PRESENTATIONS

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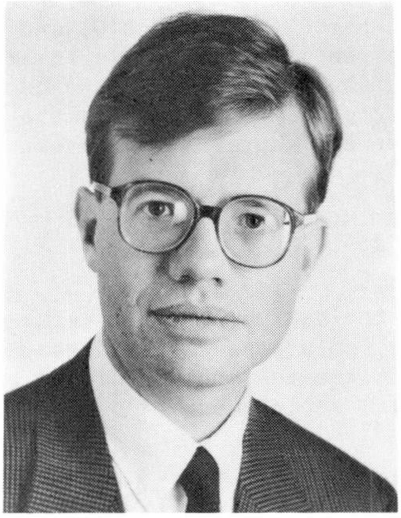
Durability Research as an Investment Strategy

Recherches sur la durabilité pour une stratégie d'investissement

Dauerhaftigkeitsforschung als eine Investitionsstrategie

Jan BRÖCHNER
Assoc. Professor
The Royal Inst. of Technology
Stockholm, Sweden



Jan Bröchner, born 1948, received his doctorate from the Royal Institute of Technology in 1978. His present research interests concern the economics of construction and management of existing structures.

SUMMARY

If the optimality of a structure as well as of its programme of maintenance and renewal is measured by total present value, any addition to our knowledge of the durability of materials and components should be measured by its contribution to this value. It follows that the economic consequences of research projects on durability can be studied with a strategic perspective.

RÉSUMÉ

Si l'optimum économique de la conception d'un ouvrage ainsi que de son programme d'entretien et de renouvellement est mesuré à sa valeur totale actualisée, tout progrès de nos connaissances concernant la durabilité des matériaux et composants doit être évalué en fonction de sa contribution à cette valeur. Par conséquent, les effets économiques de projets de recherche sur la durabilité sont étudiés dans cette perspective de stratégie.

ZUSAMMENFASSUNG

Wenn die Qualität eines Bauwerks sowohl am Bauwerk selbst als auch am zugehörigen Programm für Instandhaltung und Erneuerung als Gegenwartswert beurteilt wird, so ist jede Vermehrung unserer Kenntnisse bezüglich der Dauerhaftigkeit von Baustoffen und Komponenten nach ihrem Beitrag zu diesem Wert zu beurteilen. Auf dieser Grundlage werden die wirtschaftlichen Folgen von Forschungsprojekten über Dauerhaftigkeit mit einer strategischen Perspektive untersucht.

1. INTRODUCTION

1.1 Profit maximization

The choice of materials and components as well as maintenance and replacement policies over time can be formulated as a profit-maximization problem for the owner of a structure or group of similar structures. Let $Q(t)$ be the quality of the structure at time t and assume that the stream of revenues R from the use of the structure is a function of quality, $R = R[Q(t)]$. The stream of costs $C(t)$ is initiated by construction costs $C(0)$ and after $t = 0$ caused by maintenance and replacement expenditure. The time horizon for the maximization is $t = T$, which may or may not be the date of total demolition of the structure. Finally, the rate of discount which expresses the time preferences of the owner is taken to be ϱ, and we can express the present discounted value as follows:

$$\int_0^T \{R[Q(t)] - C(t)\} e^{-\varrho t} dt$$

Over the chosen time period, the maximum present value is to be achieved. By application of control theory, solutions can be found, giving not only the optimal maintenance paths but also the optimal timing of discontinuous increases in $Q(t)$, i.e. partial replacement [1].

1.2 Revenues

For most structures, the stream of revenues is not immediately known. Even when the structure or the services of the structure appear on the rental market or is subject to user fees, the relation between R and $Q(t)$ may be difficult to ascertain. This is especially true of the influence on revenues of small changes in functional quality, although the development of a range of quantitative indicators for the functional condition of road surfaces shows what is feasible for large systems of infrastructure under a unified management [2].

However, many of the structures lacking an easily identifiable stream of revenues are managed on the principle of approximately constant quality over time. The optimization is then equivalent to a minimum life-cycle cost approach [3].

1.3 Costs

Costs of maintenance and replacement may appear easier to predict than the effect of quality changes on revenues. An exception is when there is a possibility of cumulative damages occurring on failure of a structural part, as with reinforced concrete [4]. Risk analysis based on a mapping of failure mechanisms, their wider consequences as well as maintenance and replacement options, should then be performed, provided that the analysis in itself gives a positive net contribution to the expected present value.

1.4 Data and level of application

In current practice, direct optimization is usually ruled out because of lack of data, discontinuities and complex patterns of interaction. Depending on the type of structure, obstacles to data access arise from fragmented and competitive ownership or high costs of collecting data for a unique structure subject to a unique environment. To some extent, advances in information technology will improve the availability of data. Regarding discontinuities and complexities, the theory as it is can only be applied at a suitable level of aggregation: applied to a system of several structures rather than the individual one, so that behaviour over time and resource flows into maintenance can be approximated by continuous functions.

2. THE CONCEPT OF DURABILITY

2.1 Durability and service life

The related concepts of durability and service life are properly of interest only for the special case of materials and components that either perform their function or fail to do so. However, as concepts they provide a convenient focus for studies into the long-term behaviour of materials and components, not least for the large group of structures where there is a technology for recurrent maintenance as well as partial replacement, prolonging their lives and postponing total replacement. In practice, we encounter a long scale from partial replacement to total renewal.

Viewed under the appropriate magnification, any maintenance activity including cleaning or repainting falls into one or several of the categories of removal, replacement or addition of materials (or components). By disaggregating, a level can always be found where service life is a meaningful concept. Unfortunately, there are few cases where this level allows direct optimization. Nevertheless, research into durability of materials and structures is important because it focuses on $Q(t)$, or in other words, performance over time, in the form of degradation studies.

2.2 Degradation

Degradation of materials and components under the influence of various degradation factors [5] is the main object of durability studies. A variety of methods is available to the researcher, field studies, accelerated aging and the parallel development of theory, as in the case of concrete structures [6]. Degradation in an existing structure gives rise to maintenance and replacement needs.

3. MAINTENANCE AND INSPECTION

3.1 The quality path over time

The choice of maintenance policies is simplified in the constant quality case, although the determination of an optimal period of renewal will depend on the maintenance cost function. We should avoid looking at maintenance as an operation which in principle just restores the $Q(0)$ of the structure; in many cases, there is a technologically inseparable combination of activities intended to raise the load-bearing capacity or other qualities above the initial level. Conversely, it may be profitable to reduce the original quality level and remain at a lower level. Thus, from the economic point of view, the $Q(t) = Q(0)$ case is fortuituous or, if imposed as a restriction on the management of the structure, sometimes more due to difficulties in information handling than to functional analysis of the use of the structure.

Formulating the optimal maintenance path problem with the help of the theory of optimal control usually implies that the present value maximization is supplemented with a quality change restriction, where maintenance enters as a control variable:

$$\dot{Q}(t) = m(t) - \delta(t)Q(t)$$

where $m(t)$ is a measure of maintenance resources consumed at time t and $\delta(t)$ is the degradation or depreciation function. If $m(t) = 0$ and $\delta(t) =$ constant, the quality of the structure decays exponentially, as in Fig. 1. The two degradation paths based on δ_1 and δ_2 show the possible effect of durability research: either an improvement in the material so as to attain the δ_1 level or a narrowing of the uncertain area between δ_1 and δ_2, reducing the risk in choices during the design stage. In Fig. 2, the effect of durability studies on materials or components with a pattern of sudden failure is shown: either a prolongation of service life from t_2 to t_1 or a more narrow interval of life being predicted.

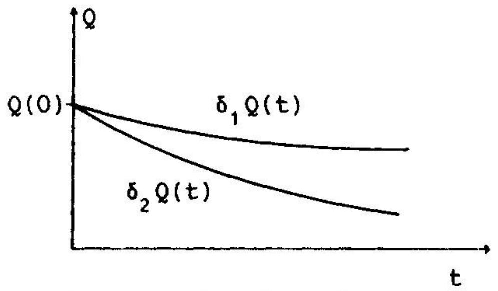


Fig.1 Successive degradation

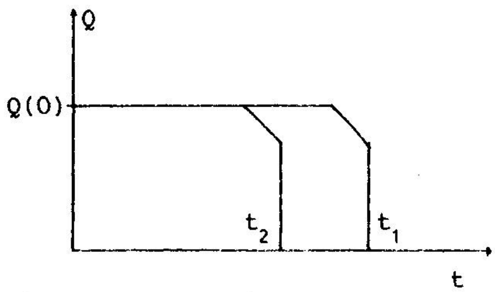


Fig. 2 Sudden failure

3.2 Preventing damages

Preventive maintenance may raise the present value if there is significant irreversibility in processes which start when quality declines below a certain level. Structural collapse is a typical example, roof leakage a less dramatic one. Optimal strategies for preventive maintenance can be condition-based, including routine inspection, or time-based (periodical), which should be the choice when the cost of inspection exceeds the expected increase in total present value associated with the structure.

The development of durability studies increases our ability to recognize and anticipate irreversible quality changes in a structure. Such knowledge can be translated into alternative patterns of inspection or even as the continuous monitoring of decay through the information network of an 'intelligent' building. Inspection, the choice of intervals, the resources devoted to it and consequential savings, should be seen as a part of the maintenance strategy and as a part of a joint optimization problem for the assignment of scarce resources [7].

4. EVALUATING DURABILITY RESEARCH

4.1 The economic value of research findings

The findings of durability research can be used in three contexts:

(a) in the materials industry, when developing new materials or combinations of materials,

(b) at the design stage, leading to better choices of materials and components;

(c) after completion of the structure, when choosing or modifying strategies of inspection, maintenance and replacement (partial or total).

For a given type of research projects, the relevant set of structures with reasonably similar technology and similar environmental conditions must be identified first, both existing structures and projects in the foreseeable future. Secondly, the situations of economically important choices in the three contexts (a - c) should be analysed. The analysis comprises an assessment of the range of substitute materials, both at present and over the lifetime of the structure, alternative methods of maintenance and the risks involved in deferred maintenance and replacement, at the present state of knowledge and as expected from the project in question. After that, present value analysis should be considered. As an example, the design choice between two paints A and B with different degradation properties, following as most surface materials do the pattern of Fig. 1, can be based on the ensuing maintenance costs [8]. The calculation is more complicated when assessing the value of risk reduction (in terms of Fig. 1, that the true degradation pattern is δ_1 and not δ_2, e.g.), and in the situation of Fig. 2 with sudden failure, the emphasis of the analysis is shifted towards a probabilistic approach, identifying risks and their relations. Finally, the potential for spin-off effects owing to new, more fundamental insights into decay mechanisms with a wider application should be assessed, although this is mostly a matter of intuitive judgment.

4.2 Recipients of benefits

Like most forms of research with a fragmented pattern of beneficiaries, it is difficult to finance activities by user fees according to benefits. Where there are single users of the information, such as materials producers with local monopolies in a uniform environment, or the structures - like many bridges - form part of a technical or legal monopoly, something approaching an efficient allocation of research resources should appear spontaneously. However, if materials and structures are sold in markets, where durability information is costly to transfer from seller to buyer, there will be insufficient incentives to improve the long-term performance through research. A textbook solution, when inspection and quality control costs are high in market transactions and there is a considerable risk of latent defects, is vertical integration through joint ownership of the stages of production. Since this is seldom the case for buildings and the physical infrastructure, cooperation and government support is needed to reach an efficient level of durability research.

The case of the monopoly producer of a material holds another complication. If the extended life of products reduces the total materials consumption per annum, and there are economies of scale in production, some of the returns to scale will be lost when sales decline [9]. On the other hand, the superior durability may lead to a wider market for the product, which compensates for the immediate loss of volume.

4.3 Quality of research

Especially where component failure may lead to severe damages, questionable validity and reliability of research findings introduces an important stochastic component. Also, it is often so that durability findings are loosely applied to a slightly different material subject to slightly different environmental action. Care should be taken that research methods and presentation of findings minimize additional risks in application. Expert evaluations of research projects is one of several ways to monitor the level of quality in durability studies [10].

5. AN INVESTMENT STRATEGY

Returning to the initial present value maximization, we can look at durability studies in the same perspective. The body of knowledge in the field of durability is treated analoguously with the structure, and the successive maintenance inputs have their parallel in additional research findings. Due to new discoveries and the general development of technology, parts of the existing knowledge are rendered useless, similar to the successive deterioration of Fig. 1. The revenues from durability research have to be derived from the savings made in materials production and in the management of erected structures.

In practice, the total optimization implied by this approach has to be replaced with a strategy of suboptimization: a set of priorities for investment in durability research should be established working backwards, primarily identifying alternative methods of maintenance and replacement, analysing how great the economic losses can be when durability characteristics of materials and components are known imperfectly.

6. CONCLUSION

Within the framework of present value analysis, it is necessary to consider both the long-term revenues and the costs associated with structures. A flexible view of maintenance and programs of partial replacement can then be applied and the value of durability research assessed in this context. For the specific case of sudden failures with major irreversible consequences, risk analysis should be used according to priorities based on an identified potential to contribute to overall present value. The lack of incentives for research is traced partly to the costs of knowledge transfer between sellers and buyers when markets for materials and structures exist. A tendency to underinvest in durability studies should be met by research cooperation and strong government funding.

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Role of Codes in Bridge Durability

Rôle des codes pour assurer la durabilité des ponts

Die Einführung von Normen zur Verbesserung der Dauerhaftigkeit von Brücken

Roger A. DORTON
Manager, Structural Office
Ministry of Transportation
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 MTO in 1972. He is chairman of the Ontario Highway Bridge Design Code Committee, and a member of various AASHTO, TRB, ACI and CSCE Committees.

SUMMARY

If durable bridges are to be produced, reliability and serviceability must be addressed in new design codes. Probabilistic methods enable this to be done by calibrating serviceability conditions to agreed levels. Rehabilitation codes are lacking in reliable data for satisfactory calibration. Tendering methods affect durability levels. Four alternative tendering methods are reviewed. The build/operate/transfer method appears likely to produce the most durable structures.

RÉSUMÉ

Afin d'augmenter la durabilité des ponts, il faut envisager de nouveaux codes de projet. Ce but peut être atteint par des méthodes de probabilité, en calibrant les conditions de service à un niveau acceptable. Les codes de réfection manquent de données valables pour leur calibrage satisfaisant. Le système des soumissions a un effet sur les niveaux de durabilité des ponts. On considère quatre types de soumissions; celle qui semble devoir donner les meilleurs résultats quant à la durabilité des structures, combine la construction, l'opération et le transfert final au propriétaire.

ZUSAMMENFASSUNG

Um bei dem Bau von Brücken einen hohen Grad an Dauerhaftigkeit zu erreichen, müssen in bezug auf Zuverlässigkeit und Instandhaltungsmethoden neue Normen geschaffen werden. Bestimmte Prüfmethoden erlauben es, gewisse Normen für die Brückeninstandhaltung nach bestimmten Richtlinien festzulegen. Reparaturvorschriften geben keine genauen Auskünfte über zuverlässige Instandhaltungsmethoden. Die üblichen Ausschreibungsmethoden beeinträchtigen die Dauerhaftigkeit. Vier verschiedene Methoden werden besprochen. Die Bauen/Betrieb/Übertragungsmethode scheint den höchsten Grad an Dauerhaftigkeit und Zuverlässigkeit beim Brückenbau zu erreichen.

1. INTRODUCTION

Durability is described as the ability of the structure to maintain its level of reliability and serviceability during its lifetime. In the past, durability has been considered mostly in terms of serviceability items, such as cracking and spalling of concrete, corrosion of steel, and limiting maintenance costs. These items were considered as primarily the preserve of construction specifications, site inspection and quality control, rather than that of design codes. Such codes might specify minimum concrete cover, for instance, and other items of good construction practice, but could not adequately address the question of reliability as these older working stress codes were deterministic rather than probability based.

With the introduction of probabilistic limit states bridge codes around 1980, lifetime reliability has become one of the most significant areas for code development. If the required statistical data is available, load and resistance factors can be calibrated to achieve target safety indices at various serviceability limit states as well as the ultimate limit state. Durability has thus become very much an area of interest for design codes.

The 20 year boom in new highway and bridge construction peaked in North America and a number of European countries in the late 1960's. As many of these bridges are now ageing and require extensive maintenance, there has been an increased interest in bridge rehabilitation in the last ten years. This interest initially focussed on repair techniques and materials rather than rehabilitation design, as codes did not address this issue. If rehabilitation is to be cost effective, much work needs to be done on putting rehabilitation design in the same reliability based context as new designs. Much more data is needed on the life expectancy of rehabilitated bridges before this can be completed, but the third edition of the Ontario Highway Bridge Design Code (OHBDC) will have a new section on rehabilitation design when issued late in 1989.

Bridge codes have generally been written to cover frequently built bridge types in the short and medium span range. For long span bridges special design criteria are usually prepared which may not address durability adequately, particularly if probabilistic data is not available which may be the case for a unique design. It is in the long span bridge range that different tendering methods are likely to be used, such as alternative designs, design/build proposals, and more recently build/operate/transfer (BOT). These various methods can produce in themselves wide variations in durability. The latter method, BOT, holds promise for a high level of reliability and serviceability, however, and a current Canadian example is presented.

2. DURABILITY ASPECTS IN DESIGN CODES

Whatever basic code philosophy is used, all bridge design codes should precribe details to ensure ease of maintenance, and specify design details that are considered good practice and likely to produce durable structures. The OHBDC 1983 explicitly addresses maintenance and durability aspects, and some of its provisions will be identified as typifying what can be covered for bridges in a corrosive environment, regularly subject to winter salting.

The components most susceptible to deterioration have been concrete bridge decks, expansion joints and bearings[1]. The minimum slab depth is specified as 225 mm with a minimum cover to the top reinforcing steel of 50 mm. Placing tolerances for reinforcement are given which have to be allowed for in setting the dimensions on the drawings to ensure that the minimum cover is achieved in the field. The OHBDC commentary references the use of epoxy coated reinforcement and membrane waterproofing for decks. Deck drainage and drip detail requirements are given, with downspouts to protrude below soffit level to keep salt water off the superstructure.

Sealed deck expansion joints are normally used, but as they frequently leak, the use of continuous spans to minimize joints and thus improve durability is encouraged. Seals have to be replaceable, and must be set below the riding surface to reduce wear. If the joints eventually leak, access is needed between the abutment ballast wall and the deck for cleaning, and a minimum gap of 200 mm is called for to enable this to be done. Bridge seats must have a grade of at least 5% so water will drain away from bearings. Bearings have to be accessible for inspection and maintenance and be replaceable without damage to the structure and without removing anchorages permanently attached to the structure.

To facilitate inspection and maintenance of steel or concrete box girders, and enable interior formwork to be removed, access openings have to be provided for each cell, and have closely fitting hinged covers. Such girders shall not contain sewers or water pipes inside them due to the possibility of breakage or leaking, and subsequent girder deterioration or danger of collapse. Gas and oil pipelines are prohibited from all highway bridges on account of fire and explosion hazards.

All these items may appear rudimentary, yet they have to be specified at the design stage, and need to be included in the design code to ensure implementation, otherwise the durability of the built structure may be compromised.

3. RELIABILITY BASED CODES

The move in recent times towards reliability based codes, enables codes to be calibrated to produce relatively consistent safety levels for bridges. This calibration work has generally concentrated on the ultimate limit states [2], but can equally well be applied to serviceability limit states, thereby providing another means of establishing durability levels for structures at the design stage. For the 2nd Edition of the OHBDC in 1983, such calibration was carried out for the serviceability limit states of cracking, vibration, fatigue and permanent deformation [3], as well as ultimate limit states.

The design equation for each specified limit state is:

$$\phi\, R \geq \text{total factored load effect}$$

where ϕ is a resistance factor, R is the nominal resistance, and total factored load effect is the sum of the product of the nominal loads considered multiplied by their corresponding load factor. The calibration process used was the calculation of load and resistance factors, using second moment level-2 reliability analysis [4], to obtain a reliability index close to the preselected target value. The reliability index β is a measure of safety, such that:

$$\beta = \frac{\bar{R} - \bar{Q}}{\sqrt{\sigma_R^2 + \sigma_Q^2}}$$

where $\bar{R}$ and σ_R = mean resistance and its standard deviation and $\bar{Q}$ and σ_Q = mean load effect and its standard deviation.

The target β selected for ultimate limit states was 3.5. The serviceability limit states can be reached more frequently and lower β values can thus be selected. For example, for cracking of prestressed bridges, a target value of $\beta = 1.0$ was used, which relates to concrete cracking under live load once a week. This frequency of crack opening was considered acceptable, considering the possible fatigue of the prestressing strands and the possible corrosion of strands due to the entry of aggresive salt water. From the durability point of view it should be noted that Ontario practice calls for a full waterproof membrane and asphalt wearing surface over prestressed decks in addition to the serviceability controls on the concrete.

The target reliability values can be selected according to the type of structure and its importance. For instance, for elevated transit structures, where service must be maintained at all times, and no alternative routes are available, higher target β values of 4.0 and 2.5 for ultimate and cracking limit states have been proposed [4].

4. LIFETIME SERVICEABILITY

Based on the history of bridge replacements in North America, a typical design life expectancy would be 50 years. Bridges have become deficient due to functional or geometric inadequacies, serious structural deterioration, or insufficient load carrying capacity due to an increase in vehicle weights. Most bridges over 30 years old have required significant rehabilitation work. With calibrated limit states design codes and a better understanding of the design, construction and operational needs for more durable bridges it is expected that new bridges will show better performance. It is unlikely, however, that any bridge will achieve its 50 year lifetime without some rehabilitation work being needed. If a bridge is to maintain the designed level of reliability and serviceability during its lifetime it is desirable that the code to which it is designed also includes provisions for load capacity evaluation and rehabilitation.

Evaluation and rehabilitation aspects have traditionally not been part of design codes. The first two editions of the OHBDC have covered bridge evaluation, and the third edition will have a new section on rehabilitation. The limit states format is ideally suited to evaluation and rehabilitation, as the actual bridge can be surveyed and the design values of load and resistance factors modified as appropriate. These factors can also be adjusted to suit the anticipated future life of the structure, which is unlikely to be as long as the 50 year life on which the design values were based. By using these methods the load carrying capacity will usually calculate higher than that obtained by applying new design provisions to the evluation process.

When rehabilitation design is required, the new bridge design provisions are not usually suitable, and rehabilitation code sections should again reflect changed loading conditions, structure condition and anticipated future life. The OHBDC will have three rehabilitation categories according to an anticipated future life of greater than 25 years, 10 to 25 years, and up to 10 years. Each category will have its own prescribed load factors. It will be difficult to do a comprehensive calibration of the rehabilitation load and resistance factors at this time, as life expectancy of various rehabilitation techniques is hard to establish. When enough rehabilitation data has been collected, rehabilitation design can be put on the same probabilistic basis as new structures. A code can then consistently address new design, evaluation and rehabilitation to increase the probability of maintaining a uniform level of serviceability throughout the life of a bridge.

5. TENDERING METHODS AND DURABILITY LEVELS

5.1 Background

Most methods of tendering for bridge construction give little incentive to produce the high quality work that will enhance lifetime durability. They usually award to the bidder with the lowest construction cost.

In North America this price is prepared using full design drawings and specifications prepared by the owner or his consultants. Standard design codes are usually applied. On some major projects alternative tendering methods have been used, usually on long span bridges which are beyond the range for which standard design codes apply. Special design provisions have to be prepared, including those addressing durability. The different methods of tendering can have a major impact on the likelihood of obtaining durable bridges, and four methods will be compared from this perspective.

5.2 Single Design Provided by Owner

With this method all contractors bid on the same design, with full drawings and specifications issued, and no provision for changes, except perhaps by applying value engineering after the contract is awarded. The design requirements and construction specifications need to be comprehensive and must be supported by a major quality assurance program by the owner, as the contractor, in order to obtain the job, has to bid providing no more than the minimum specified quality.

The method works reasonably well for large public authorities when building short and medium span bridges on a regular basis. Most highway departments in North America use this method for their bridges, unless the contract value is high enough to warrant going to alternative designs.

5.3 Alternative Designs Provided by Owner

On a large bridge project when it is difficult to select the single most economical design, the owner may provide two or more designs for competitive bidding. The controls needed for durability noted for the single design method are equally necessary under the alternative design method, as the award is still based on the lowest bid, with full documents provided by the owner. There are two further considerations, however, for the alternative design approach. As this method is usually applied to long span bridges, beyond the typical maximum span of about 150 m for which standard design codes apply, special design criteria may need to be prepared covering durability aspects of long spans and possibly unusual types of bridges. The other consideration comes at the design stage, and relates to preparing designs of equal durability, and as far as possible equal maintenance costs over the lifetime of the bridge. As the award is made on lowest construction cost, rather than lowest life cycle cost, the process makes sense only if each alternative is equally acceptable to the owner, which implies equal reliability and serviceability.

5.4 Design/Build with Designs Provided by Contractor

This is the method commonly used in Europe, but has been used only recently in North America. The contractor prepares his own design and bids on this, using overall requirements supplied by the owner. The method is usually applied to large projects or long span bridges, and particular attention again must be paid to design criteria preparation. This may be even more important than under the alternative design approach, as the owner does not know what types of bridge will be designed by the bidders. The specially prepared design criteria thus have to cover a very broad range of structure types and materials to ensure comparable durability. As the award is again based on lowest construction cost, the concerns about equal serviceability and maintenance costs given under the previous method are equally applicable under the design/build approach, but are probably harder to achieve.

The design/build method was used for the retractable roof stadium "Skydome" in Toronto due to open this year. A two phased procedure was followed with approval of technical concept as a first stage producing four finalists. The competing roof types being now known, the final design criteria were prepared before moving up to the second stage of pricing. This stadium roof, with barrel arches spanning up to 200 m is fully exposed to the elements when retracted, and is in many ways more similar to a steel bridge than a building. One interesting aspect of the design criteria, addressing reliability and durability, was the requirement that the roof structure had to stand up after all members within a vertical cylinder of diameter 4.5 m located anywhere in the roof were removed. This requirement produced a highly redundant winning design with a multiplicity of load paths, probably the best way of ensuring a high level of reliability.

5.5. Build/Operate/Transfer (BOT)

The Northumberland Strait Project, a 13 km long bridge linking the Provinces of New Brunswick and Prince Edward Island is the first application of the BOT method in Canada. Developers are required to finance, design and build the toll facility, operate it for 35 years, then transfer it to the Federal Government. The competition has three phases, initial qualification of developers, acceptance of concept proposals, and finally the price proposals. The project is presently held up for a year, following the completion of the second phase, pending a further environmental study.

Design criteria were written by the Federal Government before the second phase, but left many tasks to the developer, such as calibrating the criteria for the ultimate limit states to a target β value of 4.0, carrying out wind tunnel tests, and performing ship collision risk analysis. To the extent possible durability requirements were given for steel and concrete bridges, with the objective of using the best possible techniques. The eventual owner prescribed a 100 year life, in the aggressive environment of an ocean crossing with high winds, wave action, and large ice forces. To increase the chances of achieving this lifetime, and of having a bridge in good condition at the transfer stage, the use of salt as a deicer on the roadway is prohibited. The developer, as the operator, has to use other methods such as urea or CMA.

An important aspect of the BOT method is that the developer is just as interested in durability as the owner. In fact the developer may aim for durability in excess of the prescribed minimum in order to avoid a major rehabilitation cost before the transfer date. The proposals are in effect based on a total 35 year lifetime cost to the developer, as the project will not be awarded on construction cost, but on the basis of the lowest government annual subsidy requirement.

6. CONCLUDING REMARKS

Techniques are now being developed so that design codes for new bridges can properly address lifetime durability. More information on the expected life of repaired components is required before similar progress can be made on rehabilitation design codes.

The level of durability may vary with the contract tendering method adopted. The BOT method on major bridge projects holds the most promise for improved durability as it is the only method that makes long term durability a common goal for both owner and constructor.

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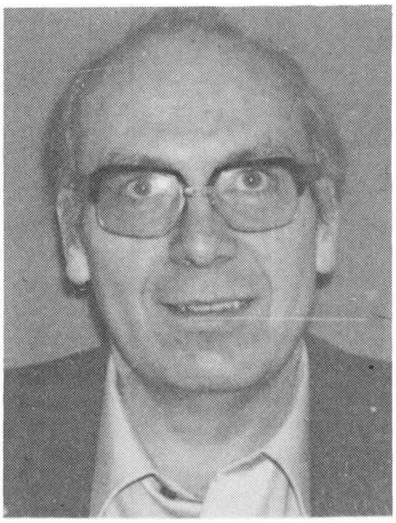
Durability, Defects and Repair of North Sea Structures

Durabilité, détérioration et réparation de plateformes en Mer du Nord

Dauerhaftigkeit, Schäden und Reparaturen von Tragwerken in der Nordsee

Svein FJELD
Vice President
Norwegian Contractors A/S
Stabekk, Norway



Svein Fjeld obtained his civil engineering degree in 1961, and Dr. Eng. degree in 1968, both from the Technical University of Norway. For twelve years he was employed by Det norske Veritas, dealing with control and certification of offshore structures. He is now employed by Norwegian Contractors dealing with project developments.

SUMMARY

The paper gives a brief summary of the findings from the in-service inspections on North Sea offshore structures. Needs and methods for maintenance and repair are outlined. The causes of the observed deficiencies are discussed. Main issues for the steel structures are accidental impact and other unforeseen overloading. Other significant findings concern cracks and the cathodic protection systems. The concrete structures have performed well. A few localized structural deficiencies required attention. On some older platforms the mechanical systems gave some concern.

RÉSUMÉ

L'article résume les observations faites lors d'inspections de plateformes en service en Mer du Nord. Les exigences et les méthodes d'entretien et de réparation sont résumées et les causes des problèmes observés sont discutées. Les éléments saillants concernant les structures métalliques sont les chocs accidentels et autres conditions de surcharges non prévues. D'autres remarques d'importance s'adressent à la fissuration et aux systèmes de protection cathodique. Les structures en béton ont un bon comportement, sauf pour quelques défauts localisés qui exigent un soin particulier. Certaines plateformes plus anciennes ont connu des problèmes de systèmes mécaniques.

ZUSAMMENFASSUNG

Der Aufsatz gibt eine kurze Zusammenfassung der Ergebnisse der Betriebs-Inspektionen von Offshore-Tragwerken in der Nordsee. Bedarf und Methoden für Wartung und Reparatur werden beschrieben. Die Gründe für die gefundenen Mängel werden diskutiert. Die wichtigsten Probleme für Stahltragwerke sind Stossbelastungen und andere unvorhergesehene Überlastungen. Andere wichtige Erkenntnisse betreffen Risse und das kathodische Korrosionsschutzsystem. Die Betontragwerke erfüllen ihre Funktion gut. Wenige lokale Mängel der tragenden Bauteile erfordern Aufmerksamkeit. Bei einigen älteren Anlagen bestehen Probleme bei mechanischen Komponenten.

1. OFFSHORE STRUCTURES IN THE NORTH SEA

1.1 Number and types of structures

Since the first steel structures were installed in the shallow Southern North Sea in the sixties more than 100 steel jackets and altogether 20 concrete structures have been installed. The water depth record is held by the 216 m deep Gullfaks C platform installed the summer 1989.

Two main types of fixed structures are considered.

1) Steel jackets i.e. space frames built up of slender tubular members with piled foundations.

2) Concrete gravity base structures composed of a voluminous caisson base with 2-4 large diameter towers projecting up throught the water surface and supporting the deck. Other platforms consist of one large central shaft surrounded by a perforated wall.

This paper covers all 20 concrete structures, and about 30 steel platforms including all on the Norwegian Shelf. Smaller structures such as loading buoys and floating units are not covered.

1.2 Environmental exposure

The environmental exposure in the North Sea climate is characterized by large static and dynamic wind and wave loads, fatigue exposure and corrosivity. Typical wave heights with one month recurrance periode is in order of 20 m whereas the design wave (100 years recurrance periode) is about 30 m.

1.3 Materials and workmanship

Low carbon structural steels with yeld stress typically 360 MPa are specified for the steel structures. All essential welds are 100% X-ray tested. The concrete structures have concrete qualities in the range from C45 to C60.

1.4 In service inspection

The oil companies as well as the Governmental Authorities require all significant points of the platforms to be inspected on a yearly or four-yearly basis. In addition special surveys are made whenever found appropriate e.g. after possible damage. In service inspection of both steel and concrete structures are based on visual inspection. Steel structures are also subject to NDT inspection i.e. MPI, ultrasonics and potential measurement of the corrosion protection. The reliability and detectability of small deficiencies by the NDT methods under water has been questioned.

2. STEEL STRUCTURES

2.1 Cracks in welds

Cracks have been observed on a majority of the platforms investigated. The maximum length was 750 mm. In platforms where the cracks cannot be related to improper design or overloading, cracks were observed in 6-7% of the welds investigated. About half of these cracks were removed by grinding to a depth of 2-3 mm. Occationally much deeper grinding has been carried out.

Particular investigations have been made in dented areas and at burn marks. The crack frequency was much higher in these areas, but a majority of them was removed by grinding.

Cracks not removed by grinding have been kept under observation to see possible propagation. Cracks caused by loads inadequately designed for only, were found to propagate as expected for a fatigue crack. There are, however, cases where cracks turned up again after apparently successfull grinding removal.

Several crack cases required comprehensive repair work i.e. by welding, bracing removal, construction strengthening etc.

2.2 Dents and deformations

Dents were observed on the majority of steel platforms. In most cases they are accompanied with deflections of the member in question. Dent observations are evenly distributed over the heigth of the platforms.

As mentioned cracks are frequent in the dented zones. The cold working associated with the denting can result in surface brittleness. The irregular distorted geometry gives stress peaks which promote fatigue. There are cases where bracings have broken as the result of these mechanisms.

Where repairs were found necessary, the damaged members have been cut out and new sections have been introduced by welding and clamping. Dented struts have had their capacity in compression reestablished by filling the tubular member with grout.

Local strengthening has been accomplished by welding on doubler plates or by mechanical clamping fixed by bolts. Clamps grouted around the weakened structure is another popular repair method. In some cases stabilizing braces have been welded on to the deflected member to avoid further deflections. In other cases a paralell bracing have been welded to unload the damaged member.

The accomplishment of these operations under water, or even more complicated, in the splash zone is a difficult and expensive undertaking.

2.3 Corrosion

General corrosion is no major concern. Only a few platforms are found to be seriously attacked, and in limited areas only.

Pitting corrosion, on the other hand, is observed on the vast majority of platforms. Pitting 5 mm deep is not unusual. The extent and seriousness of the pitting vary significantly between platforms and between nodes. No particular elevation seems more exposed than others.

Most of the pitting seems initiated shortly after the platform installation, maybe before the necessary cathodic polarisation is established.

The cathodic protection systems seems rather vulnerable. Lost or loose anodes or other anode deficiencies have been observed on more than half the platforms.

Approximately half the platforms were also found to have too high electrochemical potential (i.e. more positive than -0.85 V to a silver electrode). On about one third of the platforms more anodes have had to be added after the installation. Metallic debris in electric contact with the platform increase the anode consumption. This kind of debris have had to be removed from all the platforms.

2.4 Sea floor erosion

Platforms installed on an erodable sea floor are normally designed assuming 10 ft seabed erosion. The platforms dealt with in this paper are mainly in water depths 70 m and more. Only one of these platforms experienced erosion exceeding the design assumption.

2.5 Marine growth

A majority of the platforms experienced a significant marine growth. The first platforms installed were not designed to resist the added hydrodynamic loads associated with the fouling and cleaning has been necessary. The design criteria introduced for later platforms seems to cover the growth observed in practice.

2.6 Conclusions, steel structures

A large portion of the significant structural findings concerned impacts from ships and dropped objects. Similar type of impact and overloading occurred during installation.

Another important reason for repair is static overload. In some cases the strengthening was necessary to cope with increased loads beyond the original design basis. In other cases the cause might be classified as design errors. Some more robustness to accidental impacts and other unforeseen loads might have been desirable.

Damages classified by "cracks" and "workmanship" cover a variety of different causes. A majority of these cases should probably have been avoided by better site supervision and control.

3. CONCRETE STRUCTURES

3.1 Leakages

Leakage through porous concrete, cold joints or encased pipes was only observed on some of the platforms constructed in the early seventies. Apart from the three cases mentioned below signs of leakage has only been observed by deposits and occasionally a humid surface. This always was a temporary problem and after a few months the seepage selfhealed. It is a general observation that cracks and porosities heal and seal by deposits ($CA\ CO_3$ and $Mg\ (OH)_2$) produced by reactions between the concrete and the sea water.

In one case the ingress of water was as large as $5m^3/h$. The ballast pumps could easily cope with this leakage rate. Without further precautions the leakage was observed to decrease with time and after three years the inner surface was completely dry.

Two minor leakages were sealed by injection with epoxy. A minor leakage through a burst plastic pipe was repaired by grout injection.

3.2 Foundation cracking

Cracks were observed in a cantilever wall supporting part of a foundation slab, presumebly caused by uneven soil reactions. The cracks were injected by epoxy and the surrounding sea floor covered by scour protection.

3.3 Precast formwork spalling

Some of the early platforms were fitted with a perforated breakwater wall. For convenient construction a precast concrete formwork was applied. On one platform this formwork spalled off in a limited area. The cover to the reinforcement was reduced more than formally acceptable. Repairs were done by replacing unsound concrete by cement mortar and covering the area with epoxy.

3.4 Impact damage

Five cases of significant ship impact have been reported.

None of them resulted in damage reducing the structural strength. Four impacts resulted in minor scratches in the concrete surface only, and no repairs were necessary.

One of the impacts caused through-cracking. A leakage of 5 litres/min was observed. This damage concerned one of the first platforms in the UK-waters. It was not designed to resist ship impact.

To avoid possible rebar corrosion the area was repaired by:

- epoxy injection of the cracks and cavities
- grouting removed unsound concrete
- epoxy coating internal and external surfaces.

Eight dropped object cases have justified subsequent structural investigations. Six investigations concluded no repair to be required. These concerned pipes of diameter up to 30".

A 36" spool piece of 10.5 t weight was dropped onto the caisson roof and punched through. The structure was found to be structurally sound and operation could continue without further measures. As the punched caisson was used as oil storage a repair was undertaken by grouting cement mortar into a prefabricated form filled with gravel.

A 36" drain caison of 20 t weight was dropped onto the caisson roof. The only consequence was some spalling of the concrete cover. Epoxy was injected under a formwork to restore the rebar cover formally required.

3.5 Seafloor erosion

One case of seafloor erosion has been observed. This concerned two voluminous concrete structures installed close to each other in relatively shallow water. The situation was stabilized by filling the eroded area with sand bags.

3.6 Conclusions, concrete structures

It is remarkable how well the concrete structures in themselves have performed in the hostile North Sea climate. No signs of material deterioration, rebar corrosion or other deficiencies have been observed.

A common feature of the findings on concrete structures is their local nature. The structural integrity of the platforms has never been jeopardized. Typical cases have been impacts from ships and dropped objects resulting in local cracking of the concrete.

Concrete platforms normally have comprensive mechanical systems for ballasting, water outlet, oil storage etc. On some of the early platforms these systems were not adequately designed. Material weakness and inadequate tightening of pipe penetrations have lead to a few leakages.

A general conclusion to be drawn is that the concrete structures required significantly lower expenditures for inspection, maintenance and repair than the steel platforms. The extent of the inspection tends to be reduced over the years as no findings are made under normal operating conditions.

TABLE 1

Reported repairs of North Sea Offshore Structures.

Repair, replacement and installation of new anodes, grinding of smaller cracks and marine fouling removal not included.

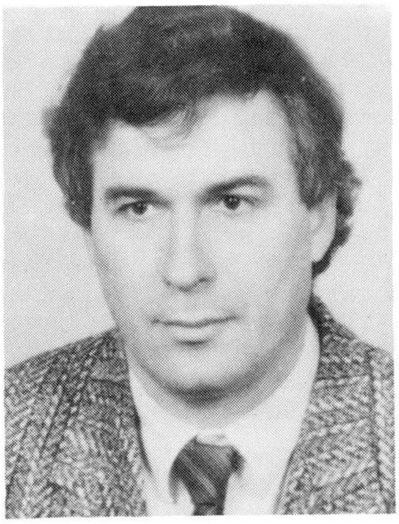
Causes	Steel structures	Concrete structures
Local foundation overload	-	1
Dropped objects	8	2
Ship impact	7	1
Workmanship	5	3
Abrasion by hanging objects	-	2
Precast formwork spalling	-	1
Static overload	8	-
Fatigue	3	-
Other cracks	7	-
Erosion	1	1
	39	11

Bridge Durability Parameters

Paramètres de durabilité de ponts

Brücken-Dauerhaftigkeitsparameter

Jure RADIĆ
Assist. Professor
University of Zagreb
Zagreb, Yugoslavia



Jure Radić, born 1953, graduated at the Civil Eng. Faculty at the University of Zagreb, and received his doctor's degree 1987. His special research interests are dynamics and bridge durability. He is now Assist. Prof. for Bridges at the Civil Eng. Faculty in Zagreb.

SUMMARY

Throughout history a great many bridges have been built on Yugoslavian soil, among them many being exceptional structural and traffic achievements. Detailed data on their present state are presented in the paper, from which follows the need for regular maintenance, but also for the definition of guidelines for design and construction in especially aggressive environments.

RÉSUMÉ

A travers toute l'histoire, une multitude de ponts a été construite sur sol yougoslave. Plusieurs de ces ouvrages sont des réalisations remarquables. Dans l'article, des données précises sur leur état actuel sont présentées. Il en ressort la nécessité d'un entretien permanent ainsi que le besoin d'élaboration de directives relatives à la conception et à la construction dans un environnement agressif.

ZUSAMMENFASSUNG

Zahlreiche Brücken, davon einige mit aussergewöhnlichen konstruktiven Leistungen, wurden in der Geschichte Jugoslawiens gebaut. Daten über ihren heutigen Zustand und die Notwendigkeit der regelmässigen Unterhaltung, sowie Richtlinien für Entwurf und Ausführung für besonders aggressive Umweltbedingungen sind ausführlich dargestellt.

1. INTRODUCTION

Throughout history, paramount accomplishments in bridge construction have often been achieved on the territory of Yugoslavia. Terrain characteristics, main European traffic routes, the presence of superior builders, from Apolodoros and Mimar Sinan to contemporary ones, as well as contacts between prevailing cultures and civilizations influenced the creation of numerous bridges of great value, many of which still endure today. Particularly strong impetus in bridge construction took place during the three decades after World War II. In this period, several thousands of interesting bridges have been constructed, among them a number with exceptional dimensions and accomplishments, as is, for instance, the Tito's Bridge between land and the Krk island with the largest, 390 m span, reinforced concrete arch in the world.

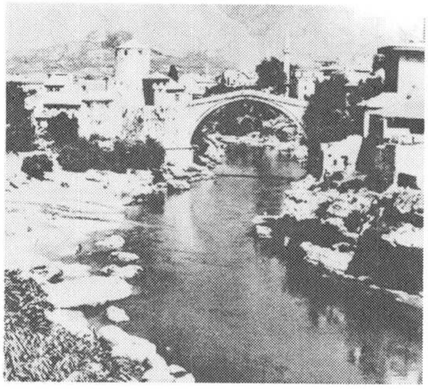


Fig 1. View of Mostar Bridge, built in XVI century

In the last several years, particular significance is being given to the bridge durability parameters analysis, by studying existing structures to determine optimal measures for enhancing their usability and durability. Also the necessary parametrical studies are carried out to investigate general data relevant for assessing the design life of structures to be designed or constructed in the future.

The first comprehensive inspection of 2210 bridges on main roads in Croatia has just been completeed. Thus, a very detailed and complex data base comprising the basic characteristics of such a large number of bridges and their present condition is being formed, on the basis of which parameters of their durability and main causes of damage are being analysed and urgent and long-term maintenance works planned.

2. DATA BASE ON BRIDGES

Modern bridge management services and the planing of all the pertaining activities calls for a comprehensive and well developed data base and an efficient working system with a prompt and regular inspection service. [1]

By the term "data base" we understand a set of all meaningful data on the initial and present properties of any particular bridge, with the programme of planned works, inspections and regular maintenance, as well as traffic and other loads imposed upon the bridge. Such a set, managed by experts and backed up by a contemporary information system and computer techniques, may at all times yield the information on:

-the levels of bearing capacity and serviceability, i.e. degree of degradation,
-the possibilities of allowing the passage of actual extraordinary loads,
-the plan, the schedule and instructions for regular maintenance work,
-specific elements associated with each particular bridge in the system,but also for groups of structures, as are data related to the planning of equipmentand means for regular and irregular works on bridges.

Therefore, the data base should contain regularly updated data:

* Main initial data on a bridge: location, disposition, structure critical details, wearable elements, design loads, construction method, interruptions during construction, built-in materijals, achieved quality, results of carried out examinations and trial loadings.
* Planned programmes of regular inspections for every bridge with directions and manner of inspection depending on the level of investigation. It would be convenient that the inspectors, during lower level inspections, be provided with forms in which they answer prepared questions.
* The plan of regular maintenance works comprising the schedule for the replacement of wearable parts and equipment, surface protection, renewal of surfacing, as well as the maintenance of special parts and equipment (e.g. on movable bridges) etc.
* Return data determined during regular and extraordinary inspections, with detailed descriptions of all observed damages, their causes, state and progress, as well as changes in the materijal. This group also comprises data on works carried out on the bridge the actual traffic conditions and irregular incidents.
* Special data on structural specifics, e.g. conduits in it, special equipment, strategic importance for the economy and other activities etc.

Besides updating these data, the bridge management system should be programmed to automatically output data on the degree of degradation, or on the threatened bridge properties, comparing them with the initial data, the data during a previous period and the allowed ones.[2]
Thus formed and updated data base presents a basis for the workout of efficijent programmes and studious projects for the maintance and protection of structures, the achievement of the planed and

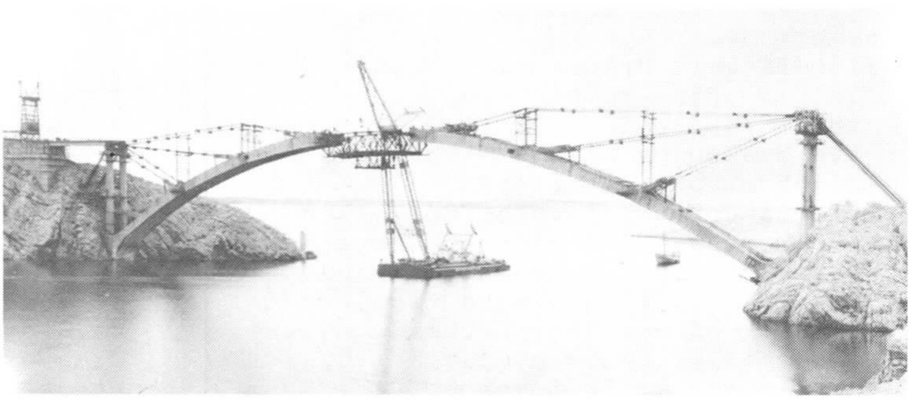


Fig 2. Reinforced concrete arch of Pag Island bridge during construction

designed durability, along with the compliance with bearability, safety and serviceability conditions.

3. INFLUENCES ON BRIDGE DURABILITY

On the basis of a comprehensive analysis and parametrical studies of safety and serviceability levels of a large number of bridges of different age the main factors of influence to the durability of bridges may be determined:

- general characteristics of traffic, particularly useful loads,
- influence of surrounding media, especially specific conditions caused by the action of sea and salt on great reinforced concrete bridges along the Adrtatic coast,
- influence of bridge characteristics (structure, materials, construction procedures, applied protection),
- influence of maintenance (from regular inspections to rehabilitation works).

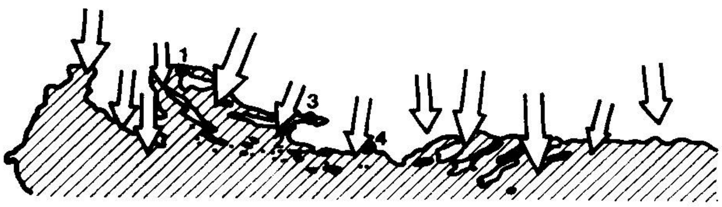


Fig 3. Locations of large arch bridges on the Adriatic coast and zones with particularly strong bura winds
Bridges: 1-Krk; 2-Pag; 3-Maslenica; 4-Sibenik

There are cases when certain, sometimes unexpected, factors lead to a significant durability decrease, and even a catastrophe, but conditions are not ancommon either when some of the mentioned factors take a dominant role and, if due measures for their neutralization are not undertaken, we may be witness unwanted consequences.

A good example of such a dominant influence on the decrease of the expected bridge durability is the aggresive action of sea water which has been observed and analysed on large Adriatic bridges for several decades already.

As known, along the coast and across channels and backwaters of the Adriatic Sea some ten very large bridges have been completed, and among them:

- the Sibenik Bridge (1966), with a 251 m span reinforced concrete arch,
- the Maslenica Bridge (1958), with a 155 m span steel arch,
- the Land-Pag Island Bridge (1968) 198 m span reinforced concrete arch,
- Tito's Bridge, between land and Krk Island (1980), with two reinforced concrete arches, the smaller a 250 m span, the larger a 390 m span.

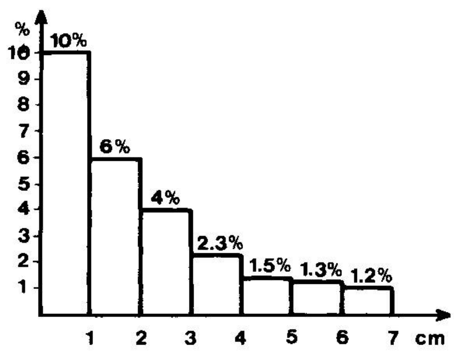


Fig 4. Diagram showing ion content compared with the quantity of cement from surface towards the interior of the Pag Bridge grider

They are exposed to an extremely aggresive action of sea water. The majority of them are located in zones with very strong bura winds, as is the Pag Bridge, where during its construction in 1967 builders recorded 241 day with force 6 winds or stronger, of which 138 days with force 8 winds or stronger. The bura is characterized by very cold winds which blow down the ridges of littoral mountains reaching greatest force along the sea surface, lifting sea drops and soaking the surroundings and these bridges above the sea.

The Adriatic Sea water has approximately the following content of aggresive ions:

SO_4^{2-}	2970 mg/l	Cl^-	21250 mg/l
Mg^{2+}	1420 mg/l	Na^+	11810 mg/l
Ca^{2+}	457 mg/l	K^+	390 mg/l

A detailed analysis and experimental investigation carried out on parts of Pag Bridge has shown that the cause of this aggresive action of the surrounding environment is best observed by the presence of free chlorides in the concrete of the bridge's pavement structure main girders. The amount of chlorides in the surface layer of concrete (up to a 1 cm depth).
In the surface layer (up to 1 cm thick) of concrete the quantity of chlorides compared to the quantity of built-in concrete is 10% and, as shown on Fig. 4., even at the depth of 7 cm (where cables are placed) the chloride content is 1.2% . All this Significantly exceeds allowed levels (e.g. according to Page [3]) even those classified as a great risk of reinforcement corrosion.[4]

The consequence of this is the significant reinforcement corrosion owing to which remedial works are carried out to upgrade the safety and serviceability of the bridge.
Sae water aggressiveness causes not only reinforcement corrosion but also concrete quality decrease, degradation of dilatations, bearings, fences etc and, in cases of metal bridges, the deterioration of every bridge part which is not maintained regularly.

The efects of these aggressive actions on durability have not been known completely and in all their manifestations during the construction of these bridges, but on the basis of actual data on their behaviour guidelines are being worked out as a foundation for the design and construction of structures along the future Adriatic Motorway and other structures in the region also

4. REWIEW OF CONDITION AND DAMAGE CLASSIFICATION OF BRIDGES IN YUGOSLAVIA

During the past five years all most important bridges in Yugoslavia have been inspected with the aim of forming a base of data and planning of the mcst urgent maintenance works on them. Data on the condition these structures are in are illustrated on the example of SR Croatia, where a total of 2210 bridges has been inspected:

- in good condition..........................25.16 %
- with minor deficiences......................56.38 %
- with major deficiences......................12.58 %
- worn out.................................. 5.88 %

On the inspected bridges frequency of defects is different on various parts of structures. Namely, on the total number of inspected bridges defects have been observed in following percentages on:

- fences and bumpers............................59 %
- dilatations....................................50 %
- gutters...47 %
- abutments......................................35 %
- span structures.............................31 %

and on the remaining parts of the structures in lesser percentages.

This already points to the fact that defects are mostly a consequence of deficiencies in design, construction or maintenance and sometimes, inordinate traffic action. a precise analysis of all deficiencies has determined that they are a consequence of:

- design ....27 %
- construction ....41 %
- lack of maintenance..........................32 %
- other causes ....10 %

But this distribution should not be accepted anconditionally, as many deficiencies have such a character that, although they are a consequence of faulty design or construction, they can be eliminated or significantly reduced with adquate maintenance. Also a minor fault in the stage of design or during construction may acquire significant dimensions due to inadequate maintenance.

5. CONCLUSION

From the presented information on the relatively inferior condition and significantly reduced serviceability of a large number of anaysed data the following ensues:

- undelayable need for forming accurate data bases on bridge condition in every transportation system,
- timely and urgent maintenance and damage remedial works at the earliest stages of their manifestation,
- definition of criteria for the design and construction of structures in particularly aggresive environments or similar extraordinary conditions.

All these activities are currently being carried out on the system of bridges in the Yugoslav road network with the goal of regaining the high standard and exceptional accomplichments achieved by their construction.

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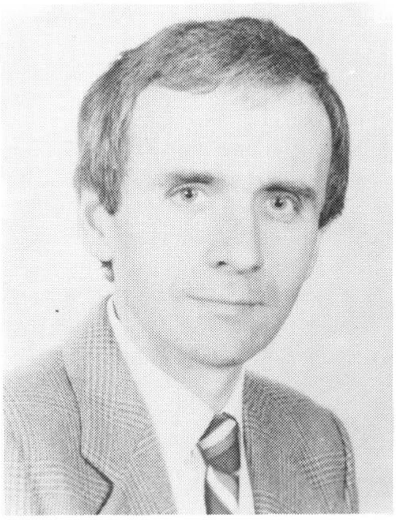
Durabilité de l'état de surface des bétons d'équipements urbains

Dauerhaftigkeit der Betonoberflächen von städtischen Betonbauten

Durability of Concrete Surface of Concrete Public Works

Michel CROC
Directeur Général Adjoint
Société du Métro de Marseille
Marseille, France



Michel Croc, né en 1948, est ancien élève de l'école polytechnique de Paris et ingénieur des ponts et chaussées.

RÉSUMÉ

Le vieillissement des parements de béton des ouvrages publics équipant les villes est loin d'être inéluctable. L'article analyse des causes de dégradations et les moyens de les prévenir.

ZUSAMMENFASSUNG

Die Alterung der Betonaussenseiten von öffentlichen Bauwerken in Städten ist nicht unvermeidlich. Der Bericht untersucht die Gründe der Schäden und die Vorbeugungsmassnahmen.

SUMMARY

The ageing of the surface of concrete public works is far from being unavoidable. This article analyses the causes of damage and the ways to prevent them.

1. LES OUVRAGES EN BETON

- Le béton occupe une place de plus en plus importante dans les ouvrages publics équipant nos villes ; ce, sous des formes multiples, plus ou moins volumineuses et marquantes.
- Au premier plan se situent, bien entendu, les ouvrages d'art tels que viaducs, ponts et grands murs de soutènement.
- Mais il y a aussi les bâtiments publics - cités administratives - hôpitaux - établissements scolaires, qui font un large appel au béton apparent, architecnique en règle général, et, à une moindre échelle, les composants des aménagements et mobiliers urbains.
- En règle générale ces ouvrages ne sont pas "habillés", le béton brut de décoffrage restant apparent. Parfois le béton reçoit une finition, du genre bouchardage ou bien une peinture, mais ces traitements peu fréquents relèvent plutôt du "cache-misère" pour un état de surface peu satisfaisant.

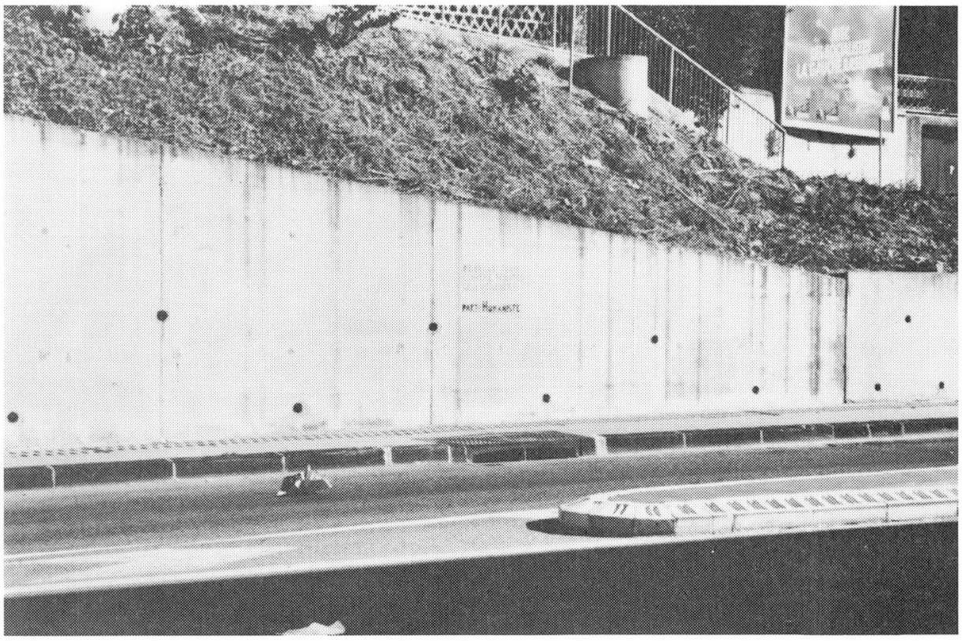


Fig.1 Béton brut sans précautions

2. LES AGRESSIONS SUBIES PAR LES PAREMENTS DU BETON

- Les agressions sont nombreuses et variées. Suivant leur nature elles entraînent des dégradations allant du simple salissement à la corrosion de surface plus ou moins profonde et plus ou moins accompagnée de lichens et autres mousses entretenant le processus de dégradation.
- Au premier rang, en milieu urbain, se situent les agressions athmosphériques, fumées des foyers domestiques et industriels, gaz d'échapement des voitures et, suivant la situation géographique, air salin. Ces fumées et gaz d'échappement, fortement chargés en gaz carbonique et dérivés de souffre ont une action chimique directe sur le béton.
- Vient ensuite la poussière. En milieu urbain les poussières sont fortement chargées en silicates (40 %), carbonate (20 %), matières organiques (10 %), sels

solubles (10 %), graisses (10 %), suie (5 %), et humidité (5 %). Ce cocktail adhère par tension superficielle à la surface du béton lorsque, et c'est presque toujours le cas, l'humidité de l'air dépasse 50 %. La perméabilité de surface du béton, même faible, les microfissurations et les faïençages permettent la pénétration des poussières dans l'épaisseur du parement, pénétration accélérée par les eaux de pluies ruisselantes. Alors le parement noircit progressivement avec une intensité plus marquée sur les cheminements préférenciels des eaux de ruissellement. A un stade avancé, lichens et mousses s'installent "mangeant" peu à peu la surface du béton.

- Concernant les cheminements des eaux de ruissellement, il importe de préciser que, dans la majorité des cas, ils sont la conséquence d'une mauvaise conception des formes architecturales de l'ouvrage ou du parement lui-même, comme, par exemple, les "moustaches" des appuis de fenêtres ou d'éléments architectoniques saillants.
- La pluie, déjà active dans le processus précédent, a également une part directe dans la dégradation des parements lorsqu'elle est chargée en acides, provenant des fumées industrielles essentiellement.
- Autre agression, humaine celle-ci, mais importante, les graffiti et affiches. Les grandes surfaces planes des parements béton sont malheureusement un support idéal pour ceux-ci et rares sont les ouvrages qui y échappent, au moins à hauteur d'homme, mais cela va parfois au-delà et la malignité humaine est alors sans limites.
- Restent, à une moindre échelle, l'abrasion par le vent chargé de sable qui, en certains sites très exposés, a des effets nettement perceptibles et les agressions mécaniques, volontaires ou non, surtout sensibles sur les arêtes des ouvrages.

3 LES MOYENS A METTRE EN OEUVRE CONTRE CES AGRESSIONS

- Pour les agressions mécaniques, on conçoit bien que les protections ne peuvent être qu'extérieures à l'ouvrage lui-même et, à l'instar des bornes flanquant les anciennes portes cochères, constituer des "pare-chocs", en général métalliques, avancés et interdisant le contact direct. En leur absence, il n'y a plus d'autres recours, après choc, qu'en la réfection.
Il existe par contre de nombreuses techniques préventives contre les autres agressions, sinon à effet définitif du moins garantissant une durabilité très nettement supérieure et des conditions d'entretien faciles.

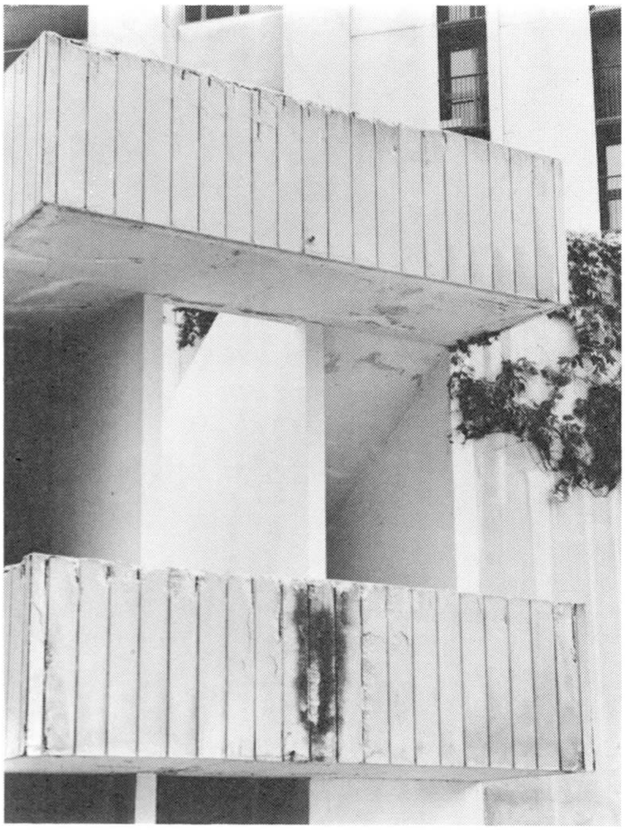


Fig.2 Béton mal traité

3.1 Qualité du béton

- La première gamme de techniques concerne le béton lui-même, tant dans sa confection et sa mise en oeuvre que dans sa conception architectonique.
- Une bonne conception architectonique doit être guidée par le soucis d'éliminer, autant que faire se peut, les causes de ruissellements systématiques, éléments saillants en particulier, ou de rétention d'eau et aussi les "masques" aux vents dominants créant des zônes jamais lavées par la pluie.
- La qualification initiale des bétons et de leur mise en oeuvre est évidemment une condition essentielle à remplir pour obtenir une bonne résistance aux agressions.

  Cette qualification porte sur :

- La composition du béton lui-même, le choix du ciment (p.ex.béton blanc), et des agrégats, dont la coloration a un rôle important quant à la couleur finale du parement et à la stabilité de celle-ci.
- L'incorporation éventuelle de colorants (pigments naturels minéraux).
- les dosages précis et constants ainsi que l'homogénéité et la densité, facteur essentiel de la durabilité, obtenues par un malaxage optimum.
- La conception des coffrages, d'une part quant à leur calepinage lui-même et à la répartition des trous de coffrage qu'il est souhaitable en outre d'équiper de cols propres et de ne pas reboucher ; d'autre part, quant à leur mise en oeuvre précise. Enfin un soin particulier doit être attaché à l'application des produits de décoffrage qui doivent être propres, sans souillures et absolument neutres à l'égard du béton.
- Le bon positionnement des armatures, évitant des fers trop proches de la surface et créant des points de faiblesse du parement.

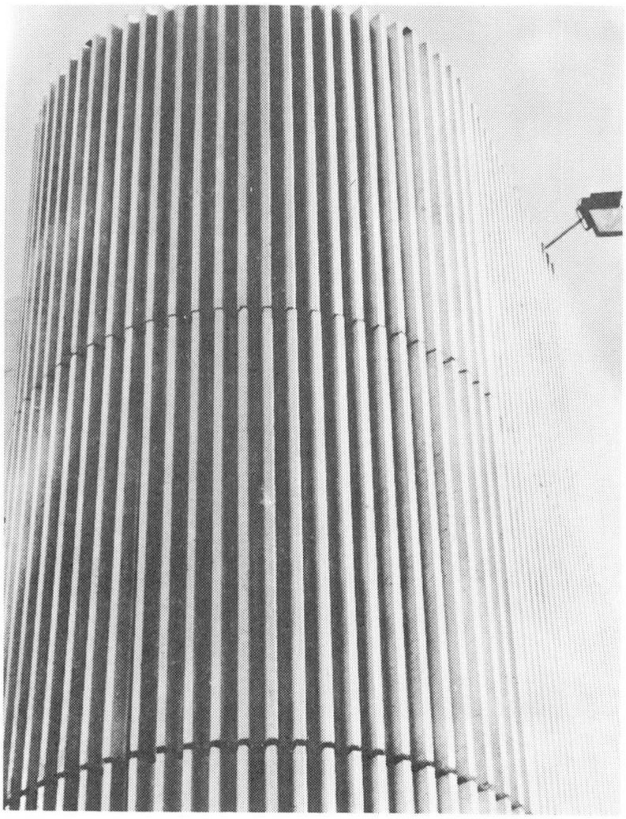


Fig.3 Béton urbain

- Une mise en oeuvre sérieuse par couches horizontales relativement minces, avec compactage à l'aiguille poussé pour obtenir un béton très homogène, très dense et sans lignes de faiblesse en parement.
- Un bon durcissement du béton, bien contrôlé, aidé au besoin par des adjuvants adaptés.

3.2 Traitement des surfaces

Ensuite viennent les traitements des surfaces dont les objectifs sont:

- soit de renforcer la dureté de surface et d'éliminer la porosité et le faïençage du parement - sources de pénétration de l'eau et des poussières, tels le lavage du parement avec un fluosilicate de magnésium - trois semaines après la coulée - transformant les carbonates de chaux en fluorures de chaux, sels durs et insolubles, résistants à l'érosion, imperméables et résistant aux attaques chimiques.

- soit d'imperméabiliser le parement, interdisant ainsi le développement des mousses et des lichens et facilitant l'auto-élimination des salissures hydrosolubles et le nettoyage des parements.

Les produits d'imperméabilisation agissent par pénétration dans le support tout en le laissant respirer.

Ces produits, pour être efficaces, doivent avoir une très faible tension superficielle assurant une pénétration profonde inférieure à 9 mm, avoir une bonne résistance à l'oxydation, à l'alcalinité du béton et à l'action des rayons ultra-violets.

Doivent être évités tous les produits aboutissant à la formation d'un film homogène non pénétrant dont l'adhérence au support n'est pas bonne et qui peuvent cloquer puis se désquamer.

De nombreux produits existent commercialement, essentiellement à base de silicones ; encore faut-il être très sélectif quant à leur formulation portant en particulier sur leur fluidité et leur pouvoir de pénétration, condition essentielle d'un bon résultat. En effet une pénétration lente se traduit par la formation en surface d'une "peau" non stabilisée et qui, par collage ou effet électrostatique, fixe les poussières, aboutissant à l'inverse du résultat souhaité.

Il existe, moins répandues sur le marché, d'autres résines plus performantes, à base d'oligomères de siloxane, très fluides et ayant donc un fort pouvoir de pénétration, qui ont la particularité d'une polymérisation lente, se terminant donc dans la masse du béton et dormant une excellente homogénéité du produit avec celui-ci.

En règle générale les meilleurs produits peuvent donner une garantie décennale mais il n'en demeure pas moins qu'alors que les traitements de durcissement sont définitifs ceux d'hydrofugation sont à renouveler périodiquement.

3.3 Protection des parements

Enfin il y a la protection des parements contre les graffiti et les affiches. Cette protection a un but essentiellement esthétique, graffitis et affiches ne remettant pas en cause la stabilité chimique et mécanique du béton. En outre, le fait de traiter les surfaces ne dispense-t-il pas d'un suivi régulier des ouvrages et d'interventions rapides de nettoyages et de renouvellement de la protection.

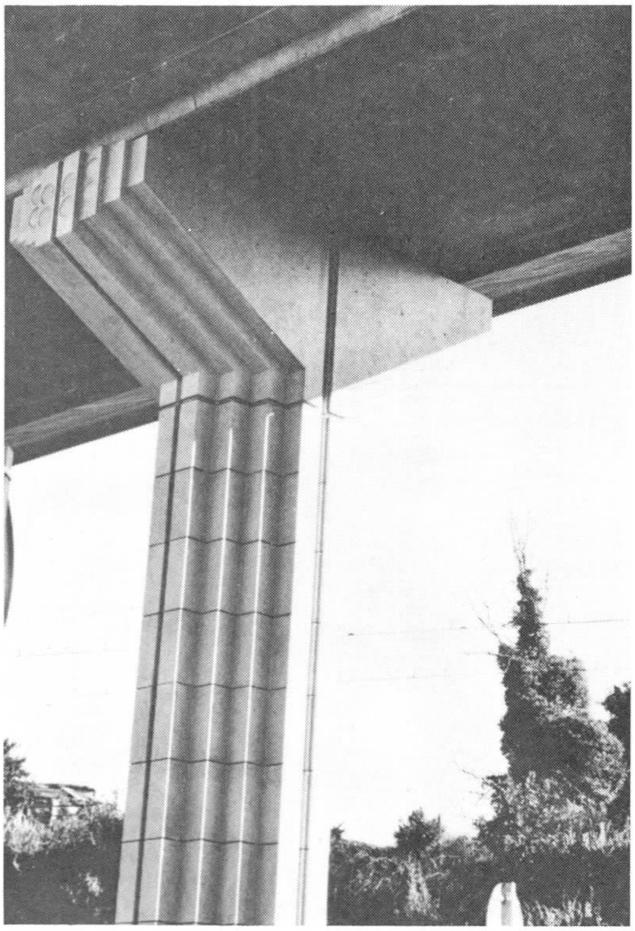


Fig.4 Pile soignée

- En effet, les résines employées empêchent effectivement la pénétration des peintures et colles jusqu'au béton mais en fixent une partie, et d'autant plus que l'intervenon de nettoyage est tardive. Pour le nettoyage il est donc nécessaire d'employer des diluants compatibles avec les résines et on aboutit à une perte de matière qui doit immédiatement être compensée par une nouvelle application.

- Il est à noter que, hydrofuges on antigraffitis, tous ces produits ont un très faible pouvoir colorant et ne modifient qu'imperceptiblement la teinte d'origine du béton. Mais ils peuvent également être colorés - essentiellement avec des pigments minéraux - si l'on recherche un effet décoratif sans, pour autant, vouloir "peindre" le parement.

4. CONCLUSION

Ainsi, comme nous venons de le voir, le vieillissement des parements de béton est loin d'être un phénomène inéluctable.

Encore faut-il que Maîtres d'Ouvrages, Maîtres d'Oeuvres et Entreprises aient pleinement conscience de l'enjeu, tant sur le plan de l'esthétique que sur celui de la maintenance des ouvrages et sachent accepter pleinement les contraintes techniques et financières seules susceptibles de garantir la beauté et la durabilité des ouvrages d'art en béton.

Grundsätzliche Anmerkungen zur Dauerhaftigkeit von Stahlhallen

Basic Remarks on the Durability of Single-Storey Steel Buildings

Remarques fondamentales sur la durabilité des halles métalliques

Joachim SCHEER
Bauingenieur
TU Carolo-Wilhelmina
Braunschweig, BR Deutschland

Joachim Scheer, geb. 1927, promovierte 1958. Als Beratender Ingenieur befasste er sich mit der Planung von Brücken, Masten und Gerüsten. Seit 1971 zunächst in Hannover, ab 1976 in Braunschweig Professor für Stahlbau, arbeitet er vorwiegend an Problemen der Plattenstabilität, der Berechnung von Masten und der Normenentwicklung.

Hartmut PASTERNAK
Bauingenieur
TU Carolo-Wilhelmina
Braunschweig, BR Deutschland

Hartmut Pasternak, geb 1954, promovierte 1981. Danach befasste er sich mit Problemen des Stahlhallen-, Kranbahn- und Silobaus. Seit 1986 ist er wissenschaftlicher Mitarbeiter am Institut für Stahlbau in Braunschweig und arbeitet an Fragen der Grenztragfähigkeit von Trägern und der Lastmodellierung.

ZUSAMMENFASSUNG

Ausgewertet werden zunächst über 100 mittlere und grosse Schadenfälle von Stahlhallen. Anhand ausgewählter Beispiele werden dann qualitative und quantitative Aspekte heutiger Gebrauchstauglichkeitskriterien im Hinblick auf den tatsächlichen Erhalt der Funktionstüchtigkeit untersucht.

SUMMARY

Evaluations are made for more than 100 middle-sized and large cases involving damage to single-storey steel buildings. On the basis of selected examples, qualitative and quantitative aspects of current serviceability criteria in regard to the real maintenance of the utility are investigated.

RÉSUMÉ

Plus de cent cas de dommages de moyenne et grande importance de halles métalliques ont été étudiés. Des aspects qualitatifs et quantitatifs des critères de l'aptitude au service ont été déterminés à l'aide de cas choisis, pour vérifier les besoins d'entretien.

1. STAHLHALLEN UND DER BEGRIFF DAUERHAFTIGKEIT

Stahlhallen gehören in den Industrieländern seit Jahrzehnten zu den Standarderzeugnissen der Baubranche; in Großbritannien werden ca. 90% aller Hallen in Stahl errichtet, in Italien sind es nur etwa 10%; die BR Deutschland liegt mit ca. 40% im Mittelfeld.

Trotzdem hat es bisher - auch in dieser Hinsicht sind Hallen typische Bauerzeugnisse - keine systematischen Schadensanalysen wie etwa in der Automobilindustrie gegeben, die Aufschluß über die tatsächliche Dauerhaftigkeit solcher Objekte geben könnten.

Unter Dauerhaftigkeit eines Bauwerks versteht man seine Fähigkeit, sich während seiner Lebensdauer funktions- und betriebsgerecht zu verhalten. Die Deutung des Begriffs Dauerhaftigkeit ist unter Bauingenieuren nicht unumstritten. Im Entwurf der SIA 160 (März 1988) ist Dauerhaftigkeit nur eine Teilmenge der Gebrauchstauglichkeit; im Entwurf des EC3 (Juli 1988) steht sie eher neben Tragfähigkeit und Gebrauchstauglichkeit, wenn u.a. gefordert wird,

- Umwelteinfüsse und Bauwerkslage,
- Nutzung und erwartete Lebensdauer des Bauwerks,
- Eigenschaften der verwendeten Werkstoffe,
- Gestaltung der Bauteile und konstruktiven Details,
- Qualität der Ausführung und Niveau der Qualitätskontrolle sowie
- eine geeignete Bauwerkserhaltung

bei der Bemessung zu berücksichtigen. Schließen wir uns der zweiten Auffassung an und werfen wir einen Blick auf die Grenzzustände von Bauwerken.

Heutige Regelwerke - wie etwa der Entwurf des EC 3 - gehen aus von einer Lebensdauer von 50 Jahren und begrenzen für diesen die rechnerische Versagenswahrscheinlichkeit auf

- p_f = 5·10E-5 für die Grenzzustände der Tragfähigkeit,
- p_f = 5·10E-2 für die Grenzzustände der Gebrauchstauglichkeit.

Dem stehen nach unserer Kenntnis folgende Orientierungswerte für Hallen gegenüber:

- für die jährliche Häufigkeit von Katastrophen mit großem finanziellen Verlust oder Personenschaden: $2*10^{-5}$,
- für die Häufigkeit von wesentlichen Nachbesserungen zur Gewährleistung der Gebrauchstauglichkeit: 1 mal pro 50 bis 100 Hallen.

Durch die in der probabilistischen Analyse nicht erfaßten Einflüsse (Fehler bei der Herstellung und Nutzung) wächst die Versagenswahrscheinlichkeit ausgeführter Tragwerke üblicherweise auf ein Vielfaches. So erscheint hier die Differenz in den Versagenswahrscheinlichkeiten im Grenzzustand der Tragfähigkeit eher gering, was auf "Reserven" der konventionellen Bemessung (etwa von Rahmen oder durch Vernachlässigung der räumlichen Tragwirkung) schließen läßt. Bedenken läßt dagegen der gravierende Unterschied bei den Versagenswahrscheinlichkeiten in den Grenzzuständen der Gebrauchstauglichkeit aufkommen.

2. SCHÄDEN - AUSWERTUNG EINER STICHPROBE

Wenn systematische Schadensanalysen nicht geführt werden oder werden können, ist die Untersuchung von Stichproben meist zweckmäßig. Nachfolgend wird über die Ergebnisse einer Auswertung von über 100 mittleren (d.h. mit erheblichem Aufwand sanierbar oder Personengefährdung) bis großen (d.h. großer finanzieller Verlust oder Personenschaden), in den vergangenen 20 Jahren bekannt gewordenen internationalen Schadensfällen an Stahlhallen berichtet. Kleinere Schäden, die den Bereich der öffentlichen Sicherheit nicht berühren, sind in der Literatur nicht dokumentiert. Die Untersuchung zeigt, wann (Bild 1a) bzw. durch welche Fehler (Bild 1b) diese Schäden auftraten - es dominieren die Nutzungsphase bzw. Planungsfehler. Zu letzteren gehören mehrfach nicht zutreffende Schneelastannahmen (teilweise wegen unzureichender Normwerte) und subjektive Fehler wie z.B. Nichtverfolgung von Schnittkräften bis in Verbindungen. Nutzungsfehler treten in erster Linie auf infolge Beanspruchung durch Lasten, die im Entwurf gar nicht vorgesehen waren. Auf Bild 1c ist die prozentuale Häufigkeit der Anzahl der Schadensursachen nach Bild 1b gezeigt; zwei Drittel aller Schäden sind auf eine Fehlerkombination (z.B. aus Planungs- und Ausführungsfehlern) zurückzuführen.

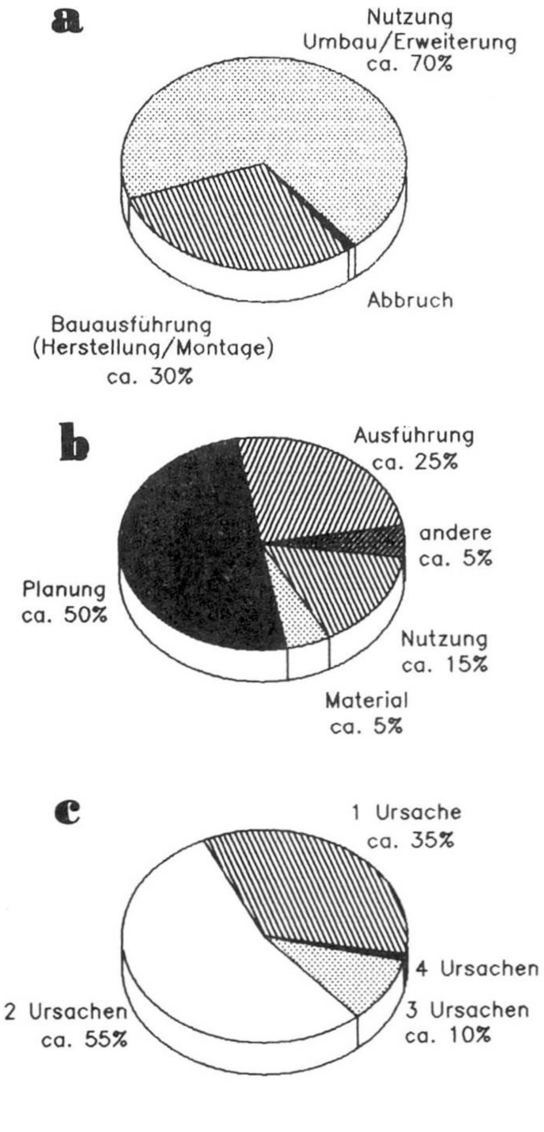


Bild 1 Schadensauswertung

Deutlich wird erneut die Eigenart von Stahlkonstruktionen: dem problemlosen Baustoff Stahl stehen hohe Anforderungen an Planung (z. B. wirken sich durch den geringen Eigenlastanteil unerwartet hohe Schneelasten stärker aus), Ausführung und Nutzung gegenüber. Mittlere und große Bauschäden von Hallen werden heute nur noch selten durch neuartige technologische Probleme (wie früher z.B. durch nicht beherrschte Stabilitätsfragen oder Schweißmängel) verursacht, sondern eher durch unzureichende Koordination der am Bau Beteiligten. Das Spektrum der Schäden ist breit. Es reicht vom Einsturz ganzer Stützen-Binder-Systeme bzw. Rahmen über das Versagen einzelner Anschlüsse bis zu Verformungen der Längsverbände infolge fehlender Temperaturfuge in Hallenlängsrichtung sowie Risse in der Bauwerkshülle durch ungleiche Stützensetzungen oder fehlende Übertragung von Bremskräften aus Kranbetrieb.

Die bisherigen Untersuchungen bestätigen für Stahlhallen die Gül-

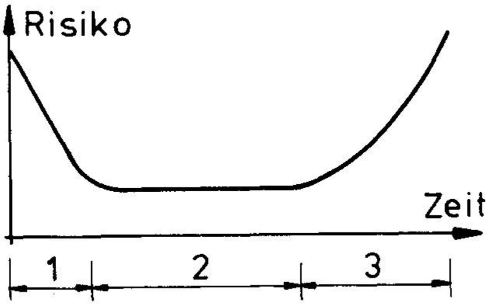


Bild 2 Risiko-Zeit-Funktion

tigkeit zumindestens der ersten beiden Abschnitte der klassischen Risiko - Zeit - Funktion (Bild 2); da Hallen keiner perfekten Kontrolle bei der Abnahme unterliegen, setzt mit der Nutzung die natürliche Selektion (1) ein; das verdeutlichen z.B. die Schneelasten: Schäden unter dieser Last sind während der ersten fünf Nutzungsjahre viermal häufiger als in den verbleibenden. Weitere Phasen sind die reguläre Nutzung (2) und das Altern (3), über dessen Beginn und Verlauf mit dieser Stichprobe keine ausreichend gesicherten Erkenntnisse gewonnen werden konnten.

Inwieweit ist die Untersuchung mittlerer und großer Schadensfälle für Dauerhaftigkeitsbetrachtungen relevant? Eine Antwort gibt Bild 3. Es zeigt anhand des nachfolgenden (sicher extremen) Beispiels, daß Fehler, die in dieser Auswertung zum Verlust der Tragsicherheit führten, unter anderen Umständen "nur" eingeschränkte Gebrauchstauglichkeit nach sich gezogen hätten.

Bei starker Schnee- und Windbelastung sowie einer Temperatur von -22°C versagten die Schrauben, mit denen das Zugband eines unterspannten Fachwerkriegels (Spannweite 15 m) an dessen Knotenblech befestigt war. Das führte zum Versagen der (unten eingespannten) Stützen im Unterbereich bzw. zu starken Abweichungen aus dem Lot. Während im Entwurf ein verschiebliches Lager für den Fachwerkbinder vorgesehen war, wurden beide Lager unverschieblich ausgeführt. Das ergab zusätzliche Kräfte aus Wind, Temperatur und unsachgemäß gelagertem Lagergut. Hinzu kam die unzureichende Ausführung mehrerer Schweißnähte. Insgesamt stürzten 10 Riegel ein.

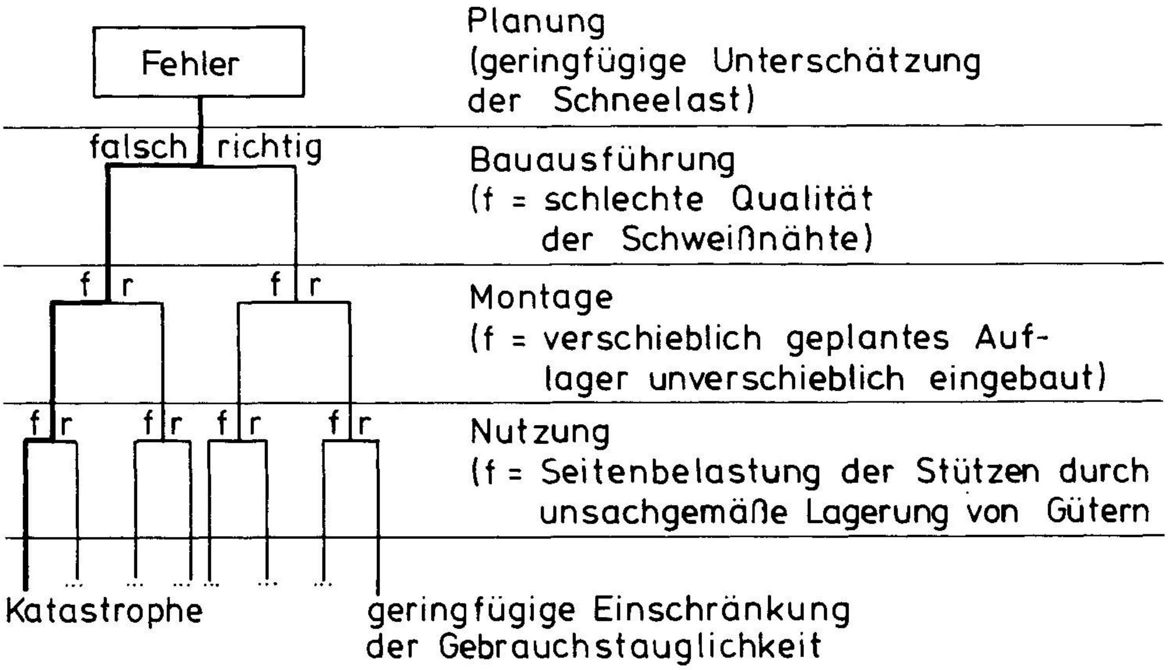


Bild 3 Ereignisbaum Stützen-Binder-System

Nicht erfaßt werden konnten mit dieser Stichprobe möglicherweise signifikante Unterschiede zwischen Massenherstellung und individueller Anfertigung, zwischen Industrie- und Ausstellungshalle. Bestätigt wurde dagegen, daß sich Nutzungsziele nicht immer durch Berechnungen erzielen lassen, sondern auch durch die Wahl geeigneter konstruktiver Durchbildung, durch Ausführung, Kontrolle und Unterhalt. Nachfolgend widmen wir uns der rechnerischen Seite.

3. ÜBER GEBRAUCHSTAUGLICHKEITSKRITERIEN

3.1 Qualitative Aspekte

Auch hier wieder ein Beispiel. Über die Dauerhaftigkeit einer Kranbahn entscheiden bekanntlich insbesondere die Schiefstellung der Kranlaufräder sowie dauerhafte und elastische Deformationen in der Horizontalebene der Fahrbahn.

Heutzutage beschränken wir uns beim sog. Gebrauchstauglichkeitsnachweis allein auf die Begrenzung der elastischen Durchbiegung der einzelnen Kranschiene auf z.B. 1/800, der Kopfauslenkung der einzelnen Kranbahnstütze auf h/300 bzw. der Differenz der Kopfauslenkung gegenüberliegender Stützen auf 20 mm, fordern also eine Art Mindeststeifigkeit. Nun ist z.B. die zweite dieser Forderungen berechtigt, etwa um das Herausspringen von Fensterscheiben beim Vorbeifahren eines Brückenkrans (ein Schadensfall in Abschnitt 2) zu verhindern. Was aber bedeuten diese Kriterien für den "Leitgebrauch"?

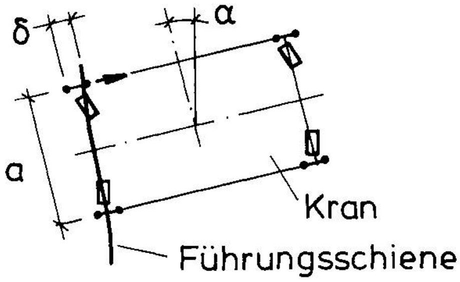


Bild 4 Kranschrägstellung

Zum Nachweis der Gebrauchstauglichkeit ist es erforderlich zu überprüfen, ob die Gesamtschrägstellung α des Krans (aus den für den Betrieb prognostizierten geometrischen Imperfektionen und elastischen Verformungen der Kranbahn in der Horizontalebene einerseits sowie infolge prognostizierter Laufradschrägstellung andererseits) kleiner ist als die überhaupt mögliche Schrägstellung, die sich als Quotient aus dem Spurspiel δ zwischen Führungselement und Schiene sowie dem Abstand a zwischen den Führungselementen ergibt (Bild 4). Ist diese Bedingung erfüllt, fährt der Kran im sog. hinteren Freilauf - einer Stellung, die sich günstig auf die Dauerhaftigkeit der Kranbahn auswirkt.

Wir erkennen an diesem wiederum drastischen Beispiel, daß Vorschriften zwar Schritte zur Sicherung der Dauerhaftigkeit vorsehen, ohne daß dann daraus später immer Schlußfolgerungen hinsichtlich der Dauerhaftigkeit gezogen werden können. Neue Gebrauchstauglichkeitskriterien müssen gefunden werden.

3.2 Quantitative Aspekte

Versagenswahrscheinlichkeit. In der letzten Zeit wird vorgeschlagen, in vielen Fällen größere rechnerische Versagenswahrscheinlichkeiten in den Grenzzuständen der Gebrauchstauglichkeit zu akzeptieren, z.B. p_f = 0.1...0.2/Jahr [1]. Aus dem Vergleich mit Abschnitt 1 sehen wir, daß diese Auffassung dem status quo im

Hallenbau näher kommt als der bisherige Wert p_f = 10E-2/50 Jahre (insbesondere, wenn man bedenkt, daß dort nur Schäden mit wesentlichen Einschränkungen der Gebrauchstauglichkeit erfaßt wurden).

Lebensdauer und Nutzungsbedingungen. Hüllelemente werden aus Gründen der Alterung aber auch des veränderten Geschmacks mitunter schon nach weniger als 20 Jahren ersetzt. Auch viele Brückenkrane überschreiten dieses Alter nicht: mal werden sie durch neue, leichtere Krane gleicher Tragfähigkeit ersetzt, mal werden Krane größerer Tragfähigkeit gewünscht. Das zieht veränderte Anforderungen an die Kranbahn nach sich. Ähnliches gilt für Installationen und Beleuchtung. Aber selbst grundlegende Änderungen der Nutzung von Hallen sind häufig. Der heutige, generell 50jährige Bezugszeitraum für Gebrauchstauglichkeitskriterien erscheint unrealistisch, individuelle Nutzungsprognosen sind erforderlich. Mit nur vage definierten Eingangsdaten ist eine sicherheitstheoretisch fundierte, quantitative Aussage über die Gewährleistung der Gebrauchstauglichkeit nicht möglich. Ein erster Schritt aus dieser Situation könnte sein, daß zukünftige Regelwerke verfeinerte Lastannahmen für unterschiedliche Bezugszeiträume anbieten.

3.3 Kontrolle

Kehren wir noch einmal zurück zum Beispiel aus Abschnitt 3.1. Die Ausgangsdaten für den dortigen Gebrauchstauglichkeitsnachweis, die Lage der Kranbahn und die Stellung der Kranlaufräder, werden bei der Montage vermessen und mit Herstellungstoleranzen verglichen, spätere Vermessungen von Kran und Kranbahn erfolgen meist nicht mehr regelmäßig (deren Interpretation wäre übrigens nicht problemlos, da es bis heute keine Festlegungen für Betriebstoleranzen gibt!), sondern vielfach erst dann, wenn Schadensfälle z.B. an der Kranbahn eintreten (wie in der o.g. Auswertung mehrmals registriert). In [2] wird gezeigt, wie zu verschiedenen Zeitpunkten durchgeführte Messungen der Kranbahnlage auf probabilistischer Grundlage ("Markov-Ketten") ausgewertet werden können und zu zuverlässigen Aussagen über die Dauerhaftigkeit führen. Ebenfalls abgeleitet werden kann so die notwendige Häufigkeit von Messungen, die zur "Vorwarnung" erforderlich ist. Möglich ist auch eine Zustandsprognose (mit welcher Wahrscheinlichkeit treten welche Imperfektionen auf?), wenn Ausgangssituation und Betriebsbedingungen bekannt sind.

4. SCHLUSSBEMERKUNG

Die Auseinandersetzung um das Konzept der Teilsicherheitsbeiwerte wurde angetreten mit der Absicht, die rechnerische der tatsächlichen Sicherheit anzunähern. Soll die für Stahlhallen bestehende Diskrepanz überwunden werden, muß insbesondere der Katalog der Gebrauchstauglichkeitsanforderungen (z.B. in SIA 160 und EC3-Entwurf) überprüft, neu gestaltet und stetig aktualisiert werden.

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