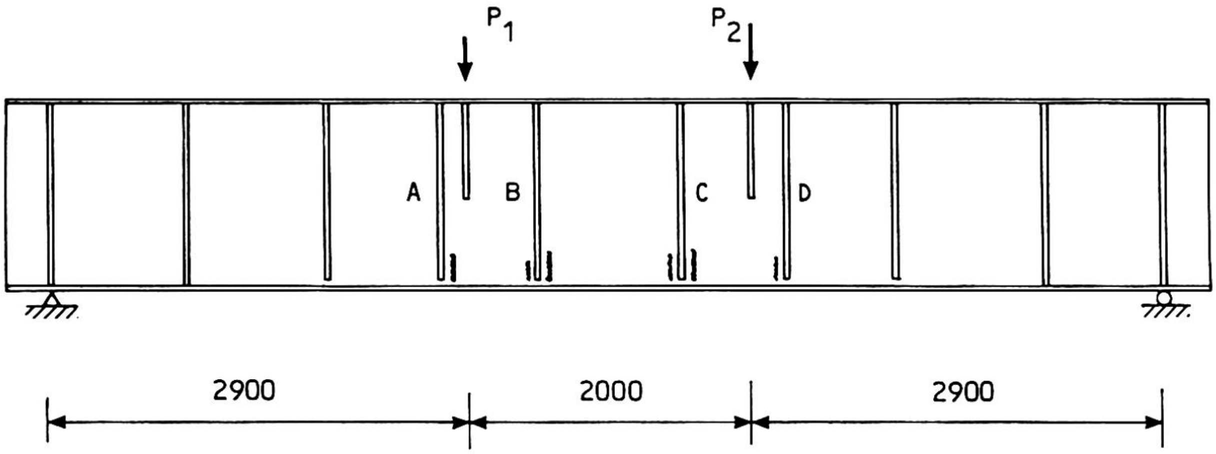


Fig. 1 Plate girder; fatigue cracking at lower ends of stiffeners A-D.

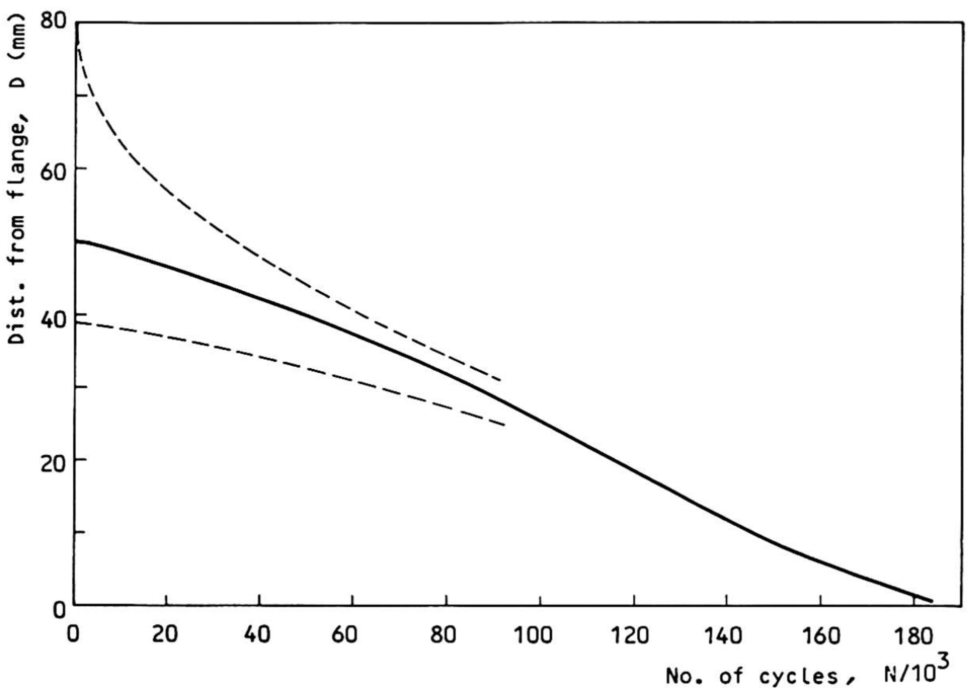


Fig. 2 Crack growth in Stage 2
Solid line: Calculated values
Dashes lines: Measured values for D_{max} and D_{min}.

Shock Transmission Units for Bridge Strengthening

Unités de transmission de chocs pour le renforcement des ponts

Stossübertragungselemente zur Erdbebensicherung von Brücken

Brian PRITCHARD
Dir. Transp. Eng. Div.
WS Atkins & Partners
Epsom, Surrey, UK

Brian Pritchard, born 1926, obtained his civil engineering degree at the University of Wales, UK, and a later Masters degree at Washington University, St. Louis, USA. Currently the Director in charge of the Bridge Section of Atkins' Transportation Engineering Division, he has 30 years' experience of international bridge design, construction, maintenance & research.

SUMMARY
The paper introduces a shock transmission unit which has been developed to meet some of the challenges of designing new bridges to resist earthquake effects and the strengthening of existing bridges. The unit and some recent applications are described.

RÉSUMÉ
L'article présente une unité de transmission de chocs qui a été développée pour répondre à certaines des exigences de conception des nouveaux ponts devant résister aux effets des tremblements de terre et pour le renforcement de ponts existants. L'unité et certaines applications récentes sont décrites.

ZUSAMMENFASSUNG
Der Beitrag stellt ein Stoß-Uebertragungselement vor, das entwickelt wurde, um einigen Herausforderungen im Hinblick auf Erdbebensicherheit bei der Konstruktion neuer und der Verstärkung bestehender Brücken zu begegnen. Das Element und einige Anwendungen werden beschrieben.

1. INTRODUCTION

Shock Transmission Units (STUs) capable of acting as a rigid member under impact loading whilst permitting slow axial movement without resistance have found important applications in all types of engineering over the years. However, their use to date in bridge engineering has been very limited. This had probably arisen because, hitherto, the STU has been a relatively complex device with a high initial cost and a need for regular maintenance and adjustment.

A special bridge STU, developed some years ago in the UK offers a new design with several advantages. It is the purpose of this paper to describe the unit and some recent successful uses in meeting the challenges of designing new earthquake resistant bridges and the strengthening of existing bridges by inducing beneficial load sharing in the bridge substructures.

2. THE 'NEW' STU FOR BRIDGING

The special bridge STU was developed in the UK some years ago and is only referred to as 'new' because it has taken some time for its benefits to be recognised sufficient to actually using it in several recent bridge structures.

The new STU introduces a simpler approach to the design of shock transmission units, sometimes erroneously referred to as dampers. Hitherto these units have been complex precision hydraulic devices but the new unit, with only a single moving part, is much simpler, more economical, robust & virtually maintenance-free.

Instead of oil the STU utilises the peculiar properties of "bouncing putty", a silicone compound which will readily deform under slow pressure but becomes rigid under impact. For all practical purposes the viscosity of the material remains constant throughout a wide temperature range. Thus the new STU can be relied upon to perform consistently under most climatic conditions.

The unit, Figures 1 & 2, is of extremely simple construction, consisting of a steel body or cylinder containing a loose fitting piston fixed to a transmission rod, the void round the piston being filled with the bouncing putty. Under slow movement this putty is squeezed slowly around the piston and displaced from one end of the cylinder to the other. The transmission rod passes through the entire length of the cylinder so that the volume of the cylinder remains constant at all positions of the piston.

The new STU has been designed primarily to function in a horizontal position but it can be adapted for vertical movement by incorporating an internal spring to return the piston to the neutral position.

The bridge units are made to resist impacts ranging between 10 & 120 tonnes, with larger requirements satisfied by increasing the number of units. Movement rates are controlled by the clearance around the piston, the usual 50 tonne unit giving a typical rate of extension of some 10mm/minute, more than adequate to meet the zero resistance slow movement demands of bridge decks arising from temperature, shrinkage & creep. Typical impact resistance/time behaviour for such a unit requires that the extension or compression shall not exceed 2mm in the first ten seconds nor 4mm in the first 20 seconds after the impact application.

3. BRIDGE STUs IN EARTHQUAKE ZONES

3.1 General

Bridge STUs have found a good application in resisting earthquake effects on the substructures of bridges located in such zones. The following sections describe the ideal substructure articulation for a typical multispan flyover and the assistance provided by STUs in adding earthquake resistance to the articulation. A large flyover using STUs for this purpose and recently built in Kuwait is described.

3.2 Ideal Multispan Bridge Deck Articulation

Multispan bridge decks or flyovers are best designed as continuous, offering not only deck & substructure economy plus ride quality but also eliminating trouble-prone movement joints over piers.

The substructure articulation is generally arranged as shown in Figure 3, less the STUs, to provide economical minimum resistance to longitudinal deck movement. A fixed rotation bearing is located at the central pier and rotation/moving rubber, sliding or roller bearings at the remaining piers and the end abutments, where the only deck joints are located.

This arrangement allows the deck to move longitudinally with minimum restraint to take up the effects of temperature, and where appropriate,. concrete shrinkage and creep. Any longitudinal deck forces arising from traction, braking or wind are resisted by the central fixed pier, assisted by friction or shear generated by deck movements at the other piers and the abutments. If the longitudinal restraint force at the central fixed pier is too much in excess of the friction or shear forces on the other piers, economy would dictate size differentials between the central & other piers. This is aesthetically undesirable and pier sizing equality can often be obtained by sharing the restraint force among two or three central piers, depending upon the extra forces generated in those piers by restrained temperature, shrinkage or creep movement.

3.3 Adding Earthquake Resistance using STUs

The longitudinal forces generated in a bridge deck subjected to earthquake, a function of the deck mass and usually well in excess of any traffic braking or traction, require the development of considerable longitudinal restraint from the substructure at bearing level. For the typical viaduct described in 3.2 this restraint would overload the central fixed pier. Ideally this effect, hopefully rare, should be resisted equally by all the supports. However, they cannot be designed to be fixed like the central fixed pier because of the considerable deck movement forces which would arise in normal service.

This is where the STU comes into its own, offering no resistance to normal service deck movement but providing a fixed pier connection during earthquake impact. Figure 3 shows the typical arrangement of STUs at the 'free' piers which temporarily convert the piers to 'fixed' to permit beneficial load sharing during earthquakes. If necessary, additional STUs can be mounted at the abutments to add further load sharing.

3.4 Interchange 3 Viaduct, Outer Bypass, Kuwait

This large viaduct, Figure 4, recently built for the Ministry of Public Works in Kuwait, is designed to resist earthquake forces in accordance with the 1973 AASHTO code with an equivalent lateral force coefficient of 4%.

It is some 800m long and curved in plan with 13 spans of 55.1m and 2 end spans of 41m. The continuous prestressed insitu concrete deck is of 5 cell hollow box construction, 2.75m deep and 15.12m wide. The elliptical piers are of reinforced concrete construction 5m wide and 1.75m maximum thickness.

Pier bearings are twin fixed at central pier 8 and twin PTFE sliding at all other piers. Longitudinal earthquake forces are resisted at pier 8 and also at piers 6, 7, 9 & 10 by twin 120 tonne STUs, Figure 5.

4. STUs FOR STRENGTHENING EXISTING VIADUCTS

4.1 General

A large number of our existing stock of viaducts feature long sequences of simply supported deck spans, often supported on a series of high & substantial piers. This is particularly evident in major river crossings where high navigation clearances require long approach viaducts.

The piers under each simply supported span inevitably carry fixed bearings for one span alongside free bearings for the adjacent span. This means that the design longitudinal traction & braking forces must be individually applied to each deck span throughout the viaduct. Main resistance is offered by the pier carrying the fixed bearings of that particular span, with generally a small additional resistance from friction generated at the free bearings carrying that span, located over the next pier. This means that a substructure of this type with, say, 10 equal height piers has a total resistance capacity of 10 times the deck design traction & braking longitudinal loads, a capacity unfortunately not available because of the simply supported articulation.

This large extra resistance capacity can be realised by placing the new bridge STUs across the joints between the simply supported spans, either at deck or bearing level.

Current integrity assessments of a number of these multi simply supported span viaducts often indicate that the piers are understrength due to increases in the traction & braking loading since original design, often accompanied by damage generated by limited road salt, carbonation or ASR. STUs placed across the joints will immediately mobilise load sharing between piers, usually sufficient to reduce pier loading to a level not requiring strengthening.

Even with existing continuous deck spans, which automatically produce similar load sharing action at the piers without resort to STUs, further beneficial load sharing can be gained by placing STUs at the expansion joint ends of the continuous span sequence.

4.2 Strengthening Viaducts on the London Docklands Light Railway

The newly completed viaducts carrying London's Docklands Light Railway, Figure 6, were designed for a train service which, due to a breathtaking increase in adjacent development, will now require considerable expansion. This will mean heavier & more frequent trains, which will add braking & traction effects in excess of those originally catered for.

Figure 7 shows a typical seven span deck unit, continuous between expansion joints. Train traction and braking loads are currently shared among the slender piers, which generally support the deck via rubber bearings. It is proposed to install STUs, Figure 8, at rail level between joints such that, when the new increased longitudinal traction & braking loading is applied to one particular seven span unit, load is beneficially transmitted and shared with adjacent seven span decks sufficient to require no pier and foundation strengthening.

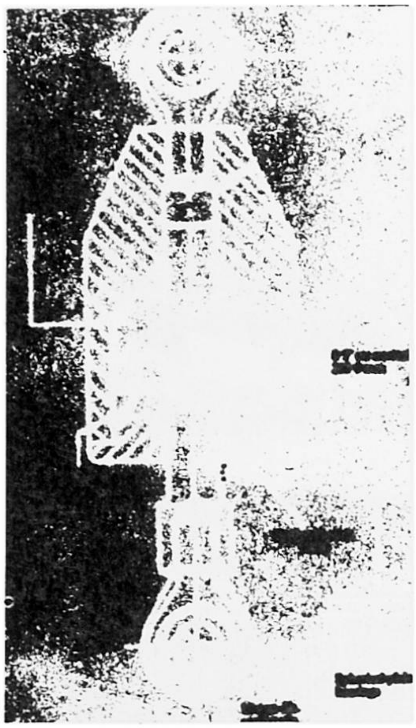


Fig. 1 Arrangement of STU

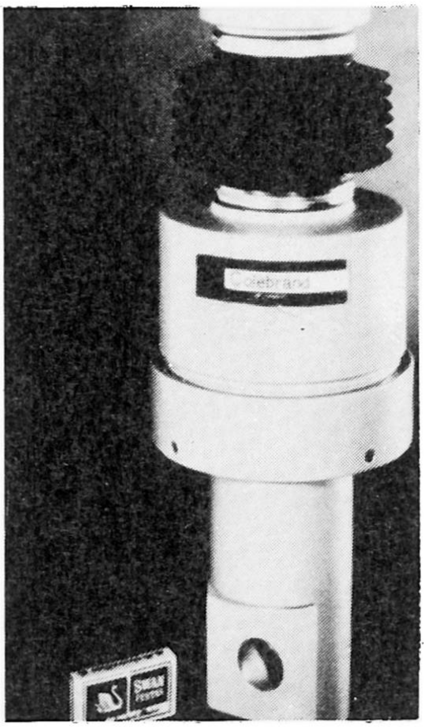


Fig. 2 50 Tonne STU

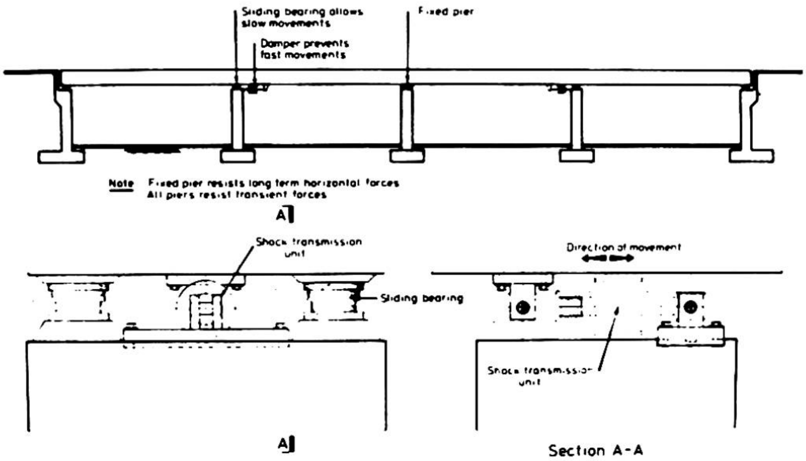


Fig. 3 Flyover Articulation with STUs

Fig. 4 Kuwait Flyover

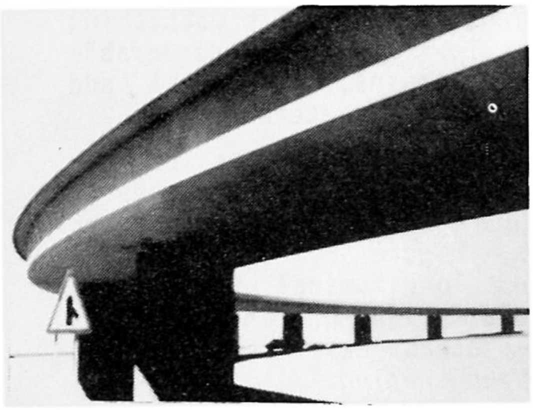


Fig. 5 Pier STUs

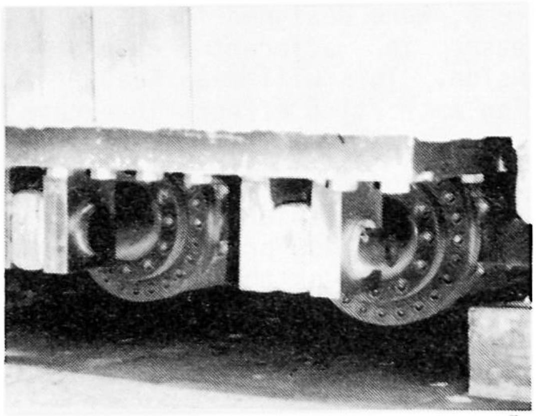


Fig. 6 Docklands Light Railway (DLR)

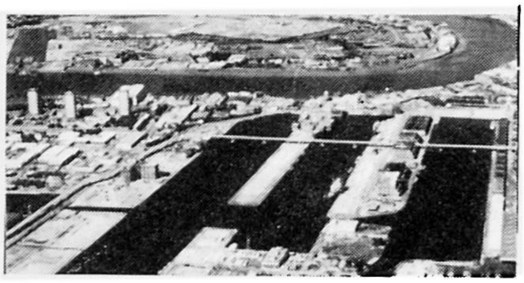
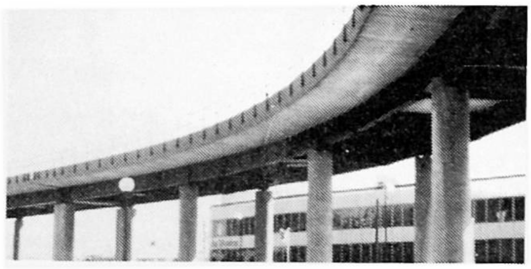


Fig. 7 DLR Viaduct



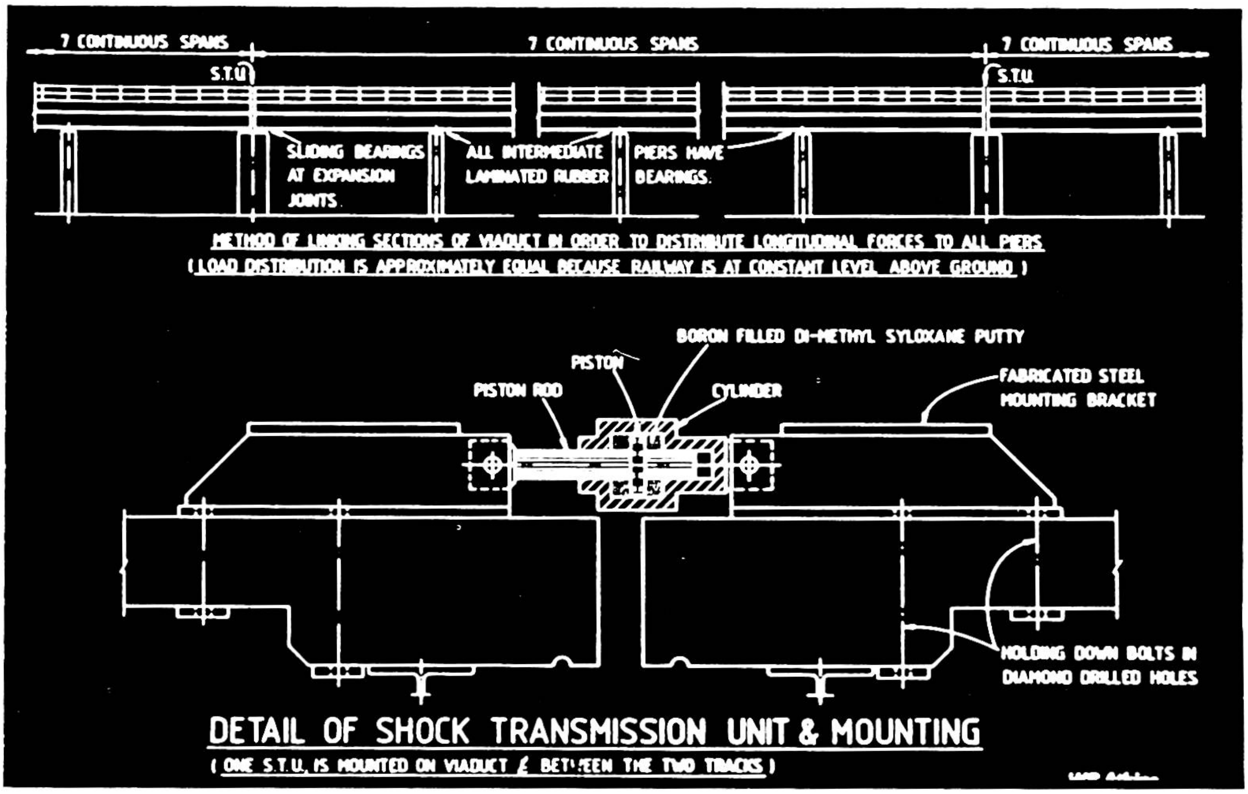


Fig. 8 Docklands STUs

Protection and Maintenance of Concrete Bridges

Protection et entretien des ponts en béton

Schutz und Instandhaltung der Betonbrücken

Erkki VESIKARI
MScTech
Techn. Res. Centre of Finland
Helsinki, Finland

Erkki Vesikari, born 1949, received his MScTech degree in the Civil Engineering Department of Helsinki University of Technology. For more than ten years he has been involved in durability problems of concrete bridges as a research scientist in the Technical Research Centre of Finland.

SUMMARY
The article deals with protection of concrete bridges against corrosion and deterioration. Protection is understood as being a series of successive measures starting from design and ending in repair. The in-service maintenance measures like coating and crack filling are specially treated. Results from tests for protective coatings and injection resins carried out in Finland are included.

RÉSUMÉ
L'article présente la protection des ponts en béton contre la corrosion et la destruction. Ici, la protection exprime toute la série des mesures prises depuis la conception jusqu' à la réparation. On a spécialement étudié la protection des surfaces de constructions en béton et le plombage des fissures du béton. Certains résultats ont été obtenus en Finlande après des recherches sur la protection et les injections.

ZUSAMMENFASSUNG
Dieser Artikel behandelt die Schutzmassnahmen bei Betonbrücken gegen Korrosion und Verwitterung. Die Schutzmassnahmen bedeuten hier eine Serie von nacheinander folgenden Massnahmen von der Projektierung bis zu den Reparaturen. Besonders werden Reparaturmassnahmen wie der Schutz von Betonaussenflächen mit Beschichtungen und die Füllung der Risse mit Injektionsmitteln behandelt. Einige Ergebnisse der in Finnland durchgeführten Versuche, bei welchen die obengenannten Reparaturmethoden zur Anwendung kamen, werden erläutert.

1. PROTECTION MEASURES IN GENERAL

In the general meaning protection of bridges is understood to cover all the measures taken to prevent deterioration and to extend service life of bridges. Accordingly, the protection of bridges starts from design.

Parallel to structural design durability design is carried out. In the case of bridges an automatic adaptation of solutions in codes and standards may not be enough for durability design. In addition to observing rules a careful evaluation of different optimal solutions with special attention to the probable service life is needed. The decisions made in the design stage pertaining to the quality of concrete, concrete cover, draining systems of bridge deck, quality of water membrane etc. are most significant in regard to the prevention of deterioration of bridges.

The purpose of quality control is to insure that quality requirements for materials and structures will be met. Some control measures must be taken at the time fo concrete cast. In Finland, for example, where salt-frost scaling of concrete is one of the most severe problems in bridges, it is essential to insure by tests before and during construction that the required air content of concrete is attained. The poor frost resistance of concrete due to lack of air in concrete is difficult to be replaced by other protective measures. Another critical point is the concrete cover of reinforcement. The supervision of the construction site must check before casting of concrete that the required concrete cover is provided for reinforcement to prevent too early corrosion.

Design	Durability design
Construction	Quality assurance
Service	Inspection Maintenance Repair

Fig. 1 General protection scheme of bridges.

The inspection and maintenance of bridges belong to the protection of bridges and are an essential part of it. The purpose of the inspection activity is to follow the condition of bridges so that extensive deterioration can be prevented by right time maintenance. The parts of bridges which normally are invisible such as submerged structures and bridge decks under pavement should be taken under control by special inspections.

Maintenance and repair follow inspections when defects or damage are discovered. By maintenance we understand here preventive measures such as sealing concrete with protective coatings or filling cracks.

2. PROTECTION WITH COATINGS

2.1 General remarks on coating

Although coating of bridges has not been very extensively used in Finland it is obvious that in many cases the service life of bridges can be extended with special protective coatings which retard the chloride penetration and carbonation processes in concrete. Also by setting a barrier to moisture ingress the frost-salt deterioration can be diminished.

The performance of a protective coating depends on its adhesion. It is essential that the concrete surfaces to be coated are throughly clean and sound. The moisture content of the concrete at the time of coating is also important. Most coatings require a dry substrate.

In the following description protective coatings are classified into three groups: surface sealers, polymer coatings and cement based coatings.

2.2 Surface sealers

Surface sealers partly penetrate into the capillary pores of concrete and leave a thin water repellent film on concrete surface. They do not significantly change the appearance of concrete. Surface sealers produce an effective barrier to water from outside but they are relatively permeable to water vapour from inside of concrete. By the chemical composition surface sealers are usually silanes, siloxanes or silicone resins.

Due to the water repellent property of surface sealers they can be used to prevent frost damage in concrete. They can also be used as a barrier against chlorides from outside and as a protective agent against staining. However, the rate of carbonation cannot be reduced by surface sealers.

Some of the surface sealers (especially silanes) are more effective in preventing the moisture ingress than thicker polymer coatings. It is due to their ability to impregnate all cavities and voids on the concrete surface as opposed to normal polymer coatings which leave pin holes. In Fig. 2 typical water absorption and desorption curves for specimens (100 mm cubes) with different coatings are presented. Very little moisture is absorbed into the silane impregnated specimen compared to the other coated specimens. The good vapour-permeability of silane can also be noted.

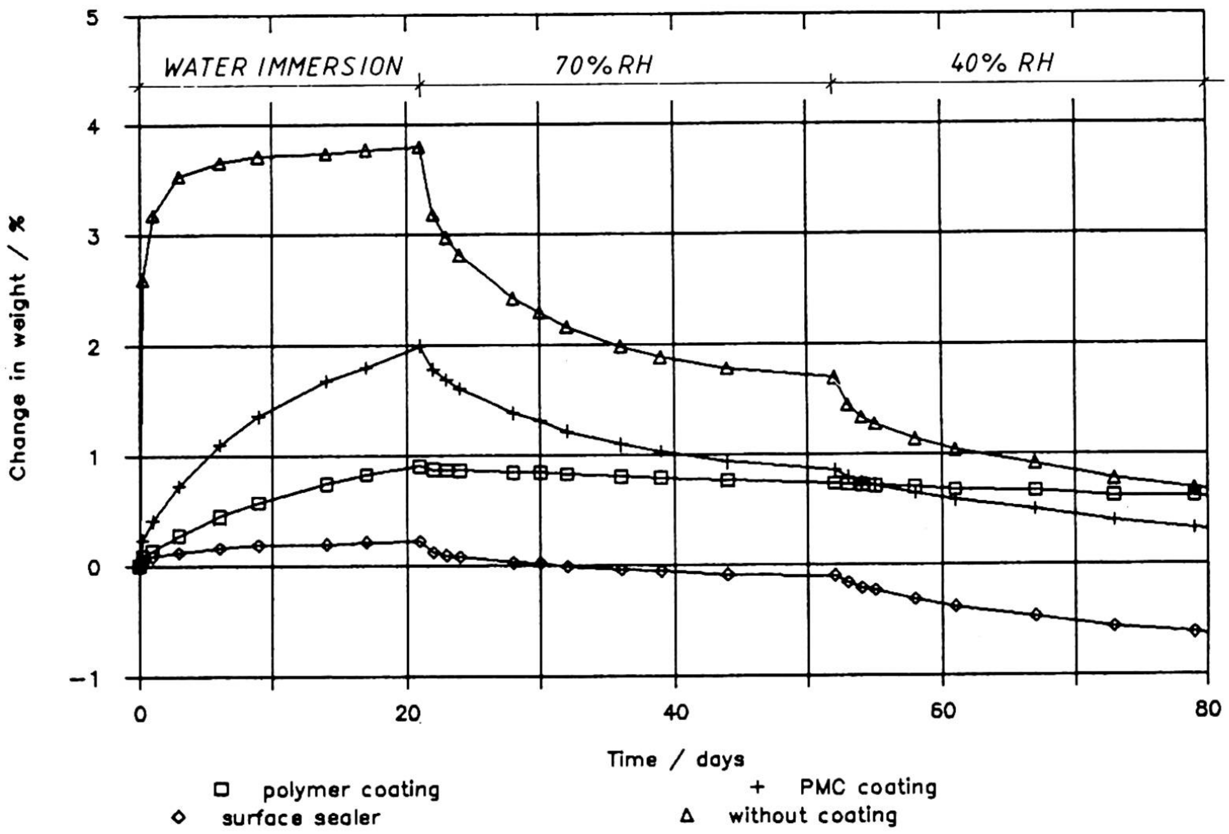


Fig. 2 Absorption and desorption of coated 100 mm cubes.

2.3 Polymer coatings

By the chemical composition polymer coatings are usually epoxy resins, polyurethanes, acrylics, polyester resins or copolymers [1]. They consist of one or two components being reactive, water based or solvent based. For better appearance polymer coatings are often pigmented.

Polymer coatings can be compiled of several treatments. Thin coatings classified in the range of (0.15 - 1 mm) produce a rather effective barrier to carbonation. However, they do not normally seal the concrete completely because of the pin holes left in the coating. Water from outside can easily penetrate into concrete through pin holes but drying is slow because of the relatively vapour-impermeable coating. That is why thin polymer coatings are susceptible to loosen from the substrate when not sheltered from rain.

Thick polymer coatings (1 - 3 mm) are normally completely water-impermeable. Most of them can also bear heavy mechanical loading. Accordingly they can be used as a water membrane on a bridge deck without protective concrete. Some modifications of epoxy and polyurethane-based thick coating have elastic properties being deformable enough to bridge living cracks.

2.4 Cement-based coatings

The binding agent of cement based coatings usually consists of cement such as portland cement and polymer (polymer modified cement). The polymer may be styrene butadiene rubber, acrylic or modified acrylic or other polymer latex.

Cement based coatings of normal thickness (< 2 mm) are spread by brush or spray. Mortars of the thickness (2...5 mm) can also be trowelled. Carbonation or penetration of chlorides cannot be effectively retarded by cement based coatings but their resistance to freeze-thaw cycles may be rather good.

In Table 1 results of the tests on polymer and cement based coatings carried out in Finland are presented. The evaluation of each coating is made using the scale (++, +, -, --) with respect to every test. The variation in compositions and properties of different coatings can be noted.

Polymer coatings		Barrier against chlorides	Barrier against carbonation	Resistance to freeze-thaw cycles Type of concrete surface			
No.	Specification			non-carb no chlorides	non-carb chlorides	carbonated no chlorides	carbonated chlorides
	thin coatings						
1	2 c primer + 1 c copolymer	++	++	++	++	++	+
2	2 c primer + 2 c polyurethane	++	++	++	--	++	--
3	2 c epoxy	+	+	++	++	++	-
4	2 c epoxy	+	+	++	+	++	-
5	2 c epoxy (water based)	-	-	++	++	++	++
6	silane + 1 c acrylpolymer	++	-	++	++	++	-
7	2 c epoxy (water based)	-	+	++	++	++	+
8	2 c epoxy	++	+	++	++	++	--
9	1 c primer + 1 c copolymer	--	-	+	--	--	--
10	1 c polyester (elastic)	-	+	--	--	++	--
11	2 c primer + 2 c epoxy (water based)	++	+	++	++	++	++
12	primer + acrylpolyester	+	+	--	--	++	--
13	siloxane + 1 c acrylpolymer	++	-	+	++	+	--
	thick coatings						
14	2 c epoxy bitumen	+	++	++	++	++	++

(continued)

Cement based coatings No.	Specification	Barrier against chlorides	Barrier against carbonation	Resistance to freeze-thaw cycles Type of concrete surface non-carb no chlorides	non-carb chlorides	carbonated no chlorides	carbonated chlorides
	normal coatings						
15	PMC-coating + acryl resin	+	+	+	-	-	-
16	styreneacrylic + PMC-coating	--	-	++	++	--	--
	mortars						
17	PMC mortar (2 c copolymer)	--	--	++	++	++	++
18	cement based mortar	--	-	++	++	++	++

Table 1 Summary of results of tests for coated specimens (++ very good, + good, - not good, -- poor).

3. CRACK FILLING

Concrete structures are designed in such a way that controlled cracking may occur in some circumstances. However, unfavourable cracks also appear causing a risk to the durability of reinforcement or structural integrity. Such cracks must be repaired as soon as possible.

In the Finnish recommendations for bridge structures all cracks exceeding the width of 0.3 mm should be filled by resin. In the filling of structural cracks injection techniques are used. Small surface cracks can be also filled by manual gravity feeding.

In the manual feeding of small cracks latex polymers may be advantageous because of their good capillary suction properties. However, complete filling of the crack is not expected when manual feeding is used.

Filling a crack by injection involves introducing low viscosity resin into the crack by due pressure and holding it there while it sets to a non-flowing state. A number of different sealing methods are used (putties, sealing tapes etc.). In case it is impossible to seal the outlet of the crack thixotropic epoxy resins can be used. Thixotropic resins do not easily flow out of an open crack.

One of the most important properties of injection resins is the bond to concrete. In Fig. 3 the results of a slant shear test carried out in Finland are presented. The bond at the slant joint is evaluated by crushing the prism in a compression testing machine and recording the failure load. The results show the variation in bond strengths on dry and moist concrete surfaces.

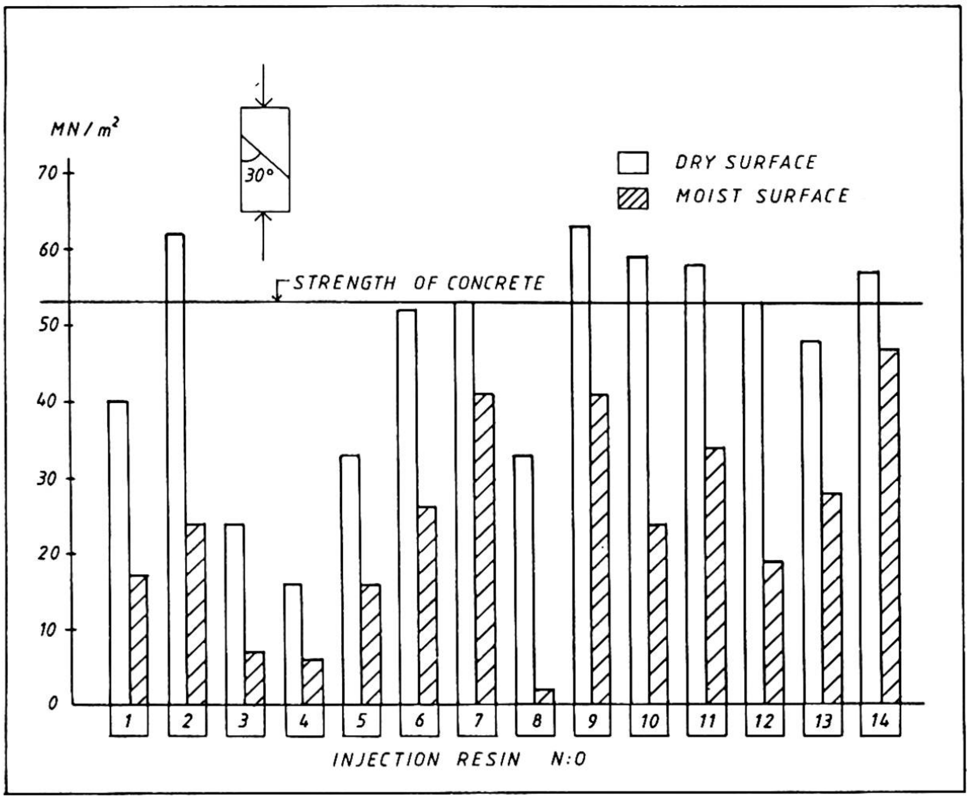


Fig. 3 Test results of bond (slant shear) test for 14 injection resins.

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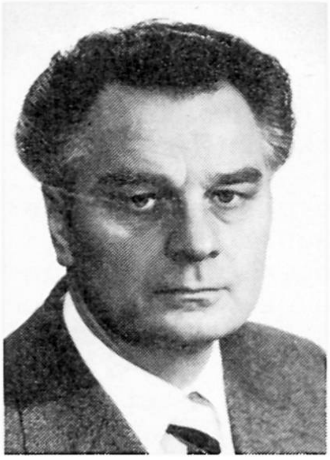
Experimentelle Erprobung von Stahlbetonbauwerken in situ

Tests on Reinforced Structures in situ

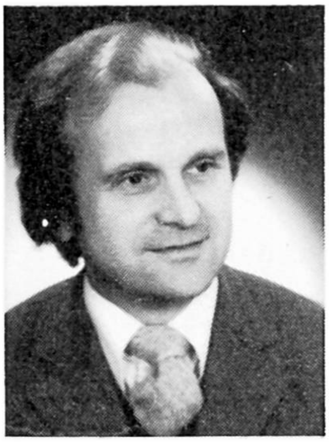
Expérimentation in situ des ouvrages en béton armé

Horst SCHMIDT
o.Prof. Dr.-Ing.
Technische Universität
Dresden, DDR



Horst Schmidt, geboren 1927, promovierte 1960 an der Technischen Universität Dresden. Er war als leitender Ingenieur im Industriebau an der Errichtung vieler Vorhaben und speziell an der Entwicklung des Stahl- und Spannbetonfertigteilbaues beteiligt. Seit 1976 ist er ordentlicher Professor für Festigkeitslehre und experimentelle Baumechanik.

Heinz OPITZ
Dr.-Ing.
Technische Universität
Dresden, DDR



Heinz Opitz, geboren 1937, promovierte 1970 an der Technischen Universität Dresden. Er arbeitete vorwiegend in Lehre, Forschung und Projektierung auf den Gebieten des Stahlbetonbaues, des Versuchswesens und der Baumechanik. Er ist Oberassistent am Lehrstuhl Festigkeitslehre und experimentelle Baumechanik.

ZUSAMMENFASSUNG
Eine neue Vorschrift der DDR über den Nachweis der Trag- und Nutzungsfähigkeit bestehender Bauwerke und Bauwerksteile aufgrund experimenteller Erprobungen wird auszugsweise vorgestellt. An Beispielen werden Erfahrungen bei der Prüfung bestehender Stahlbetonbauwerke mitgeteilt.

SUMMARY
A new GDR standard on the indication of ultimate bearing capacity and serviceability of existing buildings and parts of structures based on experiments is presented by extracts. It is illustrated by experience gained in tests on existing reinforced structures.

RÉSUMÉ
On présente des extraits d'un nouveau règlement établi en R.D.A. portant sur la charge admissible et l'utilisation possible de constructions existantes et de leurs composants à partir d'essais. Des exemples illustrent les expériences faites lors des essais d'ouvrages en béton armé.

1. EINE NEUE VORSCHRIFT ZUR EXPERIMENTELLEN ERPROBUNG BESTEHENDER STAHL- UND SPANNBETONBAUWERKE

In Sonderfällen kann es zweckmäßig sein, das Trag- und Verformungsverhalten bestehender Konstruktionen aus Stahl- oder Spannbeton durch eine experimentelle Erprobung in situ zu bestimmen, insbesondere wenn kein geeignetes Berechnungsmodell vorliegt, das die tatsächlich vorhandenen Verhältnisse ausreichend genau beschreibt. Dadurch mögliche Reserven der Trag- und Nutzungsfähigkeit, die durch eine experimentelle Erprobung aufgezeigt werden können, liegen z. B. in folgenden Wirkungen, die oft theoretisch nicht berücksichtigt werden, begründet:
- Gewölbewirkung von Balken und Platten durch Verdrehungsbehinderung an den Auflagern
- Nicht erfaßte räumliche Tragwirkungen
- Einbeziehung der wirklich vorhandenen Baustoffkennwerte im Bauwerk
- Rechnerisch nur näherungsweise zu erfassende Lastverteilungen oder Lastumlagerungen
- Mitwirkung von Fußbodenaufbauten an der Tragwirkung.

Experimentelle Erprobungen können auch dann sinnvoll und zweckmäßig sein, wenn durch Brand, Korrosion oder andere Schädigungen Trag- und Nutzungsfähigkeit nicht mehr genügend genau eingeschätzt werden können, wenn Zweifel an einer fachgerechten Ausführung des Bauwerkes oder Bauwerksteiles aufgetreten sind oder wenn veraltete Bauweisen zu beurteilen sind, über deren Wirkungsweise und Beschaffenheit keine genauen Aussagen mehr getroffen werden können. Weil in solchen Fällen das Ergebnis der experimentellen Erprobung (auch als Probebelastung bezeichnet /1/) als Ergänzung oder als Ersatz für theoretische Berechnungen gelten muß, wurde hierfür in der DDR eine entsprechende Vorschrift /2/ erlassen. Gegenüber früheren Festlegungen über Probebelastungen z. B. in /3/ wird der Nachweis aufgrund experimenteller Erprobung in /2/ nach der Methode der Grenzzustände und in einer neuen Form geführt. Grundsätzlich werden zwei Probebelastungsarten unterschieden:
- Probebelastungsart A ist dadurch charakterisiert, daß die Belastung nach einer festgelegten Be- und Entlastungsfolge bis zu einer Größe, bei der noch keine Schädigungen auftreten, die die Brauchbarkeit des Bauwerkes oder Bauwerksteiles im künftigen Nutzungszeitraum beeinträchtigen, erfolgt.
- Probebelastungsart B ist dadurch gekennzeichnet, daß die Belastung einzelner im Regelfall vom übrigen Bauwerk getrennter Bauwerksteile einer gleichartigen Bauteilgruppe bis zur Grenzbelastung der Trag- und Nutzungsfähigkeit erfolgt, wobei keine Beeinträchtigung des übrigen Bauwerkes eintreten darf.

Die folgenden Ausführungen behandeln vorwiegend nur die Probebelastungsart A.

2. NACHWEISFÜHRUNG DER TRAG- UND NUTZUNGSFÄHIGKEIT AUFGRUND EXPERIMENTELLER ERPROBUNGEN IN SITU

Für den Grenzzustand der Tragfähigkeit eines Bauwerkes oder Bauwerksteiles wird bei Probebelastungsart A in /2/ folgende Nachweisgleichung vorgeschrieben:

$$\sum_{i=1}^{i=1} \mathrm{adm}\ F_{u,i} \cdot n_i \cdot k \cdot h \leq \mathrm{obs}\ F_u \prod m_j \qquad (1)$$

Dabei bedeuten

obs F_u die bei der experimentellen Erprobung ermittelte Grenzbelastung, ohne während der Prüfung wirkende ständige Lasten.
Werden Lasten bei der Prüfung getrennt erfaßt, dann gilt mit obs $F_{u,i}$ dem i-ten Anteil von obs F_u

$$\text{obs } F_u = \sum_{i=1}^{i=1} \text{obs } F_{u,i}$$

adm $F_{u,i}$ experimentell ermittelter, obs $F_{u,i}$ entsprechender i-ter Anteil der Normlastkombination für den Nutzungszeitraum ohne während der Prüfung wirkende ständige Lasten

n_i Lastfaktoren, k Lastkombinationsfaktor und

h Wertigkeitsfaktor des Bauwerkes nach /4/

$\prod m_j$ Produkt der Anpassungsfaktoren, die im Nutzungszeitraum auftretende Einflüsse, die bei der Prüfung nicht erfaßt werden, berücksichtigen.

In /2/ werden Anpassungsfaktoren m_j für Langzeitwirkung auf die Festigkeit, für den Abbau der Gewölbetragwirkung durch Schwinden und Kriechen bei seitlicher Verschiebungsbehinderung an den Auflagern von Balken, Platten und Gewölben und für die Vergrößerung der Außermittigkeit druckbeanspruchter Bauwerksteile infolge Kriechen angegeben.Zusätzlich sind gegebenenfalls andere Einflüsse aus zeitabhängigen Formänderungen infolge von Schwinden und Kriechen auf Zwangs- und Vorspannkräfte sowie aus Temperatur-und Umweltbedingungen, die sich gegenüber der Prüfung ungünstig auf die Tragfähigkeit im Nutzungszeitraum auswirken können, zu berücksichtigen. Die aus der experimentellen Erprobung abzuleitenden Normlasten für den weiteren Nutzungszeitraum sind aus Gleichung (1) oder explizit nach Gl. (2) zu bestimmen

$$\text{adm } F_{u,i} \leq \frac{\text{obs } F_{u,i} \prod m_j}{n_{tot}} \tag{2}$$

mit dem mittleren Lastfaktor

$$n_{tot} = \frac{\sum_{i=1}^{i=1} F_i \, n_i \, k \, h}{\sum_{i=1}^{i=1} F_i} \tag{3}$$

F_i = i-ter Anteil der Normlast,

wobei die während der Prüfung wirkenden ständigen Lasten nicht zu berücksichtigen sind.

Eine entscheidende Problematik bei der Probebelastung liegt in der Festlegung der Grenzlast obs F_u, bei der voraussetzungsgemäß noch keine solchen Schädigungen auftreten dürfen,die die Brauchbarkeit des Bauwerkes oder Bauwerksteiles im künftigen Nutzungszeitraum beeinträchtigen.In /2/ wird festgelegt, daß die Prüfung abgebrochen werden muß, wenn einer der folgenden Zustände eintritt:

- die gemessenen Betonstauchungen $\varepsilon_b \geqq \varepsilon_{b,lim} - \varepsilon_0$, die gemessenen Dehnungen des Betonstahles $\varepsilon_s \geqq 0{,}9 \left(\frac{R_s^0}{E_s} - \varepsilon_{s0}\right)$ oder des Spannstahles $\varepsilon_s \geqq 0{,}9 \left(\frac{R_p^0}{E_p} - \varepsilon_{s0}\right)$ werden

mit

$\varepsilon_{b,lim}$	Grenzwerte der Betonstauchungen nach Tab. 1
ε_0, ε_{s0}	rechnerisch ermittelte elastische Betonstauchung bzw. Dehnung des Beton- oder Spannstahles infolge Vorlast, während der Prüfung wirkender ständiger Lasten der Bauwerksteile und Versuchsaufbauten sowie infolge von Vorspannkräften zum Zeitpunkt der Prüfung
R_s^0, R_p^0	Grundwerte der Rechenfestigkeiten des Beton- oder Spannstahles

Tabelle 1 Grenzwerte der Betonstauchungen $\varepsilon_{b,lim}$ in ‰

Betonklasse	Biegung	Querkraft Torsion	Längsdruck-kraft
$\leqq$ Bk 20	1,0	0,7	0,6
> Bk 20	1,0	0,9	0,8

- bei schlaffbewehrten Bauwerksteilen Biegerisse und/oder Schrägrisse sich über die gesamte Querschnittshöhe auszubreiten beginnen, oder wenn Rißbreiten $w \geqq 0{,}5$ mm auftreten,
- bei Spannbetonkonstruktionen mit Vorspanngrad I oder II Rißbreiten $w \geqq 0{,}1$ mm auftreten,
- bei Spannbetonkonstruktionen mit Vorspanngrad III Rißbreiten $w \geqq 0{,}35$ mm auftreten,
- ein Schubdruck- oder Schubzugbruch aufgrund der Verformungsmeßwerte der Bügel, Aufbiegungen oder des Betons und aufgrund des Rißbildes zu erwarten sind,
- Anzeichen für den Beginn eines Verankerungsbruches erkennbar sind,
- Stabilitätsversagen zu erwarten ist,
- Meßwerte, die insgesamt in ihrer Entwicklung während der Prüfung zu verfolgen sind, bei weiterer Laststeigerung auf plastische Formänderungen schließen lassen, die zu einer Schädigung des Bauwerksteiles führen.

3. BEISPIELE VON DURCHGEFÜHRTEN EXPERIMENTELLEN ERPROBUNGEN

Die Durchführung von Probebelastungen erfordert Erfahrungen und Sorgfalt, eine einfach regelbare dem konkreten Fall angepaßte Belastungstechnik und eine ausreichend genaue, zuverlässige und oft robuste Meßtechnik.

Als Belastungsmöglichkeiten bei in situ auszuführenden Probebelastungen kommen in Frage:

a) Einleitung von hydraulisch oder pneumatisch erzeugten Kräften (vgl. z. B. /1/)
b) Aufbringen von Masseteilen
c) Belastung durch Wasser in Folienwannen /5/

Zu der unter b) genannten Möglichkeit wurden von den Autoren bei Probebelastungen verschiedene Varianten angewendet. So wurden bei der Prüfung korrosionsgeschädigter Stahlbetonkranbahnträger einer Freikrananlage die Lasten mittels am Kranhaken angehängter Betonblöcke, die auf einem mit Hydraulikhebern unterstützten Podest auflagen, durch stufenweises Anheben und Absenken eingetragen. Die Ablesung der Kräfte erfolgte über ein zwischengeschaltetes Dynamometer (Fig.1). Während aller Belastungs- und Entlastungsstufen wurden Durchbiegungen an verschiedenen Punkten, Stahldehnungen an freigelegten Bewehrungsstählen, Betonstauchungen an der Balkenoberseite und Stahldehnungen der Kranbahnschiene gemessen. Risse wurden auch bei maximaler Last obs F_u nicht beobachtet.

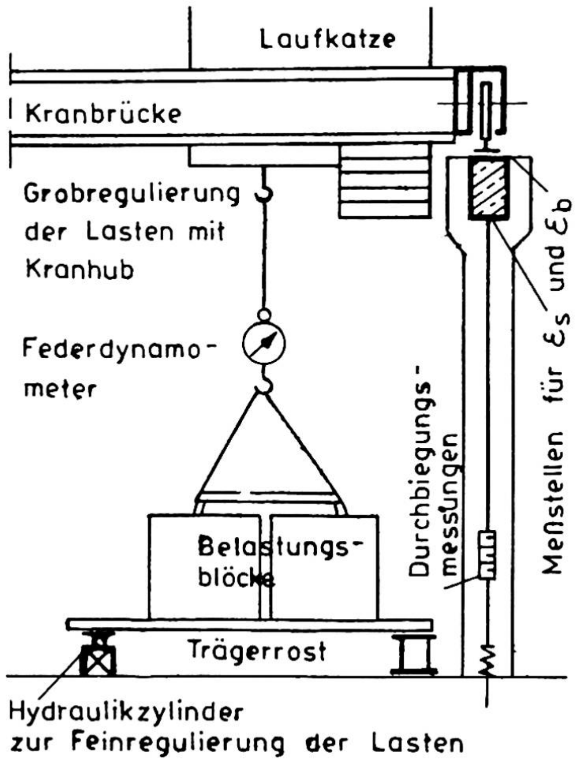


Fig.1 Prüfung eines Stahlbeton-Kranbahnträgers

Im Ergebnis einer nach /2/ erfolgten Auswertung und einer unverzüglich eingeleiteten Sanierung der korrodierten Bewehrung konnte die Freikranbahn für den Betrieb mit 20 t Nutzlast erhalten bleiben.
Bei einem anderen Beispiel sollte experimentell die Tragfähigkeit 24 m weitgespannter über 50 Jahre alter Hallenbinder, die große und teilweise über 1 mm breite Risse aufwiesen und aufgrund veränderter Technologien jeweils eine Zwischenstütze erhielten, geprüft werden. Über Gehänge wurden mit Gabelstaplern vorher genau abgewogene Betonplatten aufgebracht (Fig.2). Die Schädigungen der Binder erwiesen sich als unbedenklich, wie die Messungen mit einem Rißweitenmesser auf induktiver Basis und die Durchbiegungs- und Krümmungsmessungen zeigten. Mit /2/ konnten die Binder für die weitere Nutzung nach Verschluß der großen Risse freigegeben werden.
Die Nachrechnung von auf kräftigen Stahlbetonrahmen aufliegenden kassettenförmigen Dekkenplatten in einem mehrgeschossigen Industriebau ergab keine ausreichende Sicherheit

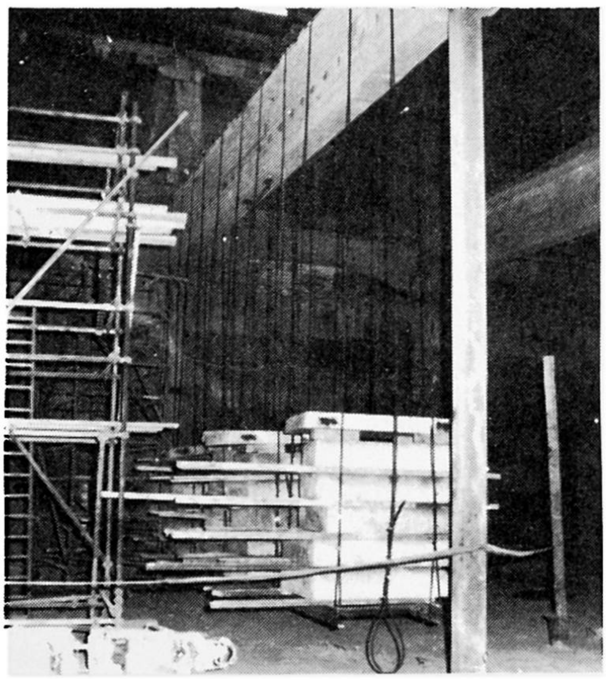


Fig.2 Prüfung eines Stahlbeton-Hallenbinders mit 24 m-Spannweite

für Hubwagen-oder Gabelstaplerbetrieb. Durch Aufbringen von handlichen Stahlstücken konnte über ein Belastungsgestell kurzfristig eine Prüfung der Kassettenplatten mit hohen Einzellasten erfolgen, wobei die genaue Größe der Kraft über elektrische Druckmeßdosen registriert wurde (Fig.3). Durch Gewölbewirkung und Mitwirkung der Estrichschicht wurden große Reserven der Tragfähigkeit gegenüber der Berechnung festgestellt, so daß das künftige Befahren dieser Decke mit Gabelstaplern erlaubt werden konnte.

Fig. 3 Belastung einer Kassettendeckenplatte in einem mehrgeschossigen Industriebau durch Einzellasten

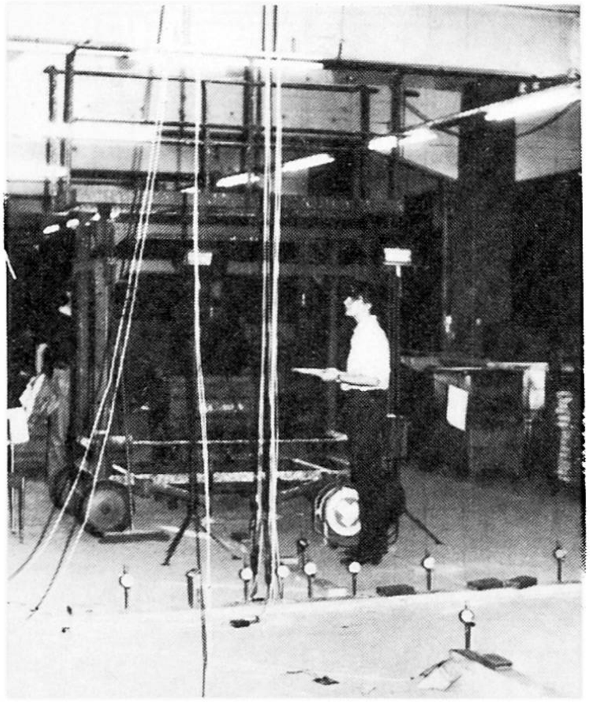


Fig.4 Meßanordnung bei der experimentellen Erprobung der Kassettendeckenplatten

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