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SESSION 1

PROBABILISTIC CONCEPTS IN STRUCTURAL EVALUATION

KEYNOTE SPEAKER

Codes of Practice for the Assessment of Existing Structures
Normes pour l'évaluation de structures existantes
Normen für die Beurteilung existierender Bauwerke

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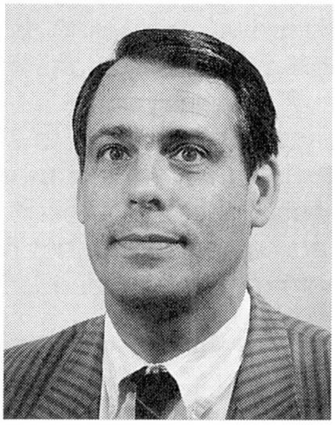

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SUMMARY
This paper summarizes the essential characteristics of the assessment of existing structures and the possibility of codification in this field. The present situation in various countries is summarised. An overview is given of those items that should be addressed by a code on existing structures. Suggestions for the development of such a code are presented.

RÉSUMÉ
Cet article présente un résumé des caractéristiques essentielles pour l'évaluation de structures existantes et les possibilités de normalisation. La situation actuelle dans différents pays est présentée sous forme de résumé. Les aspects importants qui devraient être traités dans une future norme pour les structures existantes sont présentés. Quelgues propositions sont faites en vue du développement d'une telle norme.

ZUSAMMENFASSUNG
Diese Publikation fasst die wichtigsten Merkmale der Beurteilung existierender Bauwerke und die Möglichkeiten der Normung auf diesem Gebiet zusammen. Sie enthält eine Zusammenfassung der heutigen Lage in mehreren Ländern. Es wird eine Übersicht des Hauptgesichtspunkte gegeben, die eine Norm über existierende Bauwerke ansprechen muss. Vorschläge für die Entwicklung solch einer Norm werden präsentiert.

1. INTRODUCTION

Most building codes deal with the design of new structures only. In many cases these codes are even quite explicitly referred to as design guides. The problems which are encountered when assessing existing structures are often not mentioned. In the historical and social context this is fully understandable. Especially in the period after the second world war, most countries experienced an enormous growth in building production of all kinds and there was a clear need to have this production guided by means of a system of codes.

This period of large building activity seems, at least to some extend, to have come to an end. On the other hand, there is an increasing number of existing structures that are questioned with respect to their fitness for use. As a result, assessment of existing structures is no longer an occasional job for some specialists, but becomes more and more a part of common engineering practice, which requires guidance in the same way as design.

2. PRESENT SITUATION

In discussing codification related to exisiting structures, it is useful to distinguish between code type documents and guidelines. At present, only a few countries have a general applicable and real code type document for the assessment of existing structures. As far as the author is aware, only in Czechoslowakia [1] and in The Netherlands [2] such a document exists. In Canada and USA codes are in preparation [21, 22]. Guidelines exist in a larger number of countries. Examples are references [3-5].

The typical difference between a code and a guideline is that codes essentially deal with minimum requirements on the structures, while guidelines primarily provide information about how to plan and carry out the assessment activities in a systematic and economic way. A guideline does not touch the subject of possible reductions in safety targets or load levels, or gives recommendations only. A code, on the other hand, has to tell explicitly what the acceptance criteria are. As a rule, it will not inform the engineer what particular inspections or calculations might be useful for an economic assessment within certain circumstances. As it is, there seems to be a need for both types of documents.

In the first paragraphs of this section only general documents were discussed. Many countries may have documents which deal with the assessment of particular aspects or special catagories of structures, like bridges [6-10], towers [11], seismic aspects [12, 13], offshore structures, remodelling [14], and so on. These documents are sometimes of a guideline and sometimes of a code type nature. It is clear that these documents have been developed from a special need. Many bridges, for instance, are confronted with much heavier loads than anticipated during design. Both with respect ot ultimate load capacity as to fatigue resistance, this has caused much concern. Apart from that, the maintenance of bridges is an activity where budgets are always limited, and priorities have to be set in a very careful way.

Considering the present state of codification, it is also interesting to have a look at international codes. In the revision of ISO-2394 [15] on Reliability of Structures some general statements will be given, indicating on what type of arguments criteria and rules should be based. Some of the recommendations in the further sections of this paper will be based on the present draft of this document.

In the present draft of Eurocode 1, Basis of Design [16], the statement is present that Eurocodes, without appropriate modification, cannot be used for the assessment of existing structures. It was decided that further guidance would require too much prenormative research to be ready for the present edition.

In this respect it is also worthwile to point to prenormative research that has been carried out by CEB [17-19] and that is under progress by the Joint Committee on Structural Safety [20]. This may raise the expectation that in the next editions of Eurocode and ISO the situation will be improved. Also the present paper is intended to stimulate this process.

3. FUNDAMENTAL DIFFERENCES BETWEEN DESIGN AND ASSESSMENT

In the design situation, the engineer has many degrees of freedom to adapt the structural dimensions or even the concept of the structural system. This way he may, with relatively little effort, prove that the structure meets all the design requirements. If necessary he can, without much additional costs, strengthen a structure which does not fulfil the (calculatory) requirements.

This situation, however, changes fundamentally once the structure has been build. Compare for example the possibility to add a single reinforcement bar to a concrete beam in the design stage to the same modification in an existing structure. In the first case the additional costs are very small, in the second case they may prove to be prohibitive. Furthermore, the history of a building and all the changes and damages that have occurred, may lead to a complex and often fuzzy structural system. In addition there may be a great uncertainty with respect to the geometrical and material properties in the structure. This means that even if an existing structure meets the requirements, it may be very difficult to prove it. The conclusion should be that all requirements that are used in the design situation, and with good reason, are not automatically applicable in the 'as built' situation. We will come back to this in chapter 5.5.

On the other hand, the advantage of existing structures compared to structures to be designed is the possibility to measure their properties: one can measure the geometrical dimensions, the material properties, some of the loads and loading parameters, the structural behaviour, the structural response, it's degree of deterioration, and so on. In practice these possibilities of course are limited, because of the costs involved. But even visual inspection or the observation that a structure has survived some heavy load situations without any damage, may help to get a better view on the properties of the structure than is possible in the design stage.

Finally, in the design stage one has to prove that a structure is fit for a certain intended period of use, the design working life. This may lead to special and often implicit durability requirements. In some design concepts even the reference period for the design loads is related to the design working life. If a designed structure does not meet these requirements the design should be changed. It will be clear that the same requirements, implicit or explicit, do not hold for an existing structure: if an existing structure does not fulfil a long term durability requirement, it does not mean that the structure should be rejected immediately. One should consider the costs: it might be more economical to leave the structure as it is and accept a (possible) shorter remaining life time.

Summarizing this section, the following fundamental differences between structures to be designed and existing structures have been observed:

- the increased cost to strengthen the structure
- the increased difficulty of structural analysis
- the possibility to do measurements and observations
- the possibility to reduce the reference period

What further may be concluded is that the process of assessment the existing structure is a far more diffuse process than the process of design. The design process is much more universal, while problems encountered in the appraisal of exising structures seems to be more of a unique nature. This means that, for the case of assessment, it will be more difficult to give detailed guidance to the engineer. As an example, consider the following statement from the Czech code [1]:

> "The extend of tests depends on the type of materials, structural system, execution method, homogeneity of the material, technical possibilities of sampling and also on the purpose of the tests."

A statement of this type certainly may help the engineer to find the right way of thinking, but it leaves on the other hand many possiblities where he has to find the answers on the basis of his own judgement. Design guides may be expected to give more specific information.

4. GENERAL ASPECTS OF CODIFICATION

The conclusion of the previous discussion is that a design code can not be used directly for the assessment of existing structures: some clauses will need modification, some may not apply at all and a number of additional clauses may prove to be necessary. This seems to justify the writing of a special assessment code. In order to be complete and operational, such a code should adress at least the following aspects:

1. Criteria to do an assessment
2. Structural properties and loads
3. Evaluation of inspection results
4. Structural analysis
5. Acceptance criteria

The first item already indicates a typical difference between a design code and a code for existing structures: every new structure should simply be accounted for in one way or another but when exactly an existing structure should be assessed and to what detail is already a difficult matter in itself. The most essential difference, however, between the two types of codes becomes manifest in the last item: to what degree can it be justified to release the design requirements and to accept reduced criteria in the case of assessment. All five items mentioned above will be discussed to some detail in section 5.

Additional to the specification of a set of minimum requirements in a code type document, there is, as stated in section 2, a need for giving some guidance to the engineer how to tackle the assessment. Such a guidance will, according to section 3, necessarily be of some global nature. Most problems related to existing structures are of a rather unique nature and guidance for that reason cannot be very specific. It will leave more detailed decisions to the judgement of the engineer. What guidance can be given will be discussed in section 6.

5. DISCUSSION OF A CODE TYPE DOCUMENT

In the previous section five aspects have been mentioned that should be covered by a code on existing structures. In this sections these five items will be discussed one by one.

5.1 Criteria to do an assessment

The code should list the various reasons that might exist to start an assessment procedure for an existing structure. A distinction should be made between situations where the assessment is required by the code and situations where the need to assess comes form other sources, but the code nevertheless applies. The most typical cases where a code could require an assessment are:

- Routine

A routine assessment can as a rule be relatively simple: the structure is inspected and if the result is within predefined limits, the assessment is that the structural capacity is still sufficient. If the result is outside the predefined limits a further investigation might follow, or it might be decided to repair or maintain the structure immediately.

Nowadays, routine inspections are performed by owners on their own initiative (public bodies for instance) or on the basis of a requirement by insurance and classification companies (offshore platforms). The degree and nature of the inspections might differ substantionally, varying from looking into appearance aspects only to intensive searches for fatigue cracks.

A country, having a code on existing structures, could require routine assessments for all types of buildings, or alternatively for certain classes, for instance public buildings. The degree in which this is possible, of course, depends on the legal status of the code.

- Deterioration or (suspected) damage

When there is obvious damage to the structure, for instance corrosion, spalling of concrete, cracks, leakage, heavy settlements, and so on, a structural appraisal should be required by a code. The difficult point of course is to indicate the markation between "innocent deterioration" and "deterioraton demanding further investigation". A code should provide as much guidance as possible, but this typically will remain one of the cases where engineering judgement will be decisive.

The code may also require investigations in the case of "suspected damage". Suppose that a structure has been loaded by some extreme load (earth quake, fire, tornado, explosion). In those cases there might be a request for further investigation, even if this particular structure does not show any visible damage at first sight. The same may hold for other types of shortcomings, resulting for instance from construction errors, bad workmanship, unexpected behaviour and so on. In those cases the defects observed on a single structure often brings a large group of similar structures under suspicion. The interesting advantage in such cases is that the available budget for research and analysis is usually relatively high, enabling the use of advanced engineering tools in the assessment.

Finally, a code should specify that only the parts suffering from real or suspected damage (deterioration) should be checked. There is no reason to inspect all parts if only some parts or aspects are under suspicion.

- Increased loads/extended life/change of use

Structures loaded by higher loads than anticipated during design or structures being at the end of their intended service life, should be investigated. This is an actual problem in many countries as far as bridges are concerned [23, 24]. Many bridges have been designed for traffic loads that are much smaller than the present day traffic loads. Many engineers have the feeling that no further investigation is required if the bridge does not give signs of distress. This is a dangerous way of reasoning. If all structures give proper signs of distress before collapsing, the argument would be valid, but this certainly is not the case. It should at least be verified that the structure indeed will show signs of distress in due time.

- Reconstruction

If a structure is to be reconstructed for whatever reason, an assessment should be mandatory. The mere fact that the building is reconstructed makes it necessary to assess the present situation: the dimensions and properties of the present structure should be known in order to be able to make an analysis of the reconstructed building.

5.2 Structural properties and loads

If the original drawings and specifications still exist and if there is no reason to doubt them, the code should specify that these can be used to assess the geometrical and material properties. Deterioration effects, of course, should be taken into account. Reasons for doubt could be: damage under otherwise normal circumstances, premature aging, bad performance of the structure or of similar structures elsewhere. In cases of doubt measurements are always necessary.

Values for material properties should be taken according to the present day codes. If a material is no longer used, actual codes may not provide relevant information. In that case properties as specified in old codes can be used. Sometimes it will be necessary to make some transformation, for instance from an allowable stress value to a design value or characteristic value. For the sake of analysis it may be sometimes worthwile to do additional tests. For instance in the case that use of plastic properties is made, but for the original material only the elastic properties are known. In such a case ductility tests may be necessary.

If no data is available with respect to the grade of the material used, one may do tests, but it might turn out that it is more economical to take the lowest possible grade, used at the time of construction. A code could allow such an approach.

Loads should always be taken according to the new codes and according to the situation to be expected in the reference time for the assessment. Load reductions may follow from measuring some load model parameters (see 5.3), from economic criteria (see 5.5), or from a reduced reference period.

5.3 Evaluation of inspection results

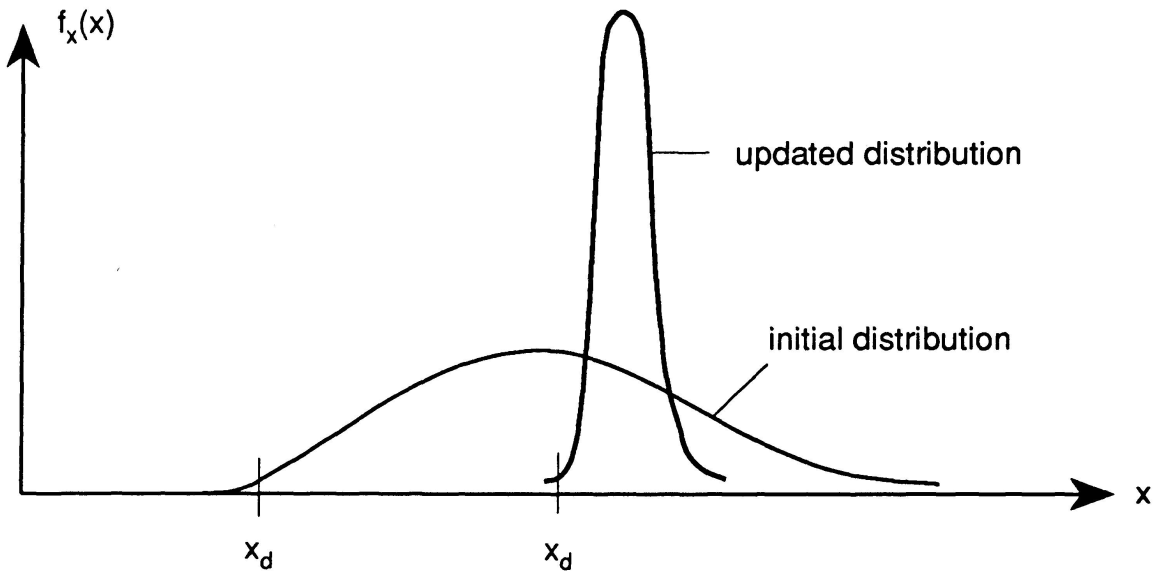
Inspections can be of various types:

- visual
- direct measurement
- non destructive testing
- response measurements
- proof load

The code should specify how data obtained from inspection can be used in the subsequent analysis. A starting point should be that all available data is of value: all information can lead to a better estimate of the structural capacity and to a reduction of the uncertainties. In principle one should combine all information: visual observations, performance in the past, measurements of various kinds and so on. This may require the use of an expert system [25]. From a theoretical point of view, probabilistic methods [26, 27] offer an ideal framework for such a procedure (see Figure 1). In an operational code these formal procedures should be translated into operational methods within a load and resistance factor design approach.

Note that the conclusions of an inspection should not only be concerned with the structure or structural part under consideration. Inspection of one part of a structure always tells something about other parts which are in similar circumstances. Finding a bad concrete quality in one column may increase the probability of finding a bad concrete in another one. In order to use information from tests and measurememts, the accuracy of the inspection method should be known. This is a weak point in the present state of the art. There is a great variety of inspection techniques, but only in a limited number of cases (for instance crack detection in offshore structures) investigations have been done into the so called probability of detection curves and into the quantification of measurement errors.

Figure 1: Original and updated probability density function for an inspected variable x.



Also inspections on the loading side may be of help in the assessment. The research will depend on the type of loading. For offshore structures one may check the local wave climate. For wind loading on special shaped structures one may measure the shape coefficients. For industrial loads measurements may indicate differences from the original design assumptions. Of course one should be careful: loads in codes are intended to repesent the maximum in say 50 year values. In general it is of course not possible to measure this directly.

5.4. Structural analysis

The analysis of existing structures may differ form the analysis of a structure under design. Especially if a structure has been damaged, some of the assumptions which are normally correct in design might be no longer true and other failure mechanisme might become important. As an example, consider a standard steel section. For a new section, the check on local buckling is normally not necessary, but if dimensions are reduced because of corrosion this might be decisive. In the case of repair and reconstruction, the cooperation between old and new members should be given extra attention. One should keep in mind that strengthening members in general will only help for the additional load. For full profit of the strengthening the ductility requirement for the old structure is more important than normal. These items require a detailed and material oriented approach.

In cases of damage it is necessary that the damage can be explained by analysis. Only if the present status of the structure can be simulated from construction, observed loads and structural data, further meaningful steps can be set [28].

If the analysis of an existing structure leads to the conclusion that the structure is not safe enough, a more advanced analysis may be of help. Sometimes the question is raised wether this can be justified. The argument is that the addtional safety hidden in normal design should be considered as an integral part of the required structural safety. This argument has some truth. If all structures would be analysed on the edge of the theoretical possibilies, the average safety would probably be lower. On the other hand, a carefull analysis may be expected to indicate the weak spots in the analysis which are normally overlooked. Anyway, the code has to come up with some statement about this matter. It is the authors' conviction that both in design and in assessment simple and advanced models can be used to meet the safety requirements. The errors made in both models should be taken care of by introducing the proper model uncertainties.

Quite another point is it wether it is allowed to include nonstructural compontens like separating walls in the analysis. This, to the authors opinion, should be done only with great care and as a kind of emergency measure.

5.5 Acceptance criteria

Criteria for acceptance of an existing structure should be based on present day codes. The mere fact that the structure fulfils the code of its time of construction can never be decisive. Codes have changed and in general for good reasons. This of course does not mean that a if a new code comes into practice with on some point more severe requirements than the old one, this should lead to immediate disapproval of all old buildings. Reasons for possible reductions of the requirments for existing structures will be discussed in this paragraph.

The possible reduction of requirements for existing structures can best be discussed in terms of a probability based code. In a probability based code the design is based on partial factors that depend on the degree of uncertainty and on the required life time reliability. Three items can be taken into consideration:

- the information (inspections, observations) that is available
- the reference period
- the cost benefit ratio of safety measures

If the assessment of an existing structure is based upon the results of extensive measurements, one may reason that the uncertainty has been reduced (see figure 1). This reduction in uncertainty may lead to the use of lower partial factors. In fact this does not mean any reduction of the real safety at all. Note further that measurements not only affect the partial factor, but also the characteristic value. This may lead to much higher values for the material properties than those to be assured in design.

A similar reasoning may be set up for the reference period. New structures are designed for a period of at least 50 years. This long period may lead to a number of requirements which are useless in the assessment of existing structures. There is no point in rejecting an existing structure because we do not expect it to last a period of 50 years from now on. Again this does not really mean a reduction in reliability.

Finally there is the item of cost. It has already been explained that the cost of improving a new structure is generally much lower than the costs involved in upgrading an existing one. This means that if the upgrading of an old structure is considered to be uneconomical, this need not be the case for a new one and vice versa. This argument may lead to a difference in requirements between new and existing structures. Of course, if the safety of human lifes is at stake, some care must be taken with this argument. We will not go into the details, but for instance see [29].

There are many examples from practice where the above type of reasoning has been followed: in many guides for existing structures reduced requirements have been introduced, for instance for fire, earth quake, and so on.

A final fundamental question is: what is an existing structure? The only logical answer can be that a structure should be regarded and judged as an existing structure, the very day that it has been erected. The criteria might even be applied for those parts that have been constructed, even if the total building is not yet completed. In combination with the proposed reduction in the requirements, this might lead to the conclusion that the contractor of a new building can produce a reduced quality without punishment. This of course cannot be true: the contractor is obliged by a contract to deliver the design quality. But if he fails, it might be better that he pays a fine than that he starts to make repairs.

6. GUIDANCE TO THE DECISION PROCESS

In section 5 of this paper the code type assessment requirements have been adressed. More than in normal cases, however, the engineer also wants guidance in the way of attacking the problems, in the decisions he has to make, even if it is of a very global nature. Typically two types of decisions have to be made by the engineer:

-one with respect to the depth of the assessment itself
-one with respect to the measures to be taken

In both cases the engineer should look for the most economical solution, keeping in mind that the total costs include assessment costs, costs of measures and the costs or benefits in the subsequent exploration of the building.

Normally it makes sense to start with a global assessment. Refinements should be done only if (1) the global assessment leads to unfavourable results and (2) the costs of the refinement are not prohibitive compared to the expected profits. Note that the expected profits depend on the probability that the outcome of the refined assessment is positive.

In the choice of possible measures, given a certain amount of information, the time aspect is very important. In general, the decison to build a new structure will lead to higher direct costs than a repair decision. In those cases, however, one should also look for the long term costs: a new building may be good for 50 years while the repair may only only solve the problem for about 10 years.

In giving guidance to these problems, a distinction should be made between the "individual assessment" and a "long term inspection and maintenance program".

The individual assessment occurs in case of damage, remodelling, and so on. A typical guidance for such cases is often provided in the form of a flowchart for activities [3, 5]. Figure 2 gives an example. The chart is based on the principles stated above.

In the long term inspection and maintenance planning one formulates the criteria for repair, maintenance, inspection periods, etc in advance. The criteria should be formulated in such a way that the total cost expectation is a minimum. To get a grip on this problem, a presentation on the basis of an event tree could be of help, see figure 3. Of course, for a quantitative optimization, the costs of inspections, repair and failures should be known, as well as the probability of failure between one inspection and the next one. Of course, in most cases the costs, let alone the failure probabilities, are only vagely known. Nevertheless a clear picture of what the ideal decision criteria are can help to improve the decision quality, even in the absence of exact data.

Figure 2: Simple flow chart for decision making in the case of individual assessment

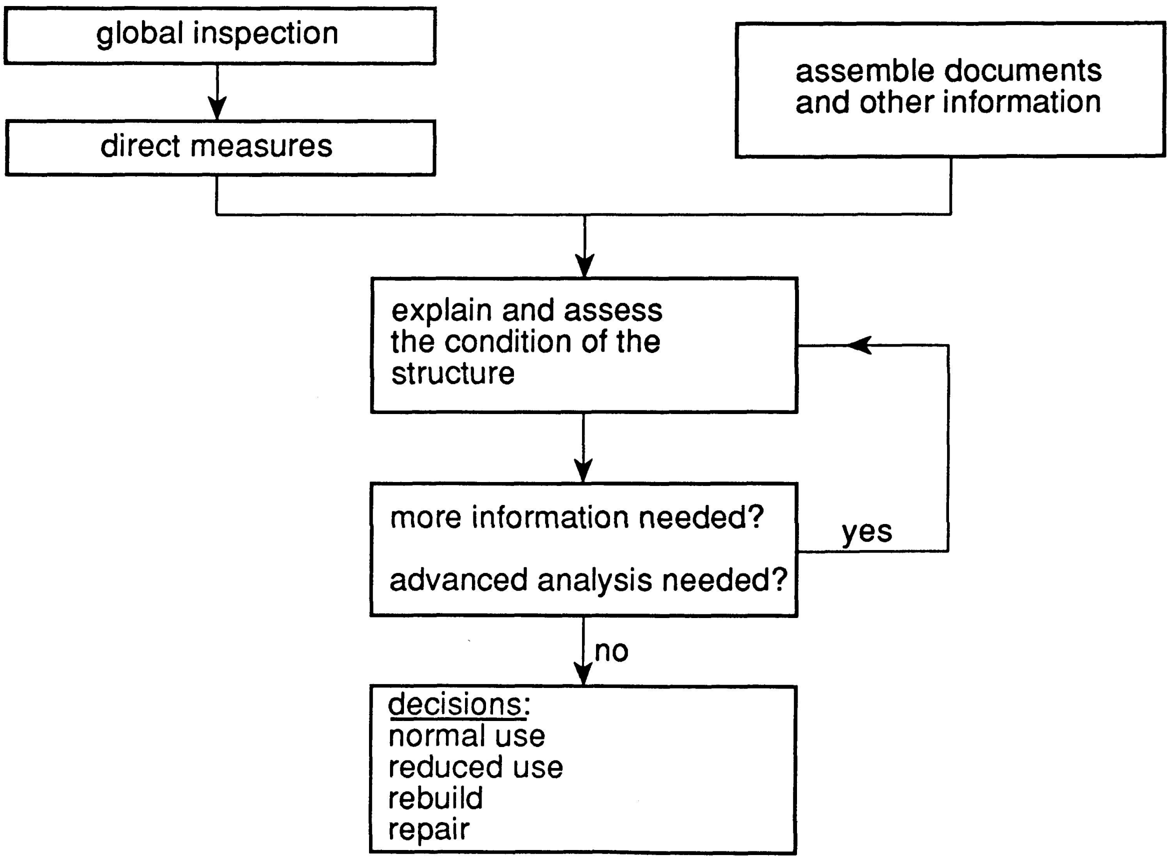
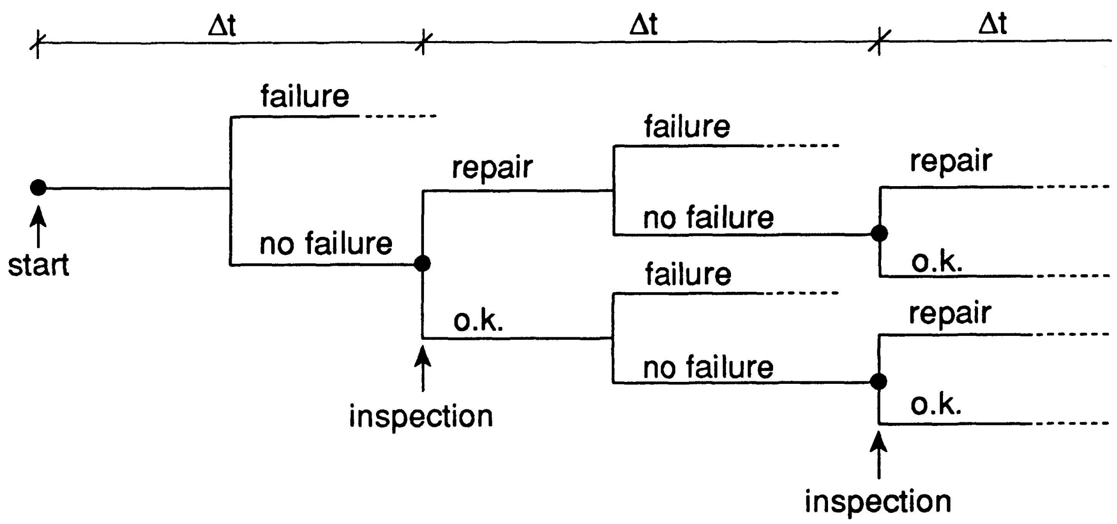


Figure 3: Event tree for a structure subject to a long term inspection and repair program; after every inspection the structure may be o.k. or need repair; during the inspection intervals the structure may fail



7. CONCLUSIONS

The codification of assessment procedures for existing structures is a relatively new field of development. Although an increasing number of codes and guidelines is published, the feeling in most countries is that making a code for existing stuctures still requires additional prenormative research. To stimulate this research this paper has tried to sum up the basic items that should be adressed by such a code. These items are:

1. Criteria to do an assessment
2. Structural properties and loads
3. Evaluation of inspection results
4. Structural analysis
5. Acceptance criteria

The most difficult items probably are the incorporation of inspection results into the total analysis and the possible reductions in performance requirements. Rational ways to deal with these problems seems possible only within the framework of probability based methods. Of course, for the application in every day practice, the results of those methods need to be translated into standard load and resistance factor procedures.

In addition to the typical code type items mentioned above, an engineer needs some guidance to judge in what circumstances what inspections or what measures should be chosen. Here a cost optimization approach should be followed. This requires a guideline type of document, telling the engineer what decisions are the most economical, given the requirements in the code and the costs of the various alternatives.

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SELECTED PAPERS

Reliability Analysis of an Existing Bridge

Analyse de la fiabilité d'un pont existant

Zuverlässigkeitsanalyse einer existierenden Brücke

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Max Karlsson has worked with practical applications of structural reliability since 1980. He is a member of the permanent Committee of the Danish Society of Engineers on Actions on Structures and he is chairman of the working group of the Danish Society of Engineers on Structures in Glass.

Johannes Morsing Johannesen got his Master's Degree at the Technical University of Denmark in 1990 and is currently PhD student. He is studying methods of reliability analysis of damaged structures.

Ove Ditlevsen has contributed to the development of the theory of structural reliability since 1961 with a large number of publications dealing with a large variety of topics. Ove Ditlevsen is a member of the Joint Committee on Structural safety and of CIB, W81: Actions on Structures.

SUMMARY

The assessment is described of the remaining structural capacity of an existing concrete bridge. A probabilistic reliability analysis is applied to a simple conventional carrying capacity model for the bridge. This simplified reliability analysis is calibrated by a random effectivity factor to give realistic results. The calibration uses some particularly chosen deterministic analyses of the bridge. These analyses are based on a refined FEM-model of the failure behaviour taking into account that the observed strength throughout the structure differs from what was assumed at the design stage. The cases for deterministic analysis are obtained through the reliability analyses of the simple model.

RÉSUMÉ

L'article traite de l'évaluation de la résistance restante d'un pont en béton armé. L'analyse probabiliste de la fiabilité du pont est réalisé sur la base d'un modèle simple de la résistance ultime du pont. Cette analyse simplifiée de la fiabilité est calibrée au moyen d'un facteur d'efficacité pour obtenir des résultats exacts et réalistes. Le calibrage utilise des résultats de certaines analyses déterministes des structures du pont. Ces analyses ont été faites en utilisant un modèle trés détaillé, par éléments finis du comportement du pont en tenant compte que la résistance observée en certaines parties du pont est différente de celles supposées lors de l'établissement du projet. L'analyse déterministe est établie sur la base de l'analyse de la fiabilité du modèle simplifié.

ZUSAMMENFASSUNG

Die Bewertung der Resttragfähigkeit einer Betonbrücke wird beschrieben. Eine Wahrscheinlichkeitsanalyse der Sicherheit ist auf der Grundlage eines einfachen Standardmodells der Tragfähigkeit der Brücke durchgeführt. Diese einfache Wahrscheinlichkeitsanalyse ist mit einem Effektivitätsfaktor kalibriert, um ein realistiches Ergebnis zu erreichen. Die Kalibrierung nutzt speziell ausgewählte deterministiche Analysen der Brücke. Diese basieren auf einem verfeinerten FEM-Modell bezüglich der Brückenkonstruktion unter Berücksichtigung von Festigkeitsabweichungen gegenüber den Bemessungsannahmen. Die betreffenden deterministischen Untersuchungen werden aufgrund des einfachen Zuverlässigkeitsmodells bestimmt.

1. Introduction

Well–developed rational reliability based methods for designing new concrete bridges are available today. However, for a number of reasons this is not the case concerning the assessment of the remaining structural capacity of an existing and deteriorated bridge.

A reliability analysis of a bridge in the design state is a formal procedure based on common practice. The models have to a certain extent become standard so that the target safety levels together with associated and selected failure modes give structural dimensions which are known to be satisfactory for normal structures. Moreover, the analytical models used in design are practically manageable in size and complexity and they are assumed to model the structural carrying properties of the bridge in a sufficiently realistic way.

A similar standard procedure for analysing existing bridge structures has not yet been developed. When considering an existing bridge the reliability analysis is no longer just a formal procedure. The potential failure modes have to be modelled realistically taking into account the available knowledge on geometry, strengths, etc. This raises the problem of how to set up such probabilistic models that are sufficiently rich in concepts to take the available information into account and at the same time can be standardized to an extent that makes the reliability analysis result comparable with the result from a similar reliability analysis of another existing bridge or with specified target safety levels.

Another problem is that the reliability analysis of an existing bridge due to observed deteriorations, errors etc. often becomes more complicated than the analysis during the design state. However, such information must be considered seriously and it makes it more difficult to set up models that are practically manageable.

In the following a method is demonstrated by which these problems can be overcome for concrete bridges suffering from severe damages.

2. The considered existing bridge

The bridge across Salpetermosevej in Hillerød, Denmark, was constructed in 1977. It is designed as a reinforced concrete frame structure. The length of the free span is approximately 6 m.

The concrete used for casting the bridge was supplied by a local plant for ready–mixed concrete and delivered by truck mixers. The workability of the concrete was very poor and too stiff for the contractor to obtain a satisfactory compaction. Thus, the hardened concrete obtained gross porosity, showing high intensity of honeycomb at the finished concrete surface and a high content of entrapped air in the interior of the structure. Furthermore, the fresh concrete even contained fractions of hardened concrete due to insufficient cleaning of the mixer. The compressive strength of the concrete was determined by cast cylinders. The test results indicated that the potential compressive strength of the concrete in the structure would be lower than prescribed, mainly due to large variability. An investigation made in 1977 with tests on drilled cores from the bridge verified this suspicion.

The present appearance of the bridge shows concrete which is seriously disintegrated by cracks and other signs of deterioration especially in the bridge deck. Due to the extensive porosity of the concrete the influence of aggressive substances from the environment is significant. The carbonation, the chloride ingress (de–icing salt) and the leaching by rainwater seeping through the concrete have been the dominating environmental actions on the concrete bridge. The effect of these attacks is a decrease of the compressive strength and a loss of protection against corrosion of the rebars in various parts of the structure. In this paper we will be content with a study of the reliability analysis of the deteriorated bridge deck.

3. The effectivity factor reliability analysis method

The probabilistic reliability analysis of the bridge deck can be made on the basis of a simple conventional upper bound yield line collapse model. In the following we will denote this model as the simple model. In order to make the results of the analysis "realistic" an effectivity factor reduction is introduced on some of the variables from which the yield moments are calculated. The effectivity factor is calculated from one or more carrying capacity results that correspond to certain optimized statically admissible stress fields. These statically admissible stress fields are obtained in a finite element model that represents a refined model of the local strength properties of the bridge deck taking into account that the yield moments vary over the deck according to some stochastic field model. This stochastic field model reflects the observed deteriorations of the bridge deck. Moreover, by using the lower bound theorem of the ideal plasticity theory searching an optimal admissible stress field it is automatically ensured that the reliability is obtained for the most critical failure mode. This finite element model will in the following be denoted as the elaborate model.

The inverted commas around the word realistic are put there because the ideal plasticity theory is not necessarily particularly realistic. However, the ideal plasticity theory is used herein in order to illustrate that a simple model by use of an effectivity factor modification can be made reliability equivalent to a far more elaborate model of the same phenomenon. The effectivity factor is obtained by a single or some few calculations with the elaborate model. The set of input values for these calculations with the elaborate model is obtained by a reliability analysis carried out by use of the simple model.

In this way an elaborate (and possibly realistic) model for the carrying properties of a structure can be reliability analysed by use of a suitably calibrated simple conventional carrying capacity model. This reliability equivalence may be the key to a rational codification of methods to evaluate remaining structural capacity. The theoretical considerations leading to the method are given in Ditlevsen and Arnbjerg–Nielsen (1992,1992). Here the method will be summarized without the argumentations for the validity of the method.

Let (x_S, x_R, x_D) be the total vector of basic variables (input variables) that are contained in the elaborate model. The subvectors x_S and x_R are the vectors of load variables and strength variables respectively. These variables are with sufficient generality defined such that they all have physical units that are proportional to the unit of force. The subvectors x_D is the vector of all the remaining basic variables (of type as geometrical and dimensionless basic variables).

Two limit state equations

$$g_r(x_S,x_R,x_D) = 0 \quad , \quad g_i(x_S,x_R,x_D) = 0 \tag{1}$$

are given representing the "realistic" model and the idealized model respectively. It is assumed that for each fixed (x_S,x_R,x_D) the equations

$$g_r(\kappa_r x_S,x_R,x_D) = 0 \quad , \quad g_i(\kappa_i x_S,x_R,x_D) = 0 \tag{2}$$

can be solved uniquely with respect to κ_r and κ_i respectively. The solutions are $\kappa_r(x_S,x_R,x_D)$ and $\kappa_i(x_S,x_R,x_D)$. By using the physical property of dimension homogeneity it can be shown (Ditlevsen and Arnbjerg–Nielsen 1993) that the two equations

$$g_r(x_S,x_R,x_D) = 0 \quad , \quad g_i(x_S,\frac{\kappa_r(x_S,x_R,x_D)}{\kappa_i(x_S,x_R,x_D)}x_R,x_D) = 0 \tag{3}$$

are equivalent in the sense that the two set of points they define are identical. The idea of the effectivity factor method is to use a suitable simple approximation to the last equation in (3) in the reliability analysis in place of the first equation in (3). The point is to approximate the effectivity factor function $\nu(x_S,x_R,x_D) = \kappa_r(x_S,x_R,x_D)/\kappa_i(x_S,x_R,x_D)$ by a constant or at most an inhomogeneous linear function of (x_S,x_R,x_D). The approximation is made such that it is particularly good within the region of the space that contributes the most to the failure probability. Let (x_S^*,x_R^*,x_D^*) be a point of this region and let $\nu^* = \nu(x_S^*,x_R^*,x_D^*)$. The equation

$$g_i(x_S,\nu^* x_R,x_D) = 0 \tag{4}$$

then defines an approximating limit state in the important region. The problem is now reduced to the problem of how to choose the point of approximation (x_S^*,x_R^*,x_D^*) . The answer to this problem is given in the reliability theory. With a judgmentally chosen value ν_0 of ν^* a first or second order reliability analysis (FORM or SORM, see e.g. Madsen, Krenk, and Lind (1986) or Ditlevsen and Madsen (1991)) is made with (4) as limit state. This analysis determines the most central point (the design point) (x_{S1},x_{R1},x_{D1}) and an approximate failure probability p_1 . Using that $\kappa_i(x_{S1},x_{R1},x_{D1}) = 1/\nu_0$ an improved value $\nu_1 = \nu_0\kappa_{r1}$ of ν^* is calculated where $\kappa_{r1} = \kappa_r(x_{S1},x_{R1},x_{D1})$. Then a new FORM or SORM analysis is made with (4) as limit state. This gives the most central point (x_{S2},x_{R2},x_{D2}) and the approximate failure probability p_2 . Proceeding iteratively in this way we get a sequence $(\kappa_{r1},p_1),(\kappa_{r2},p_2),\ldots$ that may or may not be convergent. If the sequence is convergent in the first component it is also convergent in the second component and we have $\kappa_{r1},\kappa_{r2},.. \to 1$, $p_1,p_2,.. \to p$ where p will be denoted as the *zero order approximation* to the probability of the failure event of the elaborate model.

If the sequence is not convergent we still can define the zero order approximation by simple interpolation to the value $\kappa_r = 1$ among points (κ_r,β) $(\beta=-\Phi^{-1}(p)$, Φ = standardized normal distribution function) corresponding to the sequence or simply obtained for a series of different values of ν_0 .

A check of the goodness of the zero order approximation is made by replacing the effectivity factor function $\nu(x_S,x_R,x_D)$ by its first order Taylor expansion

$$\nu(x_S,x_R,x_D) = \tilde{\nu}(x_S,x_R,x_D) \simeq \nu^* + \mathbf{a}'(x_S-x_S^*) + \mathbf{b}'(x_R-x_R^*) + \mathbf{c}'(x_D-x_D^*) \tag{5}$$

at the most central point (x_S^*,x_R^*,x_D^*) corresponding to the limit state (4) with ν^* being the effectivity factor value corresponding to $\kappa_r = 1$. The numerical determination of the coefficients $\mathbf{a}, \mathbf{b}, \mathbf{c}$ requires that the values of $\nu(x_S,x_R,x_D)$ are known at least at as many points in the vicinity of (x_S^*,x_R^*,x_D^*) as the number of variables in (x_S,x_R,x_D) . These values of ν are obtained by solving the equations (2) with respect to κ_r and κ_i respectively at each chosen point (x_S,x_R,x_D) .

With (5) substituted for κ_r/κ_i into the last equation in (3) we get a limit state for which both the probability of failure and the value of κ_r in general will be different from the probability p and the value $\kappa_r = 1$ as obtained by the zero order approximation. However, by a unique scaling factor k_r on the load vector x_S we can achieve that the limit state defined by $g_i(k_r x_S, \nu(k_r x_S,x_R,x_D)x_R,x_D) = 0$ corresponds to the failure probability p . With the Taylor expansion (5) substituted into this equation we get the limit state equation $g_i(k_r x_S, \tilde{\nu}(k_r x_S,x_R,x_D)x_R,x_D) = 0$ for which we can determine k_r by iterative application of FORM or SORM analysis such that the corresponding failure probability becomes p . The pair (k_r,p) will be called the *first order approximation.* The size of the deviation of k_r from 1 can then be used to judge the accuracy of the zero order approximation. Also the change of the most central point contributes to this judgment.

4. Reliability analysis of deteriorated concrete bridge

The slab structure of the Salpetermosevej bridge is shown in Figure 1. The slab is one span and clamped in both ends. Actually the slab is skew with skew reinforcement, but the skewness is relatively small – the angle between a free edge and a clamped edge is 82^0. Finite element calculations verify that assuming orthogonal reinforcement and a rectangular slab shape gives a small relative error on the load carrying capacity. Only one load case with fixed load is considered in the present study. According to the rules for loads on Danish road bridges, Vejdirektoratet (Danish Road Directorate) (1984), the critical truck load on the undamaged slab structure is found to consist of two trucks as shown in Figure 1. A uniformly distributed load is also prescribed but is found to be negligible in this case. For reference later (when reliability index versus load parameter curves are found), it is mentioned here that the load parameter value corresponds to the prescribed characteristic load including corrections for dynamic loading. In order to reduce the computational efforts in this illustration the analysis is made solely on one half part of the slab structure utilizing the geometrical and loading symmetry, Figure 1 . This is made possible by prescribing the torsional moment to be zero along the symmetry line in the finite element model. (From a stochastic modelling point of view this symmetrization is not necessarily correct).

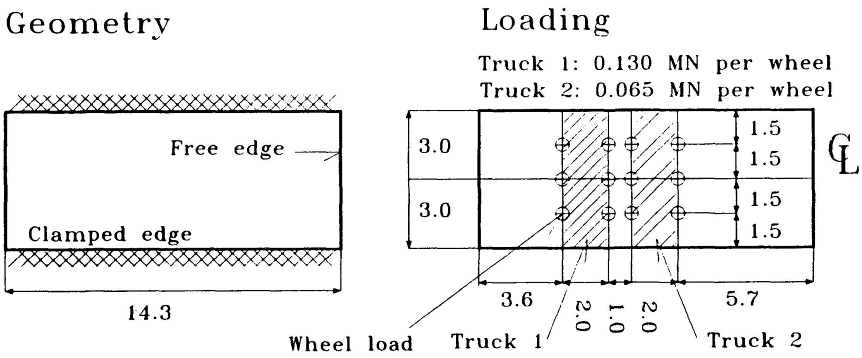

Figure 1. The slab structure of the Salpetermosevej bridge. Length units = [m].

In the elaborate model a lower bound solution is used. The analysis method is based on the lower bound theorem, which states that stress fields in equilibrium not violating the yield criterion are admissible solutions. The solution method is to find the stress distribution that maximizes the load obtained by proportional loading. Polygonization of the yield condition leads to a linear programming problem. A stress based finite element code is used, Høyer (1989). The FE code is described shortly in the following. The stress state is given by a set of stress parameters that always satisfy the local equilibrium conditions in an element, here a triangular 3–noded element. Global equilibrium is obtained in the system of nodal forces that correspond to the stress parameters and which are in equilibrium with the external nodal forces. The polygonized yield condition is checked at each node. The polygonized yield condition is given as

$$m_{xy} = \pm \min\{m_{F_x} - m_x, -m'_{F_x} + m_x, m_{F_y} - m_y, -m'_{F_y} + m_y\} \tag{6}$$

in which $m_x \in [-m'_{F_x}, m_{F_x}]$ is the moment per length unit in a cross section perpendicular to the x–axis and corresponding to compression in the upper side, $m_y \in [-m'_{F_y}, m_{F_y}]$ is defined analogously, while m_{xy} is the torsional moment per length unit. Lower index F indicates yield capacity (absolute value) and prime indicates compression in the lower side of the slab.

It is noted that the polygonized yield surface (6) is inside the yield surface defined by

$m^2 = \min\{(m_{F_x}-m_x)(m_{F_y}-m_y), (m'_{F_x}+m_x)(m'_{F_y}+m_y)\}$ the latter being the standard yield surface for reinforced concrete slabs. Thus the polygonized yield surface leads to a lower bound solution as compared to the usual solution.

The simple model is based on the upper bound theorem in the theory of plasticity for ideal plastic materials. The work equation method is used, e.g. Nielsen (1984). A simple expression for the load carrying capacity of the undamaged homogeneous slab structure is set up as follows. The yield line pattern is shown in Figure 2. The fixed length of the positive yield line in the middle of the slab and the assumptions $d/x < 1/2$ and $x < d+b_T$ are found to be reasonable for the strength values of the undamaged slab structure. The load parameter λ is then given by (for symbols see Figure 2) $\lambda P = (\alpha x^2+\beta x+\gamma)/(10x-d)$ where $\alpha = 8(m_{Fy}+m'_{Fy})/b$, $\beta = 4b_T(m_{Fy}+m'_{Fy})/b$, $\gamma = 2b(m_{Fx}+m'_{Fx})$. The optimal value of x is the relevant solution to the equation $11\alpha x^2-4\alpha dx-11\gamma-2\alpha d = 0$.

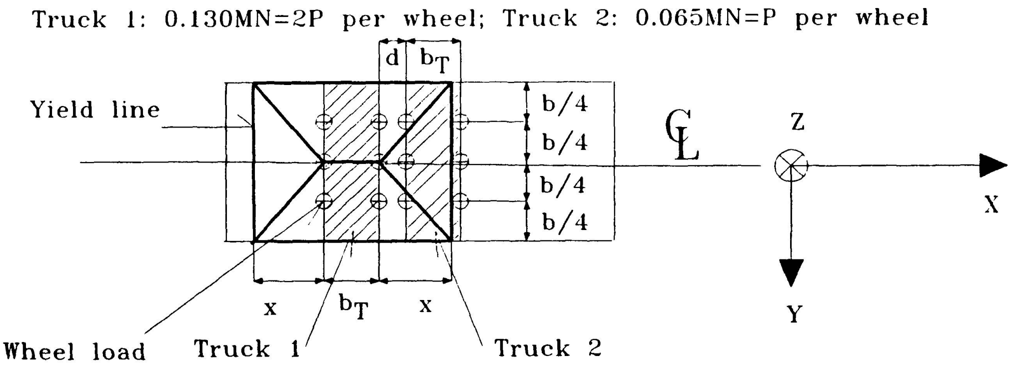

Figure 2. Yield line pattern in upper bound calculation.

For the damaged structure the slab is nonhomogeneous. However, the same yield line pattern is used for optimization of the load parameter with respect to x . The internal work is calculated approximately corresponding to the moment capacities in the different zones that model the damages of the slab structure.

The concrete strength is assumed constant over the thickness of the slab. Concrete covers and reinforcement areas are considered deterministic before the occurrence of damages. The yielding force of the reinforcement is used directly in the reliability analysis. The variables are: f_c = concrete strength, F_x, F_y = yield force per length unit in lower reinforcement in the x–direction and the y–direction respectively, F'_x, F'_y = yield force per length unit in upper reinforcement in the x–direction and y–direction respectively, d_x, d_y = effective depth of lower reinforcement in the x–direction and the y–direction respectively, d'_x, d'_y = effective depth of upper reinforcement in the x–direction and the y–direction respectively.

As it is stated earlier, the finite element code is formulated in cross sectional moment capacities (per length unit). The bending moment capacities are calculated as for a normally reinforced beam, i.e. it is assumed that the reinforcement in tension is yielding at failure. The assumption is reasonable in a deterministic analysis considering fixed values only, for example characteristic values. In a reliability analysis one or more of the input variables can take such values in the tails of their respective distributions that other than the assumed modes of bending failure can occur. This matter is not persued further in this study. Neglecting reinforcement in compression, the moment capacities $m_{F_x}, m'_{F_x}, m_{F_y}, m'_{F_y}$ are given by a formula of the form $m_F = (1-\Phi/2)\Phi d^2 f_c$, $\Phi = F/(df_c)$ with the relevant indices x or y and no prime or prime put on all the symbols m_F, Φ, d, F .

In the treated problem, the expressions (2) become $-\kappa_r x_S + \lambda_r(x_R,x_D) = 0$, $-\kappa_i x_S + \lambda_i(x_R,x_D) = 0$ where $\lambda_r(\cdot)$ and $\lambda_i(\cdot)$ are the carrying capacity functions corresponding to the elaborate and simple model, respectively. Hence the effectivity factor function simplifies to $\nu(x_S,x_R,x_D) = \lambda_r(x_R,x_D)/\lambda_i(x_R,x_D)$ showing that the effectivity factor is independent of the load. This gives the simplification relative to the general problem that solving the equation in (2) with respect to κ_r and κ_i requires only one calculation of $\lambda_r(x_R,x_D)$ and $\lambda_i(x_R,x_D)$, respectively. Furthermore, derivatives with respect to the load variable need not to be calculated.

Example 1: Corroded reinforcement. Minor cracking of the concrete Corroded reinforcement and minor cracking of the concrete can be caused by chloride ingress. If only minor cracking with no sign of corrosion at the surface of the concrete has occurred, the corrosion is normally either limited or it has the character of pitting. Pitting can lead to a total loss of strength in a section. It is assumed in this example that severe corrosion damages are observed in a relatively large zone.

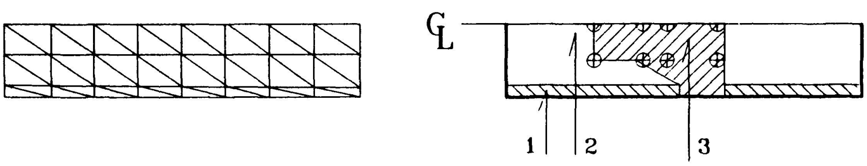

Figure 3. Finite element mesh and zones corresponding to damages (half of the structure).

The damage zones are shown in Figure 3. The damage zones are chosen to be the same as the damage zones for the actual slab of the Salpetermosvej bridge, treated in the next example, except that a much larger reduction of the lower reinforcement is assumed in zone 3. Zone 1 along the clamped edge is considered to be undamaged. The concrete strengths in the zones 2 and 3 are reduced by multiplying f_c by the random variables R_{fc2} and R_{fc3} respectively. Analogously the reinforcement areas and thus the yielding forces in the lower side in zone 3 are reduced by the factors R_{Fx3} and R_{Fy3} .

The variables entering the problem and distribution assumptions are shown in Table 1. The units correspond to [m] and [MN]. Other geometrical properties of the slab are taken to be constant.

Name	Distribution	Fixed value	Mean	C.o.v.
Load parameter	Fixed	Varying	-	
f_c	Lognormal	-	30.0	0.15
F_x, F'_x	Lognormal	-	0.3848	0.05
F_y, F'_y	Lognormal	-	0.8747	0.05
d_x, d'_x	Fixed	0.245	-	-
d_y, d'_y	Fixed	0.261	-	-
R_{fc2}	Lognormal		0.9	0.20
R_{fc3}	Lognormal		0.8	0.20
R_{Fx3}	Uniform:[0.4,0.6]		-	-
R_{Fy3}	Uniform:[0.3,0.5]		-	-
all random variables are assumed to be mutually independent				

Table 1 *Data for the reliability analysis. (C.o.v. = coefficient of variation)*

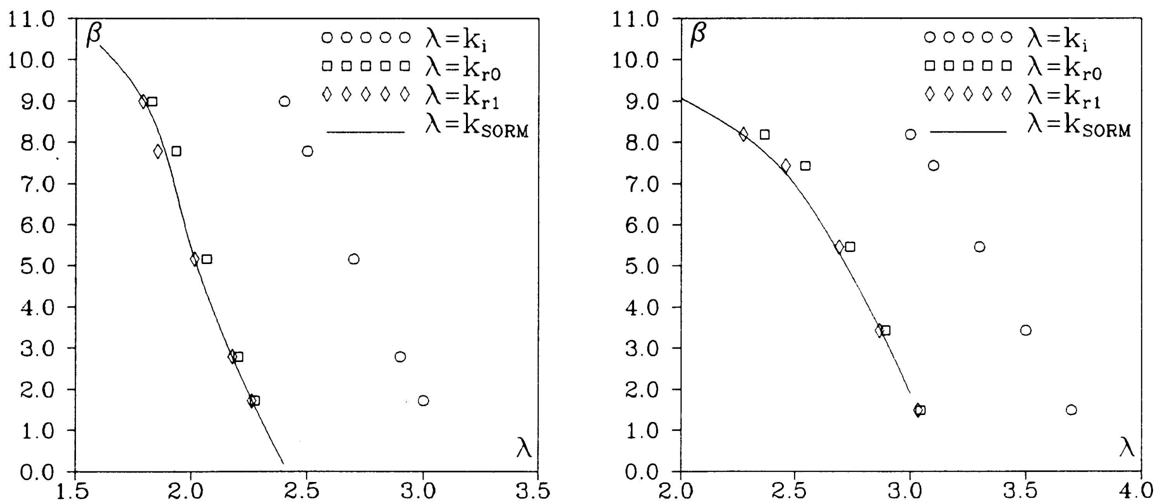

Figure 4. Reliability index versus load parameter in Example 1 (left) and in the Example 2: Salpetermosevej (right).

Corrections of the upper bound solution for a homogeneous slab are obtained by replacing α, β, and γ by (1, 2 and 3 refer to the zones as defined in Figure 3) $\alpha = 4(2m'_{Fx1}+m_{Fy2}+m_{Fy3})/b$, $\beta = 4b_T(m'_{Fy}+m_{Fy3})/b$, $\gamma = b(m_{Fx2}+m'_{Fx2}+m_{Fx3}+m'_{Fx3})$. The results from the reliability analysis are shown in Figure 4 (left). The load parameter $k_i = 1/\nu^*$ corresponds to the simple model, whereas the load parameters k_{r0} and k_{r1} correspond to the effectivity factor method calculation with a constant and a first order Taylor expansion of the effectivity factor, respectively. The fully drawn curve comes from a direct SORM analysis for the elaborate model, i.e. the finite element model. It is seen that even with the large deviations between the results of the simple model and the elaborate model the effectivity factor method yields a quite good agreement between the zero order approximation results and the results from the elaborate model. Furthermore it is seen that the agreement is improved by using the first order approximation.

Example 2 Salpetermosevej Example 1 corresponds to the slab structure of the Salpetermosevej bridge with deliberately overestimated reinforcement reductions. In this example the data are the same as in Example 1 except for the random variables R_{Fx3} and R_{Fy3}, i.e. the reduction factors of the yield forces in the lower reinforcement in the zone 3 in the directions x and y respectively. Here these reduction factors are assumed to be uniformly distributed between 0.9 and 1.0, that is, less severe reductions are assumed. Measurements of the present properties of the bridge have not been carried out but the bridge has been inspected visually. Based on engineering judgments it is anticipated that the assumed data very well can be valid for the bridge. The results from the reliability analysis are shown in Figure 4 (right). As in Example 1 there is good agreement between the effectivity factor method and results from the direct SORM analysis of the elaborate model.

Acknowledgement

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Residual Service Life of Concrete Structures
Vie résiduelle des structures en béton armé
Restnutzungsdauer von Betontragwerken

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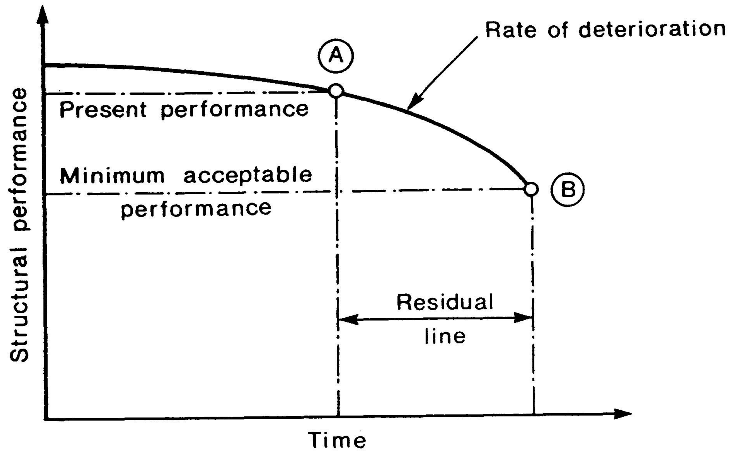
SUMMARY
This paper outlines the nature and scope of a project entitled "The residual life of reinforced concrete structures". The deterioration mechanisms covered by the project are: reinforcement corrosion, freeze-thaw damage and alkali-silica reaction. Preliminary ideas are outlined on the options avaliable for developing a deterministic model to predict future strength and serviceability, while taking account of variable client needs in terms of minimum technical performance and of future management strategy for the structure.

RÉSUMÉ
L'article décrit la nature et les objectifs du projet "La vie résiduelle des structures en béton armé". Les mécanismes de détérioration traités dans le cadre du projet sont: la corrosion de l'armature, les dommages infligés par le cycle gel-dégel et la réaction au silicate alcalin. Les auteurs offrent des idées préliminaires sur les options disponibles pour l'élaboration d'un modèle déterministe, qui servira à prédire la résistance et le potentiel d'utilisation ultérieurs, compte tenu des exigences diverses des clients en matière de performances techniques minimum admissibles et de la stratégie de gestion future des structures concernées.

ZUSAMMENFASSUNG
Der vorliegende Aufsatz umreisst Art und Umfang des Projektes über "Die verbleibende Lebenserwartung von Stahlbetonbauwerken". Das Projekt deckt die folgenden Verfallsmechanismen ab: Korrosion der Bewehrung, Schädigung durch Frost-Tau-Wechsel und Alkali-Silika Reaktion. Es werden vorläufige Ideen bezüglich der Optionen aufgezeigt, die für ein deterministiches Modell zur Vorhersage künftiger Festigkeit und Nutzbarkeit zur Verfügung stehen. Gleichzeitig werden die unterschiedlichen Anforderungen der Klienten hinsichtlich minimaler technicher Leistung und künftiger Managementstrategien für das Bauwerk berücksichtigt.

1. INTRODUCTION

When a decision is taken to appraise a deteriorating structure - perhaps based on input from routine inspection - the client will generally want a simple answer to the simple question "Is the structure safe and serviceable now, and how long will it remain so in the future ?". In posing this question, he will most probably have established his minimum acceptable requirements for technical performance - based on his future plans for the structure, primarily for financial and functional reasons - and will want to know how long it will be before those may be reached [with or without interim repairs], i.e. the residual service life.



The situation is shown schematically in Figure 1. The problem is to establish the performance at Point A, and how long it will take to reach point B. Point B itself requires definition; this will depend on future client needs, while also taking account of the levels of safety and serviceability provided in the original design.

Fig. 1 Schematic illustration of assumed behaviour

The essential steps in the process, and the links between them, are shown in Figure 2. In short, where one or more specific deterioration mechanisms are involved, these have first to be identified, their effects quantified, and the results integrated into the procedures usually adopted for conventional structural appraisal.

A number of differences between assessment work and original design should be noted. The most important of these are:

- material properties can be measured, not assumed
- dead loads can either be measured, or otherwise determined with some accuracy
- more realistic estimates of live load can be made
- more rigorous analytical methods can be used, while taking account of interactions between all elements, and possibly establishing alternative load bearing mechanisms
- the relative importance of load effects may be different. While basic effects such as bending, shear and compression will have to be checked, actions such as anchorage and bond can become more important when deterioration is involved.
- the establishment of the likely future environment [especially micro-climate] is of critical importance, since it will influence future deterioration rates.
- lower factors of safety, and smaller "margins", can be used, because of the greater knowledge of the structure - compared with the assumptions made in design.

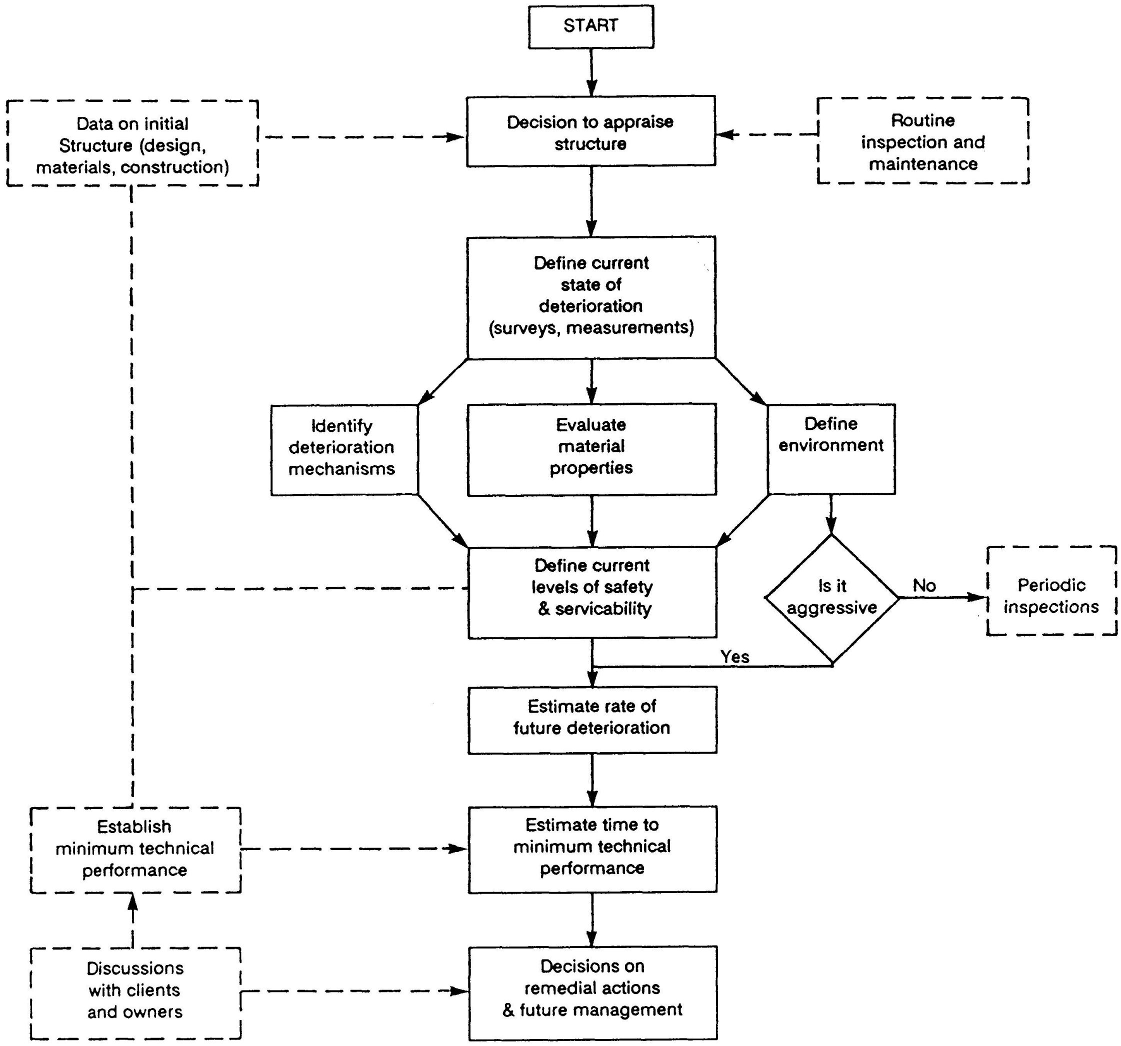


Fig. 2 Steps involved in assessing structures and residual service life

2. BRIEF DESCRIPTION OF BRITE PROJECT BREU-CT92-0591

This 3-year project, which started officially on 1/3/92, is concerned with developing a quantifiable system to enable residual service life to be evaluated in accordance with Figures 1 and 2. The specific deterioration mechanisms to be covered are: reinforcement corrosion; freeze-thaw; alkali-silica reaction; with their effects considered singly and in combination.

While the project is concerned with all the steps identified in Figure 2, particular emphasis is placed on certain factors, where new data or a new approach is required; these are:

(a) definition of the aggressivity of the environment. This has to be appropriate for each deterioration mechanism, while allowing for both macro- and micro-climate.

(b) assessment of current material properties in the structure. Basically, this is concerned with test methods for all relevant material properties - and the interpretation of data from these.

(c) definition of current levels of safety and serviceability. This is a modelling process, taking account of the output from (b) above, other known characteristics, and modern methods of analysis.

(d) assessment of future deterioration rates. Using stage (c) above as a starting point, while augmenting the data from stages (a) and (b), future deterioration rates are assessed.

(e) definition of minimum acceptable technical performance. As indicated in Figure 2, this depends not only on the output from stages (a)-(d) above, but also on input from clients regarding future performance requirements.

Extensive experimental work is planned for stages (a)-(d) above, but probably the key to the whole project is stage (e), where discussions are necessary with clients/owners to establish a range of future needs for different types of structure [e.g. nuclear reactors v bridges v domestic housing]. Finally, to verify the applicability of the whole approach in practice, a series of case studies are planned; this means that, with the co-operation of clients, our system of assessment will be used on structures which have been examined previously and decisions taken on their residual life and future management. When all of this work has been completed, the entire approach will be produced in the form of a Manual, for convenient every-day use in practice.

3. CURRENT STATUS OF THE PROJECT

Initially, most effort is being concentrated on obtaining experimental data. For each deterioration mechanism, research is well advanced in relating changes in material properties to a wide range of environmental conditions, since the realistic classification of aggressive environments is seen as a high priority. The starting point is a classification for reinforcement corrosion, concentrating on local conditions close to, and within, the structural concrete cover. Separate conditions are foreseen for corrosion due to carbonation and to chlorides, with chlorides being further sub-divided into de-icing salts and sea water. Conditions for corrosion initiation and propagation will be treated separately.

Moisture conditions in the concrete are seen as being critical. When the classification is subsequently extended to cover freeze-thaw and ASR, this will become all-important, since different moisture conditions will be critical (see, eg. Table 1, taken from the CEB Recommendations on durability[1]). Nevertheless, a quantitative approach is considered feasible, and the proposed classification for corrosion will be finalised before the end of 1992.

Effective relative humidity	Process				
	Carbonation	Corrosion of steel in concrete which is		Frost attack	Chemical attack
		carbonated	chloride contaminated		
Very low [< 45%]	1	0	0	0	0
Low [45-65%]	3	1	1	0	0
Medium [65-85%]	2	3	3	0	0
High [85-98%]	1	2	3	2	1
Saturated [> 98%]	0	1	1	3	3

Risk : 0 = not significant 1 = slight 2 = medium 3 = high

Table 1 Significance of moisture state in influencing different durability processes

Further experimental work is in hand on corrosion, concentrating initially on the relationships between levels of corrosion and both cracking and bond; information here is important in assessing levels of structural safety and serviceability.

Corresponding experimental programmes already exist for the other two deterioration mechanisms - freeze-thaw and ASR. These data will be obtained early in the 3-year programme, since considerable effort will then be required, in deriving assessment procedures for the combined effects of the three deterioration mechanisms - an important, and original, part of the project.

In parallel to this activity, preliminary ideas are being formulated for the overall framework necessary for the assessment of residual service life, while satisfying the basic need in Figure 1 and concentrating on the steps in the bottom half of Figure 2. The factors involved, and preliminary development work are described below.

4. PRELIMINARY DEVELOPMENT OF OVERALL PROCEDURES FOR ASSESSING RESIDUAL LIFE

4.1 Some basic principles

(a) A viable assessment method is made up of a number of essential and interacting elements; these are:

- a behavioural model
- criteria defining satisfactory performance
- loads under which these criteria should be satisfied
- relevant representative material properties
- factors or margins to take account of vagaries and variability in the system, and to simplify its application in practice.

There has to be a proper balance between all the elements, in deriving the reliability of the method. That overall reliability will generally [but not always] be the same as that for the original design - but calculated in a different way, using more accurately defined input.

(b) In principle, material deterioration should not be regarded as a state [to which limits might be set] from a structural point of view, but rather as something which can lead to an unsafe or unserviceable state. Its influence on individual elements and structures is what has to be assessed, in relation to defined action effects [e.g. bending, shear, compression, bond for the different regions shown in Figure 3, for a simple beam.]

(c) Element characteristics change at different rates with time, as shown schematically in Figure 4. In determining current levels of safety - e.g. at point A in Figure 4 - and in predicting future deterioration, there is a need for awareness that other mechanisms may subsequently interfere and become more critical, e.g. anchorage and bond in relation to Figures 3 and 4.

(d) Different load carrying mechanisms [see Figure 3] may have different deterioration rates, and different levels of minimum technical performance, depending on the acceptable risks for each [see Figure 5].

(e) A viable assessment method for residual service life should satisfy three essential criteria:

- the prediction of residual <u>strength</u>, with known acceptable safety factors, while properly representing the effects of the deterioration mechanisms involved,
- the meeting of future client needs, in terms of function and serviceability;

- incorporation of sufficient flexibility, to address the characteristics of individual structures, while being underpinned by a robust theoretical approach.

The attraction of a simple and pragmatic approach is perhaps a fourth criterion.

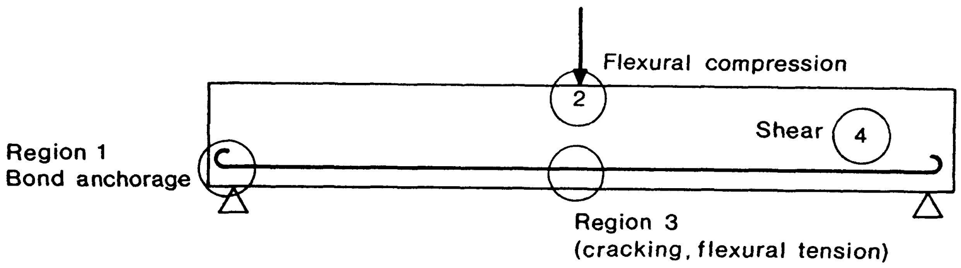


Fig. 3 Possible critical regions in a simple beam, due to deterioration

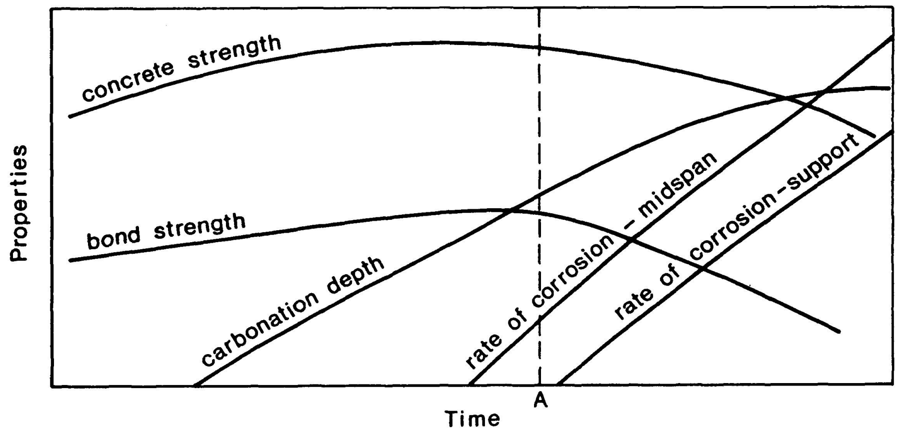


Fig. 4 Schematic change in element characteristics with time

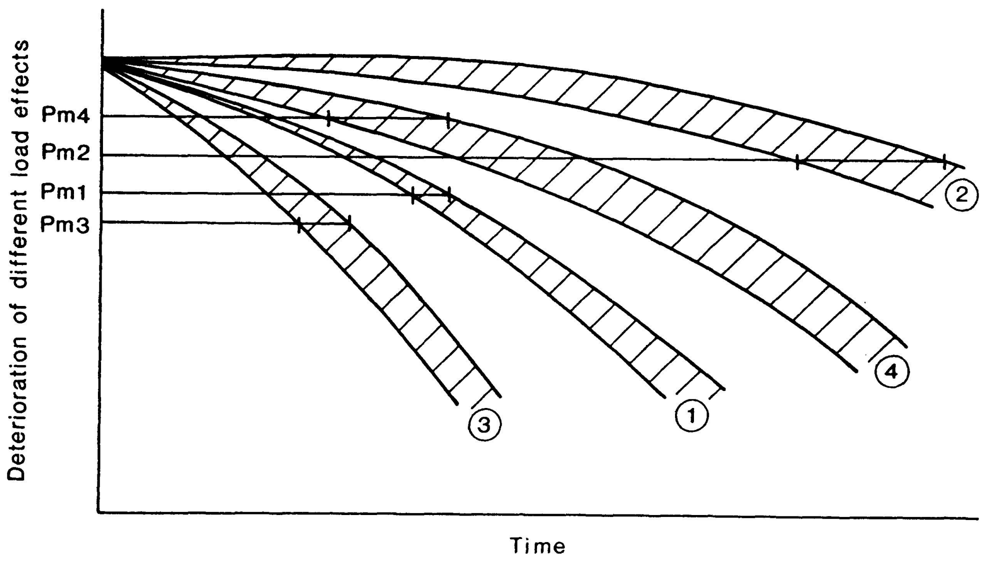


Fig. 5 Possible different deterioration rates for the mechanisms shown in Figure 3. Each may have different minimum technical performance requirements, depending on risks [P_{m1} - P_{m4}]

4.2 Available general methods of assessment

Most theoretical approaches can be put in one of two categories:

- damage classification methods, e.g. [2]
- reliability analysis, e.g. [3] [4], [5]

4.2.1 Damage classification methods

Based on condition surveys, the usual approach is to classify individual structures into one of a number of defined categories, having looked at the materials and elements, as well as the structure as a whole. The cause of the deterioration is determined, and the effects [in mechanical, physical, chemical and biological terms]. Normally, the end result is a rating system, with recommended actions associated with each rating. The discipline associated with this system has considerable merit, in terms of testing and evaluation of data.

4.2.2 Reliability methods

This favoured approach involves extending existing techniques, used in developing design methods for Codes: here, safety in assessment terms can be more easily related to that in the original design. Engineering perspectives are provided by Schneider[6] and Fagerlund[7].

4.2.3 General

Few papers and reports address the difficult question of minimum acceptable technical performance in a quantitative way: there are exceptions, e.g. [8] [9].

4.3 Practical considerations

The service lives of essentially similar structures, under apparently similar conditions, may vary considerably. There may be many reasons for this, but two key points emerge from the literature:

- some structures are inherently more vulnerable to deterioration due to their design concept and the quality of the detailing. For example, in making recommendations on the structural effects of ASR, the Institution of Structural Engineers[10] found it necessary to define the sensitivity of the structure and individual elements in three classes which reflected the potential resistance to any defined level of "load", and its associated expansion.
- if structural collapse is considered to be an extreme case of a deteriorating structure, then a study of these cases where lack of durability has contributed, e.g.[11] reveals that failure is due to a combination of factors some of which are not amenable to mathematical modelling. Again, this suggests the inclusion of a factor which reflects the quality of the original design and construction.

5. CONCLUDING REMARKS

It will be obvious that BRITE PROJECT BREU-CT92-0591 is still at an early stage. Experimental work is not finished; the development of an approach to assessment is only at a preliminary stage. What is important are the principles involved in that approach. Priority is given to meeting the needs of individual clients for individual structures, in a practical way. Nevertheless, the approach has to be soundly based technically, in accounting for the effects of the deterioration mechanisms, singly and in combination. The BRITE partners would particularly welcome comments, especially from clients and fellow professionals, both now and as the work progresses.

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Definition of Load Spectra
Définition de spectres de charge
Definition von Einwirkungsspektren

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SUMMARY
Knowledge of actions is essential for structural safety assessment. Most of the actions are random and require probabilistic models and definition of load spectra for the structural design and reliability evaluation. A review of the most important actions is presented, with information about the available data and probabilistic models now sufficiently tractable for engineering purposes. As an example, the traffic loads on bridges defined in Eurocode 1 are briefly presented. Finally some indications are given on the combination of random actions such as temperature and traffic loads on bridges.

RÉSUMÉ
La connaissance des actions est essentielle pour l'évaluation de la sécurité des structures. La plupart des actions sont aléatoires et nécessitent des modèles probabilistes de spectres de charge pour le dimensionnement et l'évaluation de la fiabilité des structures. Un panorama des principales actions présente données et modèles probabilistes actuellement accessibles aux ingénieurs. Les charges de trafic sur les ponts, traitées dans l'Eurocode 1, fournissent un exemple. Enfin quelques indications sont fournies sur les combinaisons d'actions comme le vent ou la température avec les charges de trafic.

ZUSAMMENFASSUNG
Die Kenntnis der Einwirkungen ist entscheidend für die Bewertung der Tragsicherheit. Die meisten Einwirkungen sind zufallsverteilt und erfordern Modelle der Wahrscheinlichkeitsrechnung und die Definition von Belastungsspektren für die Bemessung und Bewertung der Zuverlässigkeit der Tragwerke. Ein Überblick über die wichstigsten Einwirkungen stellt Daten und Wahrscheinlichkeitsmodelle vor, die zur Zeit den Ingenieuren zugänglich sind. Die Verkehrlasten auf Brücken, die im Eurocode 1 behandelt werden, geben ein Beispiel dafür. Und schliesslich werden einige Angaben zu kombinierten Einwirkungen wie Wind 2oder Temperatur und Verkehrlasten mitgeteilt.

1. INTRODUCTION

Knowledge and modelling of the actions on structures are a basic step in the structural safety and reliability assessment. Most of the actions on structures, either natural or due to human activities, are random and impossible to forecast. For about 20 years, many research works were undertaken on probabilistic models of actions, in order to describe and quantify their occurrences and intensities or to compute the load effects, stresses and strains induced in the structures. Many papers were published by the ASCE, WCSE, OMAE, IFIP, Structural Safety, CEB, ICOSSAR and ICASP, IABSE etc..., and some state of the art reports are prepared by the CIB, Commission W81 "Actions on Structures" [1.a to 1.j].

Then engineers discovered the great advantages of using such probabilistic models for the design and reliability evaluation of bridges, off-shore structures, buildings, nuclear power plants, and other structures. Also the experts who are writing or rewriting the new semi-probabilistic codes, such as the Eurocodes in the EC countries or the Ontario bridge code, are using such advanced methods for the calibration of the load models and the partial loads safety factors. As an example, an accurate definition of the load spectra is very important for fatigue assessment of bridges under variable vehicular loads or of off-shore jackets under wave loads.

But if the advanced probabilistic load models, based on the use of random variables or stochastic processes, are useful for accurate reliability calculations, such as those made by the FORM-SORM methods to evaluate β-indices and probabilities of failure, they are not always easy for engineers, companies, or consultants to use. Common structural design and checking are by application of codes, which mainly contains conventional load models. These load models must be well calibrated, with the help of convenient data and random models, and are often presented as load cases or load spectra.

For a given action and type of structure, the definition of load spectra generally follows the diagram:

data gathering → probabilistic model → load spectra →

→ load effects → reliability evaluation → load model calibration

2. DEFINITIONS - ACTION CLASSIFICATION AND MODELLING

2.1 Definitions

An action is an external phenomenon that induces stresses or strains in a structure. It is often expressed by a set of concentrated or distributed loads which produce load effects. An action is defined by its occurrences, the time periods of application, its spatial and directional properties, and its intensities. There are normal, abnormal or accidental actions. The correlations between several actions must be known.

The JCSS and ISO consider, depending on their occurrences:

- **permanent actions**: the period of application is the whole structure lifetime and the intensity variations are small or rare, and mainly due to changes in the structure use or affectation (e.g.: dead load).

- **variable actions**: the occurrences are discrete, the durations, intensities and other characteristics vary with time. There are some "high-frequency" variable actions (wind, wave or traffic loads), and some "low-frequency" or semi-permanent actions with slow variations (floor live loads, temperature and snow loads).
- **accidental loads**: the occurrences are exceptional, unpredictable and almost unavoidable, and they have short periods of application (e.g.: shocks, fire, explosions).

From the viewpoint dynamic load effect or structural response, there are:

- **static actions**: which do not induce significant accelerations in the structure (snow, temperature, furniture loads),
- **dynamic actions**: which produce large accelerations, vibrations, or impact forces in the structure (vehicle loads, earthquakes, wind).

2.2 Classification

Actions on structures may be classified, according to their origin, into:

- **environmental actions**: these result from natural phenomena, climatic or not, and are generally very little controlled if at all. The main climatic actions are: wind, snow, ice, rain and water currents, temperature and wave loads. The others are earthquakes and other ground motions, avalanches, etc.
- **actions due to human activities**: these result from the use of the structure and are gnerally at least partially controlled. In this category are: traffic loads on bridges and pavements, live loads in buildings, fluid pressures in tanks, etc.
- **accidental loads**: these result from human error, misuse of the structure or very exceptional and unpredictable climatic events. The vehicle collisions on parapets, ship collisions on piers, falling aircraft, explosions, fire, tornados, typhoons and tidal waves are accidental actions.

2.3 Data and models

2.3.1 General remarks

Knowledge of any action and the definition of load spectra must be based on both data and models. The data obtained by measurements of samples are necessary to choose and calibrate the models and to check their hypothesis. But they are not sufficient in most cases to fully describe an action and its characteristic values. For a time-variable random action, a set of data only gives a specified sample time history but does not represents its scattering and randomness. For example, the famous record of the El Centro earthquake provides an accelerogram which is very often used for structural design or checking, but it always reproduces the same frequency spectrum and the same magnitude and intensity.

In most problems, except for fatigue calculations, the maxima of the loads are relevant, and these maxima have to be considered over long time periods, close to the expected structure lifetimes (e.g. 50 to 100 years). It is rare to get measurements of an action time history over such long periods at the right location and under the same conditions. And even in such a case it would be not enough to derive reliable extreme value distributions or far fractiles. Then it is necessary to build good models based on verifiable assumptions with parameters which can be fitted on the available data. These data may also be used to check some of the model assumptions, e.g. by statistical tests.

2.3.2 Probabilistic models

Most of the common actions and loads on structures are random or may be considered as random because of lack of knowledge. The probabilistic models give an account for the statistical uncertainties and make possible the calculation of mean values, standard deviations, fractiles, probability of level crossing, extreme value distributions and finally probability of failure or safety indices. A "good" model must be simple enough to be tractable for calculations, compatible with the available data - and less informative, i.e. not create any additional and unverifiable information not included in the data -, but nevertheless detailed enough to give an account of the physical properties of the action. Special attention must be paid to the correlations between various loads, such as wind and waves or temperature and snow, in order to avoid gross errors in load combinations. In particular a set of n correlated random variables is only well described by an n-dimensional density called the cross-correlation probability distribution. The convenient assumption of independence must always be carefully checked.

Random time varying loads are modelled by stochastic processes, generally stationary, and some of them by random fields if there are significant space variations to be taken into account. The time-independent loads and some other loads are represented by random variables, if the time influence may be removed, above all in static or quasi-static analysis. In this case the load effects are more easily calculated, with no need for stochastic differential equations. In this common case, the notion of return period is very useful in choosing the characteristic load values:

if $X_1, X_2 \ldots X_n$ are independent random variables with the same law as X (i.e. are the random samples of X at times 1, 2, ...n), the return period T(x) of the value x of X is the mean time interval between two level crossings of x by the X_i. If F(x) is the cumulative distribution function of X, we get:

$T(x)=[1-F(x)]^{-1}$. If $\mathbb{P}(X \geq x) = p$ (small) during n time units, we have:

$T(x) \simeq -n/[Ln(1-p)]$ for any X. One must take care not to mistake the return period T(x) for the reference period n: in this way a 5% upper fractile over a reference period of 100 years has a return period of about 2000 years!
For most of the actions the characteristic values generally adopted are those having a return period between 50 and 200 years (climatic actions) and up to 1000 to 2000 years (traffic loads on bridges).

The models of extreme values commonly require the three types of asymptotic distributions [2]: type I (Gumbel) for exponentially decreasing probability distribution functions (PDF), type II (Fréchet) for geometrically decreasing PDFs and the type III (Weibull) for upper-bounded PDFs (maxima). One must be careful with multiple-peak PDF representing a mixture of several distributions, as in the case of snow loads in a maritime climate.

2.3.3 Data gathering and use

Some data are necessary to choose the PDF type of any random variable and its parameters. They may provide the basis for any statistical test of hypothesis, such as normality (Shappiro-Wilk), the choice of some fitted parameters (chi-square test) or independence.

For time independent loads, a large enough sample of data may be sufficient to derive reliable estimators of the characteristic parameters or a PDF. But for random loads varying with time, stationarity must be first checked; if not, the seasonal or periodic variations as well as the trends must be removed before any stationary model fitting. In addition the process must be ergodic (i.e. the means on time history samples converge to the statistical mean) in order to make the statistical use of the data meaningful. For periodic actions, the sampling frequency f must be adapted to the smallest period T, according to Shannon's rule: $f \geq 2/T$. A spectral analysis is often useful for variable actions (wind, waves, earthquakes, traffic loads), in order to identify the stationarity time periods and the frequency peaks of energy. A spectral representation (by the power spectral density) of the stationary processes is used in dynamic stochastic analysis (structures under seïsmic loads or off-shore jackets in random waves).

In many cases the data must be filtered when recorded to eliminate noise. The passband of the filter must be matched to the frequencies of the action. The sensitivity of the recording device must be sufficient and any bias must be eliminated or corrected. In some cases the samples are truncated and the upper tails of the PDF may be underestimated!

3. ACTION LOAD SPECTRA

3.1 Definition of a load spectrum

The improper but commonly used term of "load spectrum" means a load distribution, either derived from a sample or measurements by a statistical analysis (histogram, fig. 1) or defined by the theoretical or fitted PDF of the load parameter (e.g. intensity, fig. 2). In both cases, the load spectrum may be used to simulate randomly a time history of the load (Monte-Carlo simulation) or to calculate a load effect distribution. It is especially useful for fatigue calculations (steel bridges under traffic loads, jackets in waves, tall buildings under wind load, etc), because the whole distribution of stresses may contribute to the damage increase. A load spectrum may also be conventional, instead of being the result of a statistical or probabilistic analysis. This is the case of the code load models used for structural design or checking. Some such load models contain a set of load cases, with various intensities, each modified by a frequency or a probability.

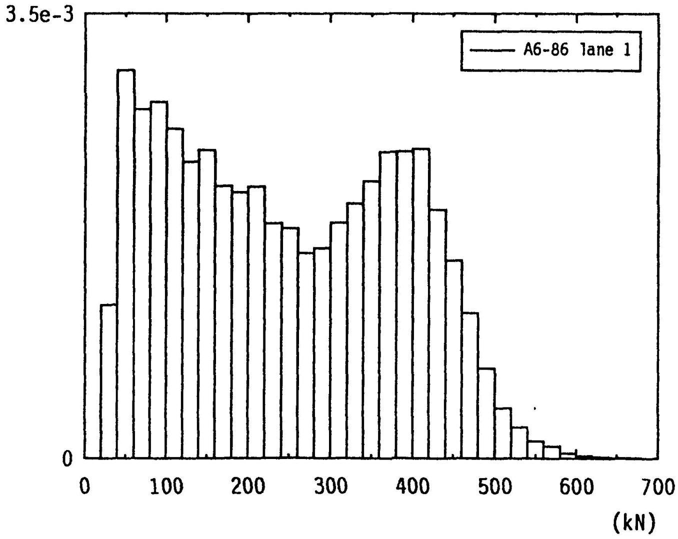

Fig.1 Histogram of the lorry weights

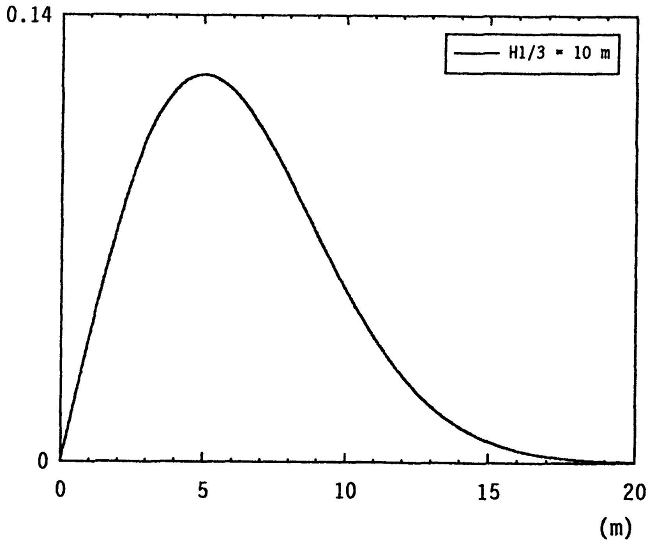

Fig.2 Rayleigh's PDF of wave heights

An example of a conventional load spectrum is given by the so-called "fatigue convoys" used for fatigue calculations of steel bridges. A "fatigue convoy" is a set of a few lorries or concentrated loads, with given gross weights, axle loads, axle and vehicle spacings and number of passages for each vehicle. For any sub-structure of the bridge, such as a cross-beam or a stiffener, the application of the load spectrum means the passage of the convoy on the bridge deck, and provides a stress variation time history. The stress cycle amplitudes may then be counted by the "rain-flow" method and the fatigue damage or the expected lifetime of the structure may be computed.

A load spectrum is generally defined by using a large sample of data (histogram) with a number of classes adapted to the sensitivity of the load effect considered. But in some cases it is derived from a theoretical model, in order to simplify the calculations.

3.2 Load spectra of climatic actions

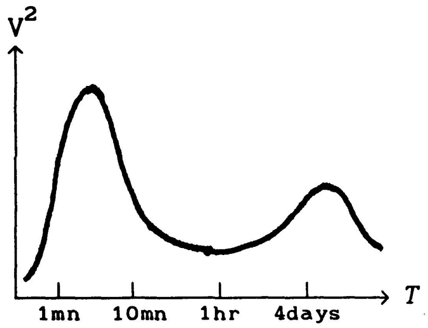

Fig.3 Wind frequency spectrum

Wind loads are applied as pressures on structures. The pressure is proportional to the square of the wind velocity. The conventional wind velocity is measured at 10 m above the ground level and increases with altitude. The peaks of the frequency spectrum are between 3 s and 10 mn and around 4 days. Because of the nonstationarity of the wind, the velocity is described by its mean values or maxima over 10 mn to 1 hr periods. These stationary periods correspond to the lower parts of the frequency spectrum (fig. 3). The velocity PDF is often described by a Weibull distribution [1.h], and the maxima over 1 to 100 years follow the type I (Gumbel) extreme distribution. The successive mean values are independent.

The wave loads on an off-shore structure or a sea wall are linked to te wave heights, which are themselves correlated with the period or wavelength. Because of the uncertainty on the wave top due to breaking, the upper 1/3 height is considered. For a given period range, a Rayleigh function is generally adopted as the PDF of this height. During short time periods of about 20 mn, the sea state is considered as stationary, giving a specified parameter a to the Rayleigh PDF, and the successive wave heights are independent.

3.3 Traffic loads on bridges

The most important design loads for bridges are the traffic loads. They are quite complex and the definition of load spectra is difficult because of the randomness of many parameters: vehicle occurrences and spacing, axle loads and gross weights. The traffic processes are generally nonstationary, correlated from one traffic lane to the next, and traffic jams or platoons of lorries occur randomly and often govern the structural design. Theoretical and numerical approaches and models have been proposed by various authors (CIB, ASCE, Eurocode) [1.f], [3].

The simplest and most common vehicular load models are built with a set of random variables; e.g. for a traffic lane: type of vehicle (from a given silhouette classification), gross weight, axle loads, spacing between vehicles and traffic flow. Vehicule length, speed, duration of jam, number of lorries in a platoon, and spacing in congested traffic may also be considered, as well as the distribution of the vehicles among lane. Correlations between some of these variables are introduced (gross weight/axle loads/silhouette/vehicle length; speed/spacing/flow, etc). Some authors have proposed more sophisticated traffic load models using random processes (Poisson and marked Poisson [4], Markov, renewal Markov [5]) in order to better describe the space distribution of the loads.

Most of these models are based on detailed mesured traffic data recorded by Weigh-In-Motion (WIM) techniques. Large samples of traffic are now available [6] providing sufficient information for model definition and calibration. Moreover these data are often rich enough (more than 30,000 lorries continuously recorded on one location) to use them directly for load effect calculations [7]. The results are then much more accurate than by Monte-Carlo simulation, especially for short and medium spans.

Load	length L	Eurocode [8]	TNO [3]
axle	-	290 kN	260 kN
tandem	-	440	410
triple	-	530	580
lorry	-	925	940
EUDL	20 m	47-56 kN/m	62 kN/m
EUDL	50	38-43	36
EUDL	100	33-38	28
EUDL	200	25-36	20

Table 1 Characteristic loads with a return period of 1000 years, slow lane.

The PDFs proposed for lorry gross weights are in most cases bimodal, with normal or Weibull modes. The speed PDFs are often normal while the spacing PDF is a gamma distribution. From these models and data, characteristic axle loads and gross weights have been derived, as well as distributed loads on a lane length. Table 1 gives some results from [4] [8]. Based on the extrapolations of the load effects having a return period of 1000 years, a conventional unified load model was built and calibrated for Eurocode 1 (Bridge loading) for the EC countries [9]. It will include also some fatigue loads.

3.4 Load combinations

The combination of random actions is a difficult problem because of the possible correlations, and in any cases because of the very low probability to observe simultaneous extreme values and the lack of cross-observations. The codes contain ψ coefficients to modify the characteristic values in order to make load combinations, but they are largely "heuristic". In any load combination it is important to know the respective variations of each component, and its frequencies. It is quite easy to combine a permanent load G with a variable one L: $\mathrm{Max}(G+L) = \bar{G} + \mathrm{Max}(L)$. If G is a quasi-permanent load or a slowly varying one, it is possile to write: $\mathrm{Max}(G+L) \simeq G' + \mathrm{Max}(L)$, where G' is a frequent value of G.

The load combinations must be studied case by case; for prestressed concrete bridges the combination of temperature and traffic loads is relevant. The temperature load (gradient) may be modelled by a sinusoidal function with a period of 24 hrs and a random amplitude, with a distribution depending on the season. The combination with the lorry loads is made by adding the bending moments due to each load, because of non-linearities in the stress calculations. A procedure using simulation and traffic records was developed for a bridge in France [10]. Some correlation exists between these loads, due to the traffic density peaks which occur at specific times in the day, like the temperature maxima or minima.

Traffic loads must be combined with the wind load for long cable-stayed or suspension bridges. This problem becomes more complicated than the previous one because both loads vary considerabily, at high frequencies. The (negative) correlation only concerns extreme wind velocities, when traffic restrictions are imposed for safety reasons.

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Fatigue Reliability Analysis of Early Steel Railway Bridges in India

Sécurité à la fatigue d'anciens ponts-rails métalliques en Inde

Ermüdungssicherheit früher Eisenbahn-Stahlbrücken in Indien

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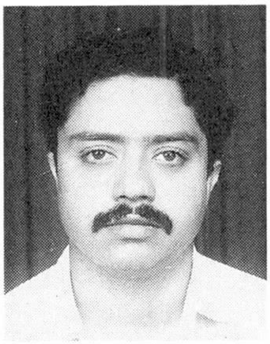

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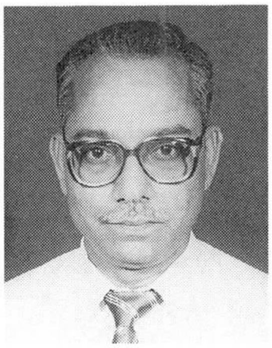

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SUMMARY

The paper deals with the fatigue reliability analysis of early steel railway girder bridges in India. Available published data on stress spectrum obtained under actual traffic load and on fatigue tests conducted on samples of the bridge members are used in the study. Limit state equation is formulated using S-N curve approach and Miner's rule. Using level II method, reliability index of girders, chords etc. are calculated. It is found that there is a linear relationship between this reliability index and the logarithm of number of cycles. The developed relationship gives insight in deciding about inspection creteria.

RÉSUMÉ

Cet article évalue la sécurité à la fatigue d'anciens ponts à poutres métalliques des chemins de fer indiens. Cette évaluation repose sur les données publiées à partir de spectres de contraintes, enregistrés sous les charges mobiles actuelles, et d'essais à la fatigue effectués sur des échantillons d'éléments porteurs des ponts. La capacité portante limite est déterminée à partir du diagramme de résistance à la fatigue et de la règle de Miner. A l'aide d'une méthode de vérification du 2e degré, l'auteur calcule l'indice de fiabilité des poutres, des membrures etc. Ce faisant , il démontre l'existence d'une relation linéaire entre cet indice et le logarithme des cycles de charge alternée. Cet indice de fiabilité fournit un aperçu des moyens de sélectionner les critères d'inspection.

ZUSAMMENFASSUNG

Der Beitrag behandelt die Abschätzung der Ermüdungssicherheit früher Stahlbalkenbrücken der indischen Eisenbahnen. Dabei stützt man sich auf publizierte Daten von Spannungsspektren, die unter Verkehr aufgenommen wurden, und auf Ermüdungstests an Proben aus Brückentraggliedern. Die Grenztragfähigkeit wird mittels des S-N-Kurvenansatzes und der Minerregel bestimmt. Mit einem Nachweis der Stufe II wird der Zuverlässigkeitsindex von Trägern, Verbänden usw. berechnet. Dabei findet man ein lineares Verhältnis zwischen diesem Index und dem Logarithmus der Lastwechselzahl, das eine Entscheidungsgrundlage für die Inspektionskriterien liefert.

1.0 INTRODUCTION

In Indian Railways, a large number of rivetted bridges which were built during the early part of this century are still in service and hence are in use for more than ninety years. With the increase in the train axle loading and their frequency of occurrence,these bridges are now subjected to relatively severe conditions and some of them have shown signs of distress. Even though most of these bridges have crossed their design life, their replacement with the new ones is not economically viable. With a rational method of evaluation, inspection and subsequent repair if needed, at regular intervals, their service life can be further extended at an acceptable risk level. A research study has been undertaken to develop a general method of reliability based design and evaluation of railway bridges. Attempts are being made to arrive at a rationally developed inspection strategy which would enable the designer to check about the possibility of service life extension of bridges.

The two key parameters required for any fatigue reliability analysis are :
a) the load spectrum or the stress spectrum and
b) fatigue test results to establish the S-N curve. Here S stands for stress range and N for number of cycles.

For getting the load spectrum, field measurements have to be done under actual traffic conditions for various members of different bridges. For conducting the fatigue tests, samples have to be taken from early steel girder bridges. An extensive study in this regard has been made by Research, Designs and Standards Organisation(RDSO)., Lucknow, India, the results being published as a report [1]. Necessary data for this study has been obtained from this report.

In this paper, the fatigue reliability analysis of Early Steel Girder Bridges has been presented. Using the results from RDSO report [1], S-N curve characteristics and equivalent stress range have been computed. A limit state equation based on Miner's rule has been formulated. Using the Advanced First Order Second Moment (AFOSM) method, the reliability index, β, has been found out for various cases.

2.0 ANALYSIS OF RESULTS OF FATIGUE TESTS AND STRESS SPECTRUM

The results of fatigue tests and stress spectrum have been taken from Reference [1].Fatigue tests have been performed on specimens obtained from early steel girder bridges namely a)Mahanadi bridge b)Netravathi bridge, and c)Koakhai bridge. The specimens contain five rivet holes and some specimens with rivets also have been tested. Using the test results, a linear regression analysis [2] has been carried out to get the S-N curve parameters. The results are presented in Table 1.

For most of the structural details, the value of slope of S-N curve, m is taken as 3 . However the results from Table 1 indicate that the range of m is 0.6954 to 4.6495 . This could be due the variation in chemical composition and other factors like age of the structure, method of rivet hole preparation, rivet clamping force ,variation in loads and their frequency etc.. In Table 2, the computed values of mean and coefficient of variation (COV) of m and K are given.

Under the actual traffic conditions, stress records have been obtained over a period of time and then using rain flow [3] method of cycle counting stress histograms are created [1].For the stress histograms available in Reference

Sl.No.	Details of the bridge	S - N curve parameters	
		Inverse slope m	Intercept K (in MPa units)
	A.Mahanadi bridge		
1	Bottom chords (BC)	4.649	2.473 E + 15
2	Stringers (ST)	1.298	1.518 E + 08
3	Cross girders (CG)	4.522	1.303 E + 15
4	Top chords (TC) + BC	1.567	7.518 E + 08
5	End rakers (ER) + Diagonals (DG)	2.651	6.233 E + 10
6	ER + DG + TC	2.628	5.389 E + 10
7	ST + CG	2.370	3.071 E + 10
8	All members	1.804	1.879 E + 09
	B.Netravathi bridge		
9	BC + DG + ER	1.033	6.780 E + 06
10	All members	0.695	1.815 E + 06
	C.Inter bridge combinations		
11	Cross girders	1.128	8.251 E + 06
12	Bottom chords	1.308	2.467 E + 08
13	End rakers	3.248	1.052 E + 12

Table 1 Linear regression results

Sl.No.	Description	Mean	Coefficient of variation (COV)
1	Intercept K	2.906 E + 14	2.575
2	Slope m	2.223	0.578

Table 2 Statistics of S - N curve parameters

[1], equivalent stress range S_{re} has been computed as follows [4]

$$S_{re} = \left[\sum f_i S_i^m \right]^{1/m} \qquad (1)$$

where f_i is the relative frequency of the stress range S_i, m is the average value of slope .The results obtained for various cases are listed in Table 3 .

Sl.No.	Description of bridge	S_{re} (MPa)	No. of cycles / day
	Mahanadi		
1	Stringer	17.881	1170
2	Cross girder	20.161	1160
3	Bottom chord	49.109	116
4	Diagonal	25.068	424
	Baitarani		
5	Stringer	28.144	1217
6	Cross girder	26.135	365
	Krishna		
7	Stringer	65.988	1966
8	Cross girder	74.273	1328
9	Bottom chord	104.799	188
10	Diagonal	52.831	36

Table 3 Equivalent stress range for all members

3.0 LIMIT STATE FORMULATION AND RELIABILITY ANALYSIS

To carry out the reliability analysis, the limit state equation has to be formulated. This equation is developed based on Miner's model for cumulative damage [5].

The cumulative damage, D, is given by

$$D = \sum n_i / N_i \tag{2}$$

where n_i is the number of cycles of stress range S_i and N_i is the number of cycles to failure at constant stress range S_i.The value of N_i is to be chosen from S-N curve. Failure state is reached when D is equal to one. If summation is written for each cycle,

$$D = \sum 1/N(S_i) \tag{3}$$

summation being carried out for each cycle. From the S-N curve, the following relation is written for constant stress range.

$$K = N\,S^m \tag{4}$$

However, for variable amplitude stress range, knowing the stress histogram, equivalent stress range can be calculated from Eq.1 . Hence Eq.3 is written as

$$D = (N/K) \sum S_i^m \qquad (5)$$

Introducing the equivalent stress range, S_{re}, the above equation reduces to

$$D = (N/K)\ (S_{re})^m \qquad (6)$$

Hence the limit state equation is

$$g = D-(N/K)\ (S_{re})^m \qquad (7)$$

and the failure surface equation is

$$D-(N/K)\ (S_{re})^m = 0 \qquad (8)$$

The modified form of limit state equation is obtained by taking logarithm on either side of Eq.6. Thus

$$\log(D\ K)-m \log S_{re}-\log N = 0 \qquad (9)$$

is the equation for the failure surface and the modified limit state equation is

$$g = \log(D\ K)-m \log S_{re}-\log N \qquad (10)$$

The random variables are D, K and m. The statistics of K and m have already been fixed and given in Table 3 . Mean value and COV of D, obtained from Reference[6], are 1.044 and 0.3 respectively. Having Eq.10 as the limit state equation, reliability index,β, is computed for various cases using a program based on AFOSM method. N represents the desired life in cycles. All the variables are assumed to be lognormally distributed. The computed values of β are presented in Tables 4 and 5 for different bridges.

4.0 DISCUSSION AND CONCLUSION

In this study, fatigue reliability analysis of early steel girder bridges with S-N curve approach has been presented. Using the fatigue test results and stress measurements obtained from Reference [1], β values for various cases have been presented in Tables 4 and 5 .Table 1 presents the S-N curve characteristics.

From Tables 4 and 5, it can be seen that the average value of β is around 2 which corresponds to a probability of failure of 2.3 per cent. This average value of β is the same as proposed by Moses et al [7] for fatigue evaluation of highway bridges. Bottom chords being the main members of a truss bridge have a higher stress range than other members and hence there is a reduction in beta value for these members. However from Table 3, it is observed that the number of cycles per day for bottom chords is very low when compared to other members. It is also seen that there exists a linear relationship between β and logarithm of number of cycles, N.For various cases, this relationship has been established and the results of this analysis are presented in Table 6 . A typical plot of β versus log N values for stringer of Mahanadi bridge is shown in Fig.1 .Considering all the members and bridges together , an average β versus log N curve is obtained as shown in Fig.2. .

Sl. No.	No. of cycles $N \times 10^6$	Reliability index β			
		Stringer	Cross girder	Bottom chord	Diagonal
	Mahanadi bridge				
1	12.81	2.076	2.001	1.517	1.870
2	2.00	2.270	2.194	1.711	2.064
3	1.75	2.283	2.208	1.724	2.072
4	1.50	2.299	2.223	1.740	2.092
5	1.25	2.316	2.240	1.757	2.110
6	1.00	2.338	2.262	1.779	2.132
7	0.75	2.365	2.289	1.806	2.160
8	0.50	2.403	2.327	1.845	2.197
9	0.25	2.466	2.390	1.907	2.260
10	0.10	2.546	2.470	1.987	2.340
	Krishna bridge				
1	12.81	1.381	1.329	1.186	1.483
2	2.00	1.575	1.523	1.380	1.677
3	1.75	1.588	1.536	1.393	1.690
4	1.50	1.603	1.552	1.408	1.705
5	1.25	1.621	1.569	1.426	1.723
6	1.00	1.643	1.591	1.448	1.744
7	0.75	1.670	1.618	1.475	1.771
8	0.50	1.708	1.656	1.513	1.809
9	0.25	1.771	1.719	1.576	1.872
10	0.10	1.851	1.799	1.656	1.952

Table 4 Value of reliability index for members of Mahanadi and Krishna bridges

S.No	1	2	3	4	5	6	7	8	9	10
$N \times 10^6$	12.81	2.00	1.75	1.50	1.25	1.00	0.75	0.50	0.25	0.10
β for* ST	1.805	1.998	2.012	2.027	2.044	2.066	2.093	2.131	2.194	2.274
β for* CG	1.846	2.040	2.053	2.068	2.086	2.108	2.135	2.173	2.236	2.316

* Note: ST- Stringer; CG- Cross girder

Table 5 β values for members of Baitarani bridge

Sl.No	Description	Equation to the line
	Mahanadi bridge	
1	Stringer	β = - 0.2219 logN + 3.6655
2	Cross girder	β = - 0.2219 logN + 3.5895
3	Bottom chord	β = - 0.2219 logN + 3.1065
4	Diagonal	β = - 0.2219 logN + 3.4594
	Baitarani bridge	
5	Stringer	β = - 0.2220 logN + 3.3939
6	Cross girder	β = - 0.2219 logN + 3.4354
	Krishna bridge	
7	Stringer	β = - 0.2220 logN + 2.9706
8	Cross girder	β = - 0.2220 logN + 2.9188
9	Bottom chord	β = - 0.2220 logN + 2.7757
10	Diagonal	β = - 0.2219 logN + 3.0719
11	All the cases	β = - 0.2219 logN + 3.2387

Table 6 Relationship between β and log N for various cases

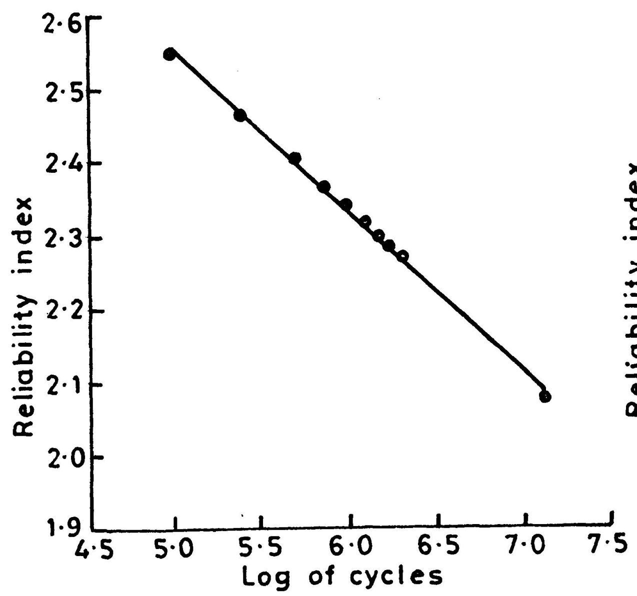

Fig. 1 Relationship between β and log N for stringer of Mahanadi bridge

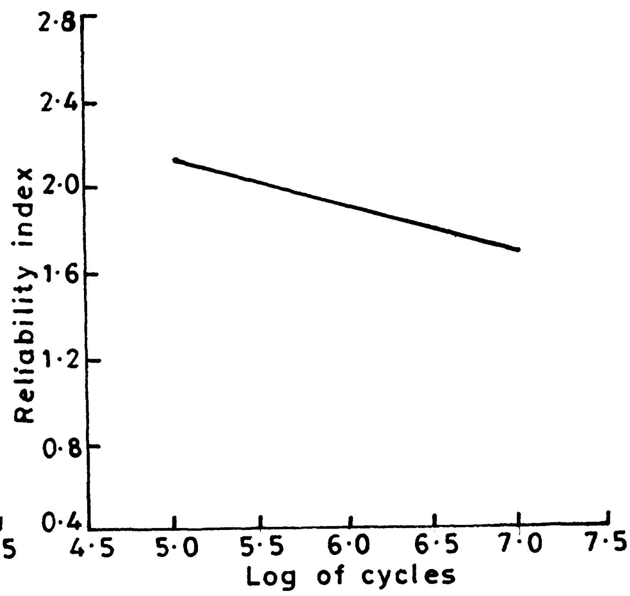

Fig. 2 Relationship between β and log N considering all cases

From Fig 2 , it can be seen that for a β value of 2 , log N is equal to 5.5. This gives the number of cycles as $\simeq 0 \cdot 3 \times 10^6$. For the stringer of Mahanadi bridge undergoing 1170 cycles per day, this would mean that a detailed inspection has to be done once in every 250 days. Similarly for the bottom chord of Krishna bridge, the optimum inspection interval would be four years. Hence if the inspection is accompanied with subsequent repair if needed, the service life of the member of the bridge could be further extended at a acceptable risk level . Thus these results provide an insight to the fatigue behaviour of early steel girder bridges and also help in deciding about the optimum interval for inspection and evaluation . These results also aid in fixing a target reliability index for the design of steel bridges.

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Reliability Analysis of Steel Railway Bridges under Fatigue Loading

Analyse de fiabilité des ponts rail en acier sous charges de fatigue

Zuverlässigkeitsanalyse von Bahnbrücken aus Stahl unter Ermüdungslasten

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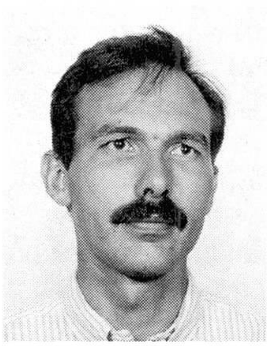

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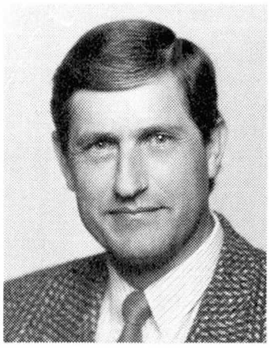

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SUMMARY

Many railway bridges constructed in the last century are still in service today, some having been repeatedly strengthened to meet to new needs. Regular assessment is an important part of the management of their future service. An important question concerning these bridges is fatigue. This paper shows how the probability of failure of construction details can be calculated taking into account the statistical scatter of certain evaluation parameters. The remaining fatigue life is essentially dependent on stress range, the structural model adopted for the assessment and the uncertainties of the traffic models.

RÉSUMÉ

Beaucoup de ponts en acier, construits au siècle passé, ont été renforcés plusieurs fois afin de satisfaire aux nouvelles exigences. Pour les maintenir en service, ils doivent être évalués et contrôlés périodiquement. Une question importante est le comportement à la fatigue. Cet article présente une méthode pour calculer la probabilité à la rupture des détails de construction à l'aide d'hypothèses probabilistes. La durée de service restante dépend essentiellement de la différence de contraintes, de la précision du modèle statique et des hypothèses sur les incertitudes liées aux charges de trafic.

ZUSAMMENFASSUNG

Viele der heute noch im Betrieb stehenden Bahnbrücken wurden im letzten Jahrhundert erbaut und teilweise in der Zwischenzeit mehrfach verstärkt, um sie den neuen Bedürfnissen anzupassen. Im Hinblick auf eine weitere Nutzung müssen sie periodisch beurteilt und kontrolliert werden. Eine wichtige Frage bildet dabei die Ermüdung. In diesem Beitrag wird gezeigt, wie die Versagenswahrscheinlichkeit von Konstruktionsdetails mit Hilfe von probabilistischen Ansätzen ermittelt werden kann. Die Restnutzungsdauer ist im wesentlich abhängig von der Spannungsdifferenz, der Genauigkeit des statistischen Modells und von den Annahmen über die Modellunschärfe bei den Verkehrslasten.

1 INTRODUCTION

Many railway bridges built at the end of the last or at the beginning of the present century are still in service today. The defined service lifes of these structures have been reached or even exceeded [1]. Owners and controlling authorities must decide how these bridges can be used in the future. This involves a re-assessment of structural safety including fatigue. An example of the evaluation of a riveted bridge built in 1875 is presented in [2]. Other reasons for an assessement of the remaining service life are new service conditions or changed structural behaviour under service loads.

The method for the **assessment of fatigue safety** comprises three stages :
1. Identification of critical construction details.
2. Evaluation of fatigue safety by calculation of failure probability.
3. Monitoring of construction details by regular inspection.

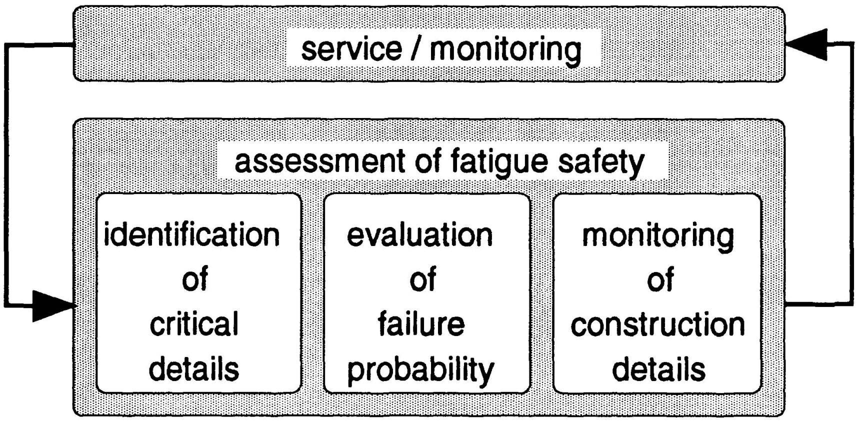


Figure 1 Method for assessing the fatigue safety of critical construction details

This paper presents a method for assessing fatigue safety, in particular the procedure for the calculation of failure probability is discussed. The relationship between the calculations of crack propagation and of damage accumulation is emphasized.

2 METHOD FOR ASSESSING THE FATIGUE SAFETY

The condition of a structure should be monitored by regular inspection throughout its service life. An assessment of fatigue safety may be required as a result of observations (displacements, vibration characteristics, corrosion, cracks) or because of changes in service conditions (increase of axle or uniformly distributed loads) or for legal reasons when a defined service life is reached. Fatigue safety is mainly dependent on the following three parameters :

- **Applied stress ranges :** The applied stress ranges are a function of the service loads and the structural behaviour of the bridge.
- **Geometry of construction details :** The stress concentrations caused by the geometry of construction details and the crack shape may lead to an acceleration of crack propagation and a decrease in fatigue category.
- **Number of stress cycles :** The number of stress cycles applied in the past directly influences the remaining service life of a structure.

In the method for the **assessment of fatigue safety** (Figure 1) [1], based on the three main parameters mentioned above, the following stages are identified assuming that the bridge to be evaluated has been defined.

STAGE 1 : For the structure to be assessed, the first task is the **identification of critical construction details**. In order to identify critical construction details, it is often sufficient to carry out an expert inspection and an intensive study of the available documents. Inspection and maintenance reports can be particularly helpful. The critical construction details can be identified using a deterministic calculation appropriate to current design methods. In this way, one can derive a list of priorities for later investigations.

STAGE 2 : In this paper a probabilistic method to **evaluate the failure probability** will be presented. This approach enables an assessement of fatigue safety that is more sophisticated than was previously possible.

STAGE 3 : The **monitoring of construction details** makes it possible to maintain the structure. To this end, the inspection intervals and techniques need to be selected.

In the following chapter, the discussion is focused on stage 2 relating to the evaluation of failure probability.

3 EVALUATION OF FAILURE PROBABILITY

3.1 Relationship between crack propagation and damage accumulation calculations

The calculation of remaining fatigue life is improved by combining the advantages of both damage accumulation and crack propagation calculations.

The advantages of the **damage accumulation calculation**, based on fatigue strength curves (S-N curves, Wöhler curves), are : this procedure is widely accepted, many test results are available and the probability density functions of the fatigue strength of the construction details are known. In addition, a classification system [3] is available, which is applicable to frequently used construction details.

The advantage of a **crack propagation calculation**, based on fracture mechanics, is that crack propagation for each individual stress range can be calculated. Using this approach the influence on crack propagation of each stress range can be considered. The important disadvantage is that the fracture mechanics parameters for the individual construction details are generally not well known.

Stable crack propagation can be calculated by using a fracture mechanics model (equation 1), shown in Figure 2. Crack propagation depends on the difference between stress intensity factors ΔK, also called stress intensity factor, defined by

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

ΔK : difference between stress intensity factors (stress intensity factor)

ΔK_{th} : threshold value of stress intensity
$Y(a)$: stress concentration factor
$\Delta\sigma$: applied stress range
a : crack size

For a constant stress range $\Delta\sigma$, the stress intensity factor ΔK will increase with increasing crack size a. In addition, a threshold value can be observed, i.e. a limit below which no crack propagation will occur. For an applied spectrum of different stress ranges, the number of the stress ranges which are greater than the threshold limit will increase as the crack size increases.

On fatigue strength curves (Figure 3) for constant amplitude stress ranges, a fatigue limit $\Delta\sigma_D$ can be observed, i.e. a limit below which no crack propagation will occur. In the case of variable amplitude stress ranges, damage will accumulate if a part of the spectrum is above the fatigue limit. Therefore, the limit below which no crack propagation occurs is no longer constant but decreases with increasing crack size. This limit is now called the **damage limit** $\Delta\sigma_{th}$, defined by

$$\Delta\sigma_{th} = \Delta\sigma_D \cdot (1 - D) \tag{2}$$

$\Delta\sigma_D$: fatigue limit
$\Delta\sigma_{th}$: damage limit
D : existing damage

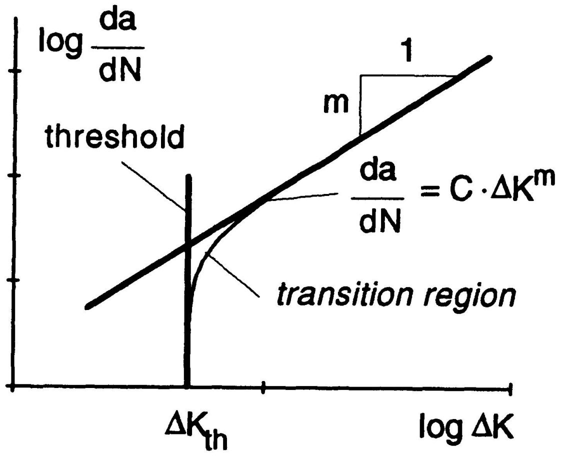


Figure 2 Crack propagation as a function of the stress intensity factor

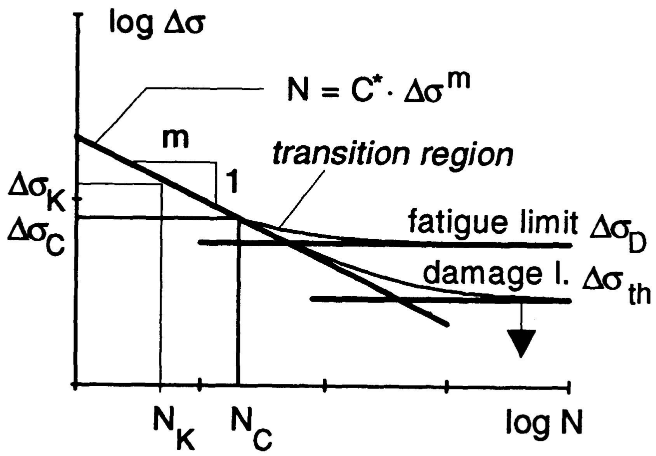


Figure 3 Fatigue strength curve

The transition region from the threshold to the Paris law in the crack propagation curve can be described as follows :

$$\frac{da}{dN} = C \cdot \left(\Delta K^m - \Delta K_{th}^m\right) \tag{3}$$

da/dN : crack propagation per stress cycle
C : crack propagation constant
m : slope of the crack propagation curve (Paris law)

By analogy a transition region can also be considered in a damage accumulation calculation. The point (N_K, $\Delta\sigma_K$) is a reference point on the S-N curve having a constant slope m (Figure 3). The position of the fatigue strength curve is defined by the fatigue strength $\Delta\sigma_C$ at N_C cycles; this allows the correlation with reference to the detail category in the tables of the ECCS recommendations [3]. The damage increase per cycle as a function of the applied stress range $\Delta\sigma_i$ is [4] :

$$d_i = \frac{\Delta\sigma_i^m - \Delta\sigma_D^m \cdot (1-D)^m}{\Delta\sigma_K^m - \Delta\sigma_D^m \cdot (1-D)^m} \cdot \frac{1}{N_K} \tag{4}$$

d_i	:	damage increase per cycle
$\Delta\sigma_i$	:	applied stress range
$\Delta\sigma_K$	:	reference stress range
N_K	:	number of cycles to failure for the reference stress range

The damage accumulation calculation with equation (4) can be used to derive the remaining service life by simulations with an accuracy of 5 to 7 % (according to the spectrum) with respect to a crack propagation calculation using fracture mechanics. This corresponds to a significant improvement compared to the fatigue strength curves used for design. The ECCS [3] and Eurocode classification systems may be used in this calculation, bringing with them the possibility of calculating the design life on the basis of the statistics of a large number of fatigue strength tests.

3.2 Evaluation of reliability

The remaining service life depends on the traffic loads used for the simulations. The traffic load model used accounts for the fatigue effect of service loads on the structure. Traffic in the past can be represented approximately by the standard UIC (International Union of Railways) trains [5]. A corresponding model has also been developed for future traffic. Dynamic coefficients are used for the evaluation of stress ranges for standard trains, thereby taking into account the dynamic characteristics of the structure and the vehicles.

A simplified standard load, called the fatigue load Q_{fat}, multiplied by a dynamic coefficient Φ, is used as a reference value. With the fatigue correction factor α, the relationship between the stress range due to the fatigue model $\Delta\sigma(\Phi \cdot Q_{fat})$ [6] and the damage due to the stress ranges of the traffic model can be established.

Based on simulation calculations, in which the statistical scatter for each parameter is considered, the statistical distribution of the correction factor $p(\alpha)$ as a function of a given number of future trains N_{fut} can be calculated. The distribution of the required fatigue strength for a construction detail in an existing bridge can be calculated as the product of the fatigue load stress range and the distribution of the fatigue correction factor [4] :

$$p(\Delta\sigma_{req}) = p(\alpha) \cdot \Delta\sigma(\Phi \cdot Q_{fat}) \tag{6}$$

$\Delta\sigma(\Phi \cdot Q_{fat})$	:	stress range due to the fatigue load multiplied by the dynamic coefficient
$p(\Delta\sigma_{req})$	:	probability distribution of the required fatigue strength
$p(\alpha)$	:	probability distribution of the fatigue correction factor for a given number of future trains N_{fut}

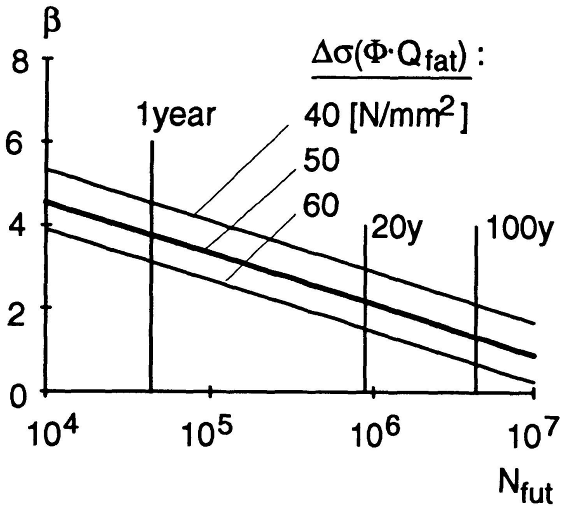
Knowing the required fatigue strength, the available strength of a given detail has to be determined. Probability distributions of the fatigue strength $p(\Delta\sigma_R)$ of typical construction details have been obtained from tests [7, 8]. By comparison of the two distributions, the probability of failure can be calculated and expressed in terms of the reliability index β :

$$\beta = \frac{R - S}{\sqrt{s_R^2 + s_S^2}} \tag{8}$$

β : reliability index
R, S : mean values of the resistance and the load effect, respectively
s_R, s_S : standard deviations of the resistance and the load effect, respectively

Figure 4 shows the relationship between the reliability index β and the number of future trains N_{fut}. For an increasing number of trains, the reliability index β can be seen to decrease. For a semi-logarithmic scale and using analytically derived expressions for R and S, this results in a straight line. This simple relationship is very convenient for a sensitivity analysis of the main parameters.

The number of trains for 1, 20 and 100 years are also identified in Figure 4. For the assessement of fatigue safety of an existing bridge, the region between 10^5 and 10^6 trains is of most interest. This corresponds to a future service period of between 5 and 25 years.



Assumptions :

- number of trains per day : 120
- year of construction : 1900
- influence length : 20 m
- fatigue limit at : $7 \cdot 10^6$ cycles
- coefficient of variation of the parameters with scatter : 10 %

Figure 4 Variation of reliability index with number of future trains

3.3 Sensitivity analysis

Figure 4 shows the influence of the stress range due to fatigue load. Improving the accuracy of the structural model directly improves the value of the stress range and thus increases the calculated reliability. Site measurement of bridge behaviour is recommended for calibration of the structural model. The assumptions of Figure 4 using a stress range of 50 N/mm^2 will be used for the following figures.

Figure 5 shows the effect of the fatigue limit. The position of the fatigue limit $\Delta\sigma_D$ is characterised by the corresponding number of cycles N_D. The longer a bridge will remain in service, the more important the determination of N_D will be. To this end, the fatigue limit should be defined for each category of construction detail.

The influence of the year of construction is shown in Figure 6. This parameter is normally known. The small number of trains at the beginning of this century does not influence the reliability index. In addition, with increasing number of future trains this parameter is less important. This means that the same correction factor α can be used for plus or minus 20 years, for the values assumed for Figure 4.

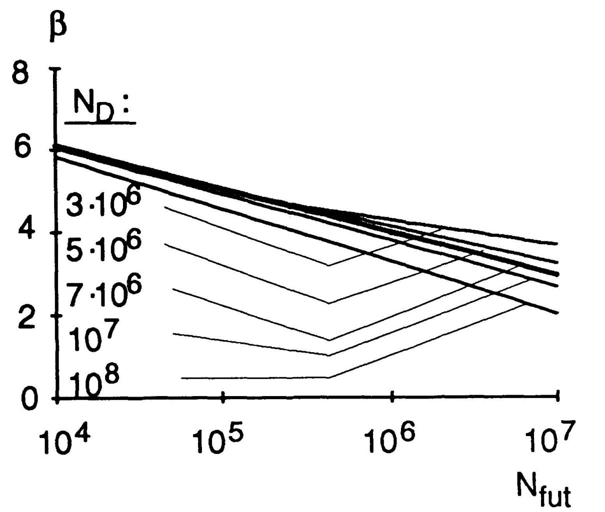


Figure 5 Influence of the position of the fatigue limit N_D

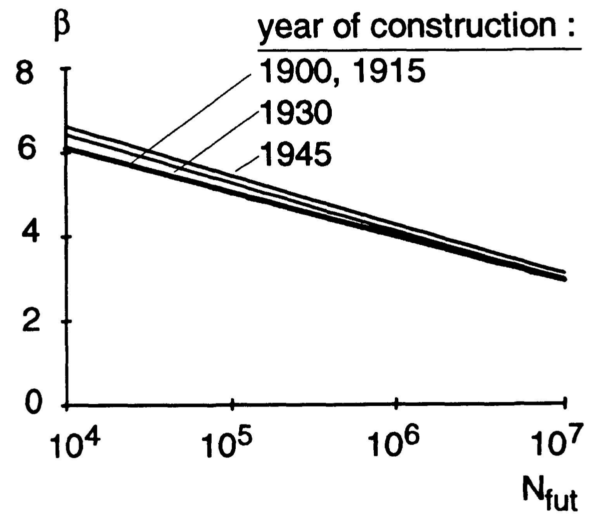


Figure 6 Influence of the year of construction

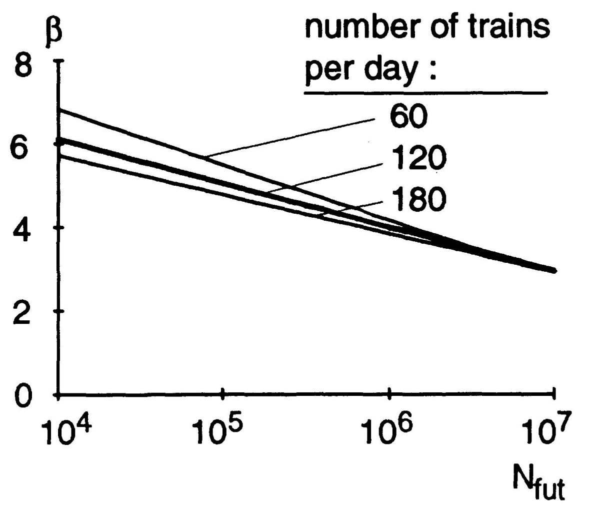


Figure 7 Influence of the number of trains per day

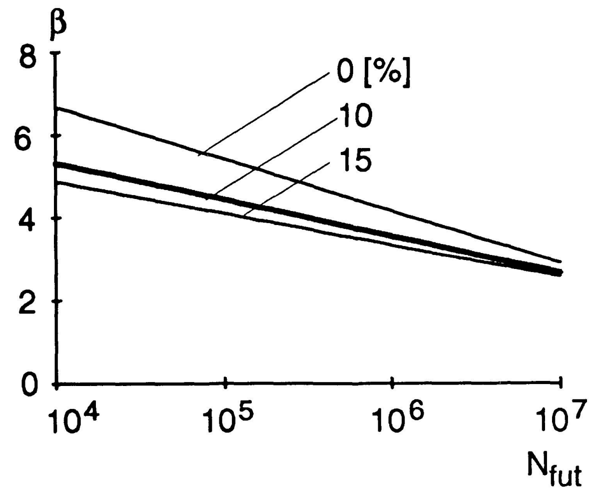


Figure 8 Influence of model uncertainty

Figure 7 shows the influence of the number of trains. The number of trains in the past has been fixed according to a proposal by UIC and is based on the actual number of trains as a reference value. The figure shows that the number of future trains is significant for less than 10^5; for a greater number this parameter has little importance.

The influence of model uncertainty is shown in Figure 8. The scatter has a significant effect on the reliability index. When the coefficient of variation increases, the reliability index decreases. Results are not conservative when the model uncertainty is neglected (higher values of β). However, it can be seen that the difference for β due to a change of uncertainty between 10 and 15 % is small. The influence of the model uncertainty decreases for increasing coefficents of variation and with increasing number of trains.

4 CONCLUSIONS

A method for the evaluation of fatigue safety for existing railway bridges has been presented. The following conclusions can be made :

- Based on the principles of fracture mechanics and especially on the concept of a threshold value for crack propagation, a new damage limit for the fatigue strength curves can be defined. This limit decreases with increasing damage. For calculation purposes, it is assumed that below this damage limit no crack propagation and therefore no damage will occur. Based on this assumption a good agreement between damage accumulation and the more complex crack propagation calculation is obtained.
- The probability of failure can be calculated by comparing the required and existing fatigue strength. The most important parameters are stress range due to the fatigue load and the position of the fatigue limit. Of lesser importance are the year of construction, the year in which service began and the influence length, if it is greater than 10 m.

The probability of failure can be related to the probability of crack detection [4]. A construction detail with a theoretical probability of failure below the target value can remain in service, when the probability of crack detection (based on a given inspection technique) and the inspection intervals are taken into account.

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Optimal Fatigue Testing - a Reassessment Tool

Essais de fatigue optimalisés - une nouvelle estimation

Optimale Ermüdungsversuche als Hilfsmittel zur Überprüfung

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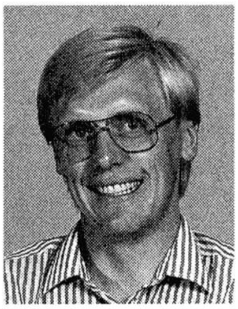

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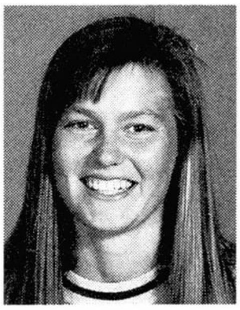

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SUMMARY
This paper considers the reassessment of the reliability of tubular joints subjected to fatigue load. The reassessment is considered in two parts namely the task of utilizing new experimental data on fatigue life to update the reliability of the tubular joint ant the task of planning new fatigue life experiments for the same purpose. The methodology is based on modern probabilistic concepts amd classical decision theory. The special case where the fatigue life experiments are given in terms of SN curves is considered in particular. The proposed techniques are illustrated by an example.

RÉSUMÉ
L'étude traite d'une nouvelle estimation de la fiabilité des jonctions tubulaires sous l'effet de charges de fatigue. La nouvelle estimation est présentée en deux parties: utilisation de nouveaux résultats expérimentaux de résistance à la fatigue pour une mise à jour de la fiabilité des jonctions tubulaires, et organisation de nouveaux essais dans le même but. La méthode est fondée sur la théorie des probabilités et la théorie classique de décision. L'étude traite spécialement les cas où des expériences sont présentées sous forme de courbes SN. Les techniques sont illustrées par un exemple.

ZUSAMMENFASSUNG
Der Beitrag behandelt die Überprüfung der Zuverlässigkeit ermüdungsbelasteter Rohrverbindungen. Dazu sind zwei Teilaufgaben zu lösen: Die Verwendung neuen Datenmaterials zu ihrer Ermüdungsdauer und die Planung neuer Ermüdungsversuche zu eben diesem Zweck. Die Methoden basieren auf modernen Konzepten der Wahrscheinlichkeitstheorie und auf klassischer Entscheidungstheorie. Der Fall, dass Versuchsdaten alks S-N-Kurven vorliegen, wird gesondert behandelt. Die vorgeschlagenen Verfahren werden an einem Beispiel erläutert.

1. Introduction

Engineering structures subjected to environmental conditions such as time varying loading and corrosion will fail when the accumulated damage of the structure reaches a certain critical level. When a structure is designed and its design is adjusted such that the target safety of the structure is maintained throughout its design lifetime. This is obtained either through classical code based design or using modern probabilistic concepts. Typically, however, for engineering structures the original use of the structure or the initial design conditions is changed several times before it is taken out of service. Such changes are e.g. a prolongation of the design lifetime, changes in the loading conditions, but also imposed accidental damage conditions have similar effects. In such cases it may be necessary to justify that the structure is capable of fulfilling its requirements in terms of safety, i.e. to reassess the structural safety. For this purpose information about the actual state of the structure is collected. Such information obviously includes the damage state of the structure, but also information about other important characteristics of the failure modes of the structure. Important examples hereof are material parameters, loading characteristics and geometry. The collection of such information can be rather expensive and cumbersome as is e.g. the case of inspection planning for offshore structures or in the case of material fatigue life testing. Therfore, it is mandatory to have access to a methodology which provides a rational decision basis on how to collect such additional information taking into account the economic aspects. The framework of modern reliability theory, see e.g. Madsen et al. [1] and classical decision theory, see e.g. Raiffa & Schlaifer [2] provides such a tool. The scope of the present paper is to present this tool and to illustrate its application in the case where the safety of a structure subjected to fatigue failure is reassessed using additional fatigue life experiment data. Two different cases of reassessment are considered, namely the case where the reliability of a structural component subject to fatigue failure is updated using new fatigue data and the case where a new experiment is planned for reassessment.

2. Experiment Planning as a Decision Tool

The use of experimental data for the purpose of modelling is recognized as one of the most important tools in the design of engineering structures, see e.g. Ditlevsen [3]. Typical examples hereof are the estimation of material characteristics such as yield stresses and modulus of elasticity, but experimental results are also used for the estimation of parameters in parametric equations such as fatigue crack growth models. In the past experiments have normally been performed such that the uncertainty associated with the measured quantity is adjusted to some specified acceptable range, see Viertl [4]. These methods disregard economic aspects and the actual engineering application where the statistics of the considered quantity are used. Experiment planning was i.e. seen as an isolated problem.

The increasingly accepted application of modern probabilistic methods such as FORM/SORM methods in structural engineering allows for a more refined formulation of experimental planning. This is due to realistic probabilistic modelling of loading, consistent representation of experimental results together with efficient tools for the estimation of probabilities. Using these tools it is possible to perform experiment planning from a more rational basis namely to reduce the total expected costs for the considered engineering structure. This approach is fundamentally different from the classical approach mentioned above as it allows to perform experiment planning in a cost optimal fashion. Following results from classical decision theory, see e.g. Ang & Tang [5] the optimal experiment plan is the experiment plan which minimizes the expected total cost $E[C_T]$ of the considered engineering structure. Here, total expected costs include all costs associated with the planned experiments $E[C_e]$, the expected costs of the structural design $E[C_d]$, the expected costs of maintenance $E[C_m]$ together with the expected costs of failure of the structure $E[C_f]$. Hence, the expected total costs for an engineering structure can be written as

$$E[C_T] = E[C_e] + E[C_d] + E[C_f] + E[C_m] \tag{1}$$

Experiment planning can in a wide sense be understood as the planning of any action revealing information which has impact on the predicted performance of the structure. Therefore, an experiment can be the action of performing experiments for the estimation of the structural material parameters but it can also be the action of measuring unknown (and uncertain) quantities such as structural damage, structural dimensions and characteristics of the loading environment. With this interpretation of experiment planning it is seen that experiment planning becomes an essential tool in decision making for engineering structures not only in the design phase of the structure but also in the situation of a reassessment of the structural integrity.

3. Experiment Planning in Fatigue Testing

In the fatigue life assessment of engineering structures such as steel bridges and offshore steel structures parameters of importance are among others the crack growth law material parameters, the stress concentration factors, the actual geometry of the considered structural detail and the damage state of the structure.

In the reassessment situation the reliability of the structure is updated through experiments revealing information about any of the above-mentioned quantities.

Assume that the failure probability can be estimated from

$$P_f = P(g(\mathbf{X}, N) \leq 0) \tag{2}$$

where $g(\star)$ is the limit state function, $\mathbf{X}$ is a vector including the basic uncertain variables such as geometric parameters and stress concentration factors, see e.g. Dover et al. [6], and N is the random fatigue lifetime. Then the failure probability can be updated through experiments revealing the realizations of the basic uncertain variables and/or through experiments revealing realizations of functional relationships of the basic uncertain variables.

In the following it is assumed that it is possible to perform fatigue experiments using a material which is representative for the material used in the structure under consideration. The specific problem of making a cost optimal experiment plan is treated.

Typically, fatigue life experiments are performed in order to estimate the generally uncertain parameters $\mathbf{P}$ of the lifetime distribution $F_N(n|\mathbf{P})$ for a given material. Given a distribution assumption of the fatigue lifetime in terms of the number of constants or equivalent stress range load cycles to failure N the distribution parameters $\mathbf{P}$ are estimated using standard tools from the statistics, such as the maximum likelihood method, the method of moments or Bayesian statistics. Fatigue life experiments can also be used to estimate parameters in distribution free material lifetime models using e.g. regression analysis in the statistical analysis of SN data.

The SN curve and the data points illustrated in figure 1 are taken from Dover et al. [6]. The curves represent the mean value and the two standard deviation fractile of fatigue life tests of offshore tubular joints obtained in the study [7]. The data points in the figure are used later in an example as new information in a reassessment situation.

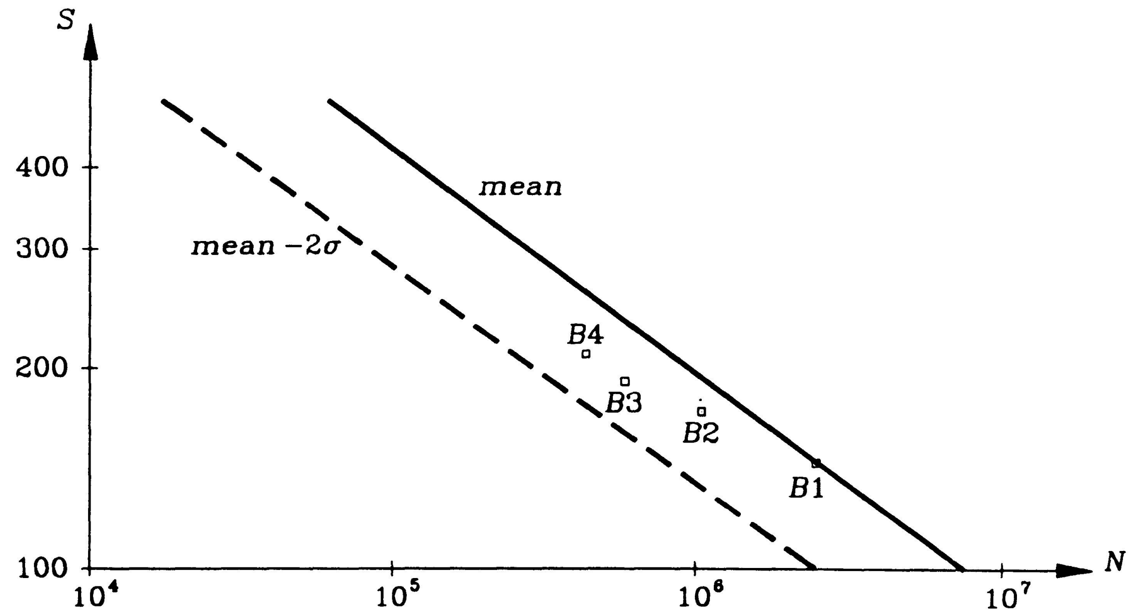

Figure 1. Illustration of typical representation of fatigue lifetime in terms of an SN diagram, Dover et al. [6].

To define an experiment plan the number of experiments, the stress range levels for the individual experiments and the maximum number of load cycles until termination are most frequently used as decision parameters. When the number of experiments is increased the uncertainty structure associated with the model parameters **P** is changed. The uncertainty will in general decrease if the number of experiments is increased. Therefore, the expected failure costs for the mechanical component considered are also changed.

The experimental costs due to additional experiments are obviously dependent on the stress range levels for which the experiments are to be performed. Therefore, when deciding if additional experiments should be performed the relevant failure criteria and all the information about the uncertain variables involved in the problem have to be taken into consideration.

As stated above the optimal experiment plan is the plan which minimizes the total expected experiment and failure costs caused by additional experiments at a given stress range level and a given maximum number of load cycles before termination. As the design costs cannot be changed in a reassessment situation their part in the expected total costs can be omitted in the present context. For simplicity, maintenance costs are not considered even though they can play an important role in the case of experimental planning for future reassessments. It is assumed that some prior information exists, for example in the form of existing experimental results and the problem is to determine an optimal plan for additional experiments. The existing experiments are assumed to be performed at M stress range levels $s_1, s_2, \ldots, s_M$. The number of additional experiments are $\mathbf{n} = (n_1, n_2, \ldots, n_M)^T$ at the M levels $s_1, s_2, \ldots, s_M$. As decision variables $\mathbf{n}$ and the number of load cycles to termination N_{ter} can be used.

Because the number of load cycles to failure in an additional experiment is random the change in total expected costs has to be integrated over all possible outcomes of load cycles to failure weighted by their likelihood. This corresponds to a pre-posteriori analysis from the classical decision theory see e.g. Raiffa & Schlaifer [2].

The corresponding optimization problem is written

$$\min_{N_{ter},\mathbf{n}} E[C_T(N_{ter}, \mathbf{n})] \tag{3}$$

Constraints related to the failure probability can easily be incorporated into (3). The total expected costs $E[C_T(N_{ter}, \mathbf{n})]$ associated with additional experiments are then

$$E[C_T(N_{ter}, \mathbf{n})] = C_f E_{\mathbf{N}^U}[P_f{}^U(\mathbf{N}^U, N_{ter}, \mathbf{n})] + E_{\mathbf{N}^U}[C_e(\mathbf{N}^U, N_{ter}, \mathbf{n})] \tag{4}$$

where C_f is the cost of failure, $P_f^U(\mathbf{N}^U, N_{ter}, \mathbf{n})$ is the 'updated' probability of failure given the additional unknown experimental results modelled by the number of load cycles $\mathbf{N}^U$ to failure at the corresponding stress range levels. How to determine this probability is described in the next section. $\mathbf{N}^U$ are modelled as random variables and $E_{\mathbf{N}^U}[\star]$ denotes the expectation operation with respect to $\mathbf{N}^U$. C_e is the experimental costs. The expectation operations in (4) can be estimated by nested FORM/SORM, see e.g. Guers & Rackwitz [8].

The above-mentioned technique can without theoretical difficulties be generalized to the situation where no experimental results are available at the time where the test planning is made. In this case subjective prior information can be used.

4. Probabilistic Reassessment

When new information becomes available the estimates of the probability of failure (and the reliability) of structures can be reassessed. The information considered in this paper is divided into two types

- information of functions of basic stochastic variables
- sample information of basic stochastic variables

The first type of information is related to information about events involving more than one basic stochastic variable. Examples of this type of information are proof load tests, non-failure observations, measurements of response quantities and inspection results related to damage quantities such as fatigue crack sizes.

The information is generally modelled using a stochastic variable Y which is a function of the basic stochastic variables, i.e. $Y = h(\mathbf{X}_1, \mathbf{X}_2, ..., \mathbf{X}_n, N)$. The actual measurements are thus realisations (samples) of Y. The observations can be modelled as equality events $E = \{H = 0\} = \{Y = y_m\}$ or inequality events $I = \{H \leq 0\} = \{Y \leq y_m\}$ where y_m can be some observed quantity.

The probability of failure of a single element with safety margin $M_F = g(\mathbf{X}, N) \leq 0$ can then be updated, see e.g. Madsen [9] and Rackwitz & Schrupp [10].

$$P_f^U = P(M_F \leq 0 | H \leq 0) = \frac{P(M_F \leq 0 \cap H \leq 0)}{P(H \leq 0)} \tag{5}$$

or in the case of observations modelled by equality events

$$P_f^U = P(M_F \leq 0 | H = 0) = \frac{P(M_F \leq 0 \cap H = 0)}{P(H = 0)} \tag{6}$$

These conditional probabilities can be evaluated by standard FORM, see e.g. Madsen [9].

The second type of information is related to situations where samples of one or more basic stochastic variables are obtained. Examples of this type of information are measurements of the geometrical quantities and test results for the fatigue life of a component. Bayesian statistical methods can be used to obtain updated (predictive) distribution functions of the stochastic variables, see Lindley [11] and Aitchison & Dunsmore [12].

Based on prior information (subjective and/or test data) a density function $f_N(n|\mathbf{P})$ for a single basic stochastic variable N is established. $\mathbf{P}$ are parameters defining the distribution function for N. The initial (prior) density function of $\mathbf{P}$ is denoted $f'_{\mathbf{P}}(\mathbf{p})$.

Next it is assumed that an experiment or inspection is performed. m realisations of the stochastic variable N are obtained and are denoted $\mathbf{n}^* = (n_1^*, n_2^*, ..., n_m^*)$. The measurements are assumed to be independent. The updated (posterior) density function $f_{\mathbf{P}}''(\mathbf{p}|\mathbf{n}^*)$ of the uncertain parameters $\mathbf{P}$ taking into account the realisations is

$$f_{\mathbf{P}}''(\mathbf{p}|\mathbf{n}^*) = \frac{f_m(\mathbf{n}^*|\mathbf{p}) f_{\mathbf{P}}'(\mathbf{p})}{\int f_m(\mathbf{n}^*|\mathbf{p}) f_{\mathbf{P}}'(\mathbf{p}) d\mathbf{p}} \tag{7}$$

where $f_m(\mathbf{n}^*|\mathbf{p}) = \prod_{i=1}^m f_N(n_i|\mathbf{p})$.
The predictive density function (i.e. the updated density function) of the stochastic variables N taking into account the realisation $\mathbf{n}^*$ is obtained by

$$f_N(n|\mathbf{n}^*) = \int f_N(n|\mathbf{p}) f_{\mathbf{P}}''(\mathbf{p}|\mathbf{n}^*)\, d\mathbf{p} \tag{8}$$

An updated estimate of the probability of failure $P_f^U(\mathbf{n}^*) = P(g(\mathbf{X}, N) \leq 0)$ can then be determined using the updated (predictive) density function $f_N(n|\mathbf{n}^*)$ as density function for N.
An updated estimate of P_f can also be obtained using the posterior density function of $\mathbf{P}$.

$$P_f^U(\mathbf{n}^*) = P(g(\mathbf{X}, N(\mathbf{P})) \geq 0) \tag{9}$$

In (9) $N, \mathbf{X}$ and $\mathbf{P}$ are stochastic variables. The density function for N is $f_N(n|\mathbf{P})$ and the density function for $\mathbf{P}$ can be the posterior density function $f_{\mathbf{P}}''(\mathbf{p}|\mathbf{n}^*)$.
Instead of using the posterior density an updated stochastic model for $\mathbf{P}$ can also be obtained using classical statistical methods, e.g. the maximum likelihood method. In this case the parameters $\mathbf{P}$ are treated as stochastic variables and the distribution parameters in the joint distribution function $f_{\mathbf{P}}$ are determined by e.g. the maximum likelihood method.

5. Example

In the following example a reassessment situation is considered for an offshore tubular joint subjected to fatigue crack growth. The joint considered in particular is the joint also considered in Dover et al. [6] where the fatigue life has been experimentally determined. It is assumed that the prior information about the fatigue life of the considered joint is given through the SN curve in figure 1. Two problems are considered here. First the problem of updating the reliability of the joint for reassessment by introduction of the four new data points in figure 1 is considered. Thereafter the problem of planning an additional fatigue experiment for the purpose of reassessment is considered.

In order to model the prior information of the fatigue life of the joint the model from Madsen et al. [1] is used with the modification that the slope of the SN curve m is assumed to be a deterministic variable $m = -\beta$. Thereby the fatigue lifetime of the offshore joint can be given as

$$\log N = \overline{y} + \sqrt{\frac{D^2 + S_{xx}(b+\beta)^2}{T_3}}(I + \frac{T_1}{r}) - \beta(\log S - \overline{x}) \tag{10}$$

where r is the total number of experiments, $\overline{x} = \frac{1}{r}\sum_{i=1}^r \log s_i$ and $\overline{y} = \frac{1}{r}\sum_{i=1}^r \log n_i^*$. The parameters S_{xx}, b and D are different combinations of first and second moments of the r experiments as defined in [1]. T_1 and I are standardized normal stochastic variables. T_3 has a $\chi^2(r-1)$ distribution. The stochastic variables are assumed to be independent.

As prior information the curves in figure 1 are used. It is assumed that the SN curve in figure 1 is based on $r = 20$ experiments, four experiments at five different levels of effective stress ranges. The logarithm to the fatigue lifetime N given S is assumed to be normal distributed with mean value equal to $29.69 - 3.0 \log S$ and standard deviation equal to $0.6/\sqrt{1+1/r}$. Based on this assumption the sample moments defining the parameters in (10) are estimated using simulation.

The reliability of the joint can now be estimated by considering the following limit state function

$$g(\mathbf{x}) = \log N - \log n_c \tag{11}$$

where it is assumed that the effective stress range S is log-normal distributed with expected value 200 MPa and standard deviation equal to 20 MPa, n_c is assumed equal to $5 \cdot 10^4$. A FORM analysis gives a reliability index $\beta = 3.772$. The reliability is next updated using the four new experiments from figure 1. The results of this updating is shown in table 1. It is seen that inclusion of experiment B1 and B2 gives an increased reliability index whereas B3 and B4 decreases the reliability. If all four experiments are used, then the reliability increases.

experiment	β
B1	3.857
B2	3.820
B3	3.765
B4	3.769
B1+B2+B3+B4	3.892

Table 1. Reassessed reliability indexes using the new experimental data from figure 1.

Finally the problem of planning an additional experiment for reassessment of the reliability is considered. Assuming that the expected cost of a fatigue life experiment is $E[C_e] = 1 \cdot 10^6 + E[N]$ and that the cost of failure of the offshore joint is $C_f = 1.1 \cdot 10^{11}$ the expected total costs $E[C_T]$ of the joint given one additional experiment are plotted in figure 2 as a function of the stress range where the additional experiment is performed. Also, the total costs corresponding to no experiment are plotted. It is seen that the largest utility is obtained by performing an experiment at $S = 340$MPa.

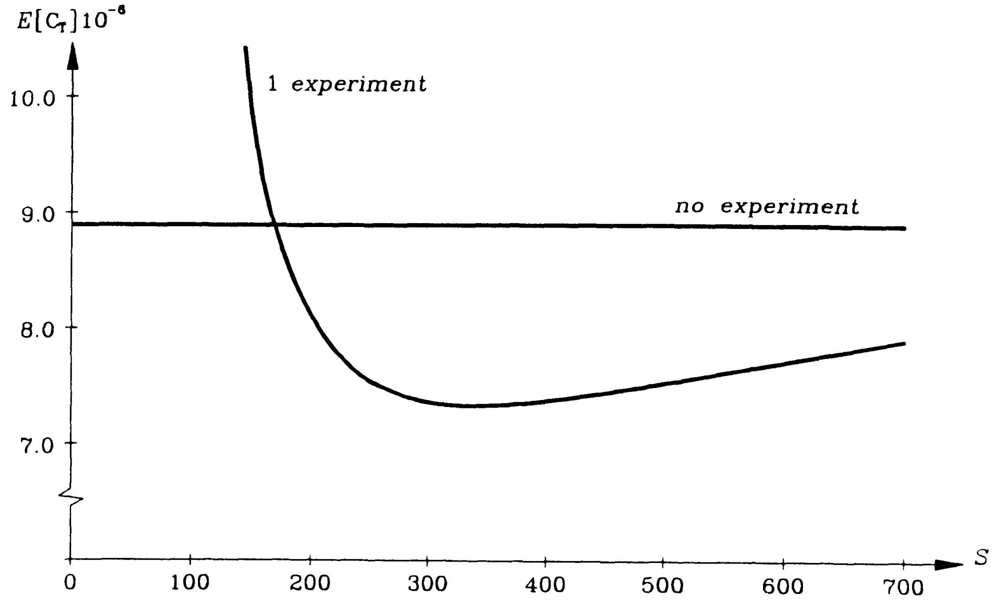

Figure 2. The total expected costs given 1 additional experiment and the total expected cost if no experiment is performed.

6. Conclusions

Based on the modern reliability theory and the classical decision theory a methodology has been proposed for the reassessment of the reliability of engineering structures subject to fatigue failure. Two situations are considered in particular, namely the situation where the reliability is updated using new information about the fatigue life and the situation where a fatigue life experiment is being planned taking economic aspects into account. The methodology is illustrated by an example where a tubular offshore joint is considered for which experimental data are available in terms of SN data. The example clearly shows the significance of additional experiments for the reliability of the joint. It is also shown that the proposed methodology for planning of future fatigue life experiments can be used to identify the most cost-effective stress ranges for additional SN experiments.

7. Acknowledgements

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Upgrading Reliability Assessment of Degraded Structures
Meilleure évaluation de la fiabilité de structures endommagées
Neue Sicherheitsbewertung geschädigter Tragwerke

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SUMMARY
The paper sets forth the possibilities provided by the use of probabilistic methods in assessing the safety of existing structures, and in interpreting the results of tests made in situ. The methodology is based on coupling proven methods with the techniques of Bayesian Inference. The goals of this procedure are manifold: to optimize the inspection programmes on existing buildings; to interpret the results of these inspections; to carry out sensitivity analyses. One example of application is discussed in detail, concerning a reinforced concrete building, erected in 1916-1920, now subjected to a detail inspection programme in order to assess its actual reliability.

RÉSUMÉ
La contribution met en évidence les possibilités dues à l'emploi des méthodes probabilistes pour l'évaluation de la sécurité des construction existantes. La méthodologie proposée est basée sur la combinaison de certaines méthodes connues avec les principes de l'inférence Bayésienne. Les buts envisagés sont multiples: l'optimisation des programmes d'inspection des constructions; l'interprétation des résultats des essais expérimentaux; le développement des analyses de sensibilité. Les auteurs examinent successivement en détail un exemple d'application concernant un bâtiment réalisé en 1916-1920 soumis actuellement à une série approfondie de recherches afin d'évaluer son niveau réel de sécurité.

ZUSAMMENFASSUNG
In diesem Artikel wird erklärt, welche Möglichkeiten sich durch die Anwendung der Wahrscheinlichkeitsmethode beim Bewerten bestehender Tragwerke ergeben. In der Interpretation der Ergebnisse von In-Situ-Messungen werden die eingesetzten Verfahren mit der Technik Bayesscher Schlussfolgerungen gekoppelt. Folgende Ziele werden durch dieses Verfahren erreicht: Optimierung der Überprüfungsprogramme für Gebäude; Interpretation der experimentellen Prüfungsergebnisse; Durchführung der Sensibilitätsanalyse. Es wird dann die Anwendung an einem Stahlbetongebäude im Detail beschrieben, das in den Jahren 1916-1920 erbaut und jetzt einer Reihe von gründlichen Untersuchungen zur Bewertung des wirklichen Sicherheitsgrades unterworfen wurde.

1. INTRODUCTION

The reliability assessment of an existing structure and, eventually, the design of upgrading operations are processes demanding in-depth knowledge of the effective response of the structure under realistic action scenarios. Moreover, the structural behaviour has to be determined taking into account the level of deterioration of the structural elements.
In achieving this objective one must always deal with the considerable uncertainty that arises in defining both the action scenarios, and the structural model and the materials' mechanical properties, which are closely tied to construction quality and generally deteriorate in random fashion over time. Without considering the actions that actually involve the structure in its future operation, the other sources of uncertainty arise out of the variability in space and time of the geometric and mechanical characteristics of the structural elements, and out of the need to adopt an analytical model of their behaviour. They also arise when the available information on the basic variable is incomplete or not wholly significant.
In most cases then, to deepen understanding of the structural behaviour, it becomes necessary to work up information got from quality control, from proof testing, from experimental tests, and from periodic inspection or continuous monitoring, this information being all that is available when the original design is missing. By means of it structural safety can be assessed more reliably: in fact, the additional information, if accurate and consistent, provides substance to the models assumed for deriving the analytical reliability evaluations, which are made on the basis of estimates of the materials' mechanical properties and of their deterioration, and on the basis of forecasts of collapse mechanisms, which would otherwise be devoid of objective support.
A probabilistic approach to evaluating structural safety is made natural by the need to establish stochastic models for each of the several sources of uncertainty. Therefore, the possibilities provided by probabilistic methods in assessing the safety of existing buildings, and in particular in interpreting the results of tests made directly in situ, are set forth in the following, with special reference to an example case of notable importance.

2. SOME REMARKS ON THE RELIABILITY ASSESSMENT PROCEDURE

The methodology examined, proposed in [1], and applied in [2], [3], [4], [5], [6], [7], is based on coupling the FORM (or SORM) methods with the techniques of Bayesian Inference.
This theory appears the most useful approach for quantifying uncertainties in structural engineering problems, especially when coupled with decision analysis. According to Bayesian Inference in fact, and proceeding in a consistent and explicit manner, design can deal with information on events and propositions (qualitative information that is, or estimates based on expert judgment). The Bayesian approach also provides a satisfactory way of explicitly introducing assumptions about prior knowledge, the relevant experience being quantified by the prior distributions. Moreover it does not break down where large amounts of data are absent, it providing a mechanism for using experience, intuition and judgment productively and in a scientifically responsible fashion. Finally, it is also compatible with first-order reliability methods, and is therefore suited to the problem's numerical treatment within a unitary formal context.
The method's goals are manifold: to optimize inspection programmes for existing buildings, with a view to more efficient repair or upgrading interventions, and according to the more likely deterioration factors; to correctly interpret the results of these inspections; and to carry on parametric sensitivity analyses. As concerns the methods for using Bayesian Inference, a "direct use" may be distinguished, aimed at updating the probability density functions of the basic random variables, as may be an "extended" use, coming out of the coupling of the criteria lying at its base with the techniques of reliability analysis of structural systems peculiar to the advanced first-order second-moment methods.

2.1 "Direct" use

Bayesian Inference can be directly applied to problems involving parameter estimation, that is, problems in which additional information are available about the parameters of the probability

density functions of the basic r.v.s, these parameters being considered as r.v.s having a prior distribution that expresses the designer's prior belief in (or knowledge of) their values. The method makes it possible to derive updated pdf's on the basis of all kind of additional information, as, e.g., those derived by experimental tests.
The prior density can be fitted empirically to observations in past experience: occasionally, subjective assignment has to be made. Depending on the probability density functions assumed for the "a priori" and "a posteriori" models, the problem's solution can be carried out in closed form or numerically. In the example case, the updating of the pdf's of the material properties has been carried out assuming that both the mean value and the variance are unknown.
Therefore, denoting by X a material property, by μ the mean value of X and by σ^2 its variance, the joint prior density of μ and σ^2 is expressed as the product of a conditional log-normal density LN $([\mu],\tau\sigma^2)$ and an inverted gamma density IG (α,β). The calibration of the parameters of the posterior pdf of X, conditional on the results of the experimental tests (represented by a vector $\underline{r}$ of actual observations upon X) can be performed by means of the updating procedure illustrated in [6] or [8].

2.2 "Extended" use

The Bayesian approach can also be useful to deal with the results of experimental tests furnishing more general information on structural behaviour, then calling into play a number of stochastic parameters. In fact those results which form one or more conditions on the vector of the basic r.v.s $\underline{X}$, may be interpreted by defining "artificial" events corresponding to functional relationships between the X_i. Such relations derive from the analytical model of structural response that is utilized to interpret the particular kind of test performed. The "artificial" events are expressed in the form:

(a) $H_r(\underline{y}) \le 0$ $r = 1,2, \ldots n$

(b) $H_s(\underline{y}) = 0$ $s = 1,2, \ldots m$

where $\underline{Y}$ is the vector of the basic r.v.s $\underline{X}$ plus others variables. These others are called into play by the particular type of test, or directly included in the analytical model to explicitly characterize the uncertainty attributed to the experimental results and to the analytical model itself.
Examples of type (a) artificial events are represented by proof loading results, where it has been ascertained that the strength of the structure is larger than the applied load; examples of type (b) artificial events are represented by direct measurements of derived quantities, that must be expressed by means of an analytical model as a function of the basic r.v.s. It is obvious that more complex experimental tests may provide information which can be represented with several events of type (a) and (b).
When put in this form, the additional information can be applied directly to the updating of the failure probabilities estimated a priori. In fact, updating the estimate of the structural reliability with respect to a given limit state by means of additional information is fairly simple if the analysis is performed by means of the advanced First-Order Second-Moment methods. The updating procedure requires the evaluation of the conditional probability expressed by the relation:

$$P_f = P\{H \le 0 | H_1 \le 0 \cap \ldots\ldots \cap H_n \le 0 \cap H_{n+1} = 0 \cap \ldots\ldots \cap H_{n+m} = 0\}$$

where: $\{H(\underline{x})\}$ is the limit state function corresponding to the limit state considered, and the experimental tests furnish data that can be interpreted by means of n type (a) conditions and m type (b) conditions. The methods for evaluating the conditional probability are set forth in [2] and [9].

3. AN EXAMPLE CASE

The basic concepts outlined in the preceding paragraphs have been applied to the reliability assessment of an important R.C. building in Turin, designed by Italian engineer Giacomo Mattè Trucco to serve as an industrial plant for the production of cars and industrial vehicles, and now re-analyzed in view of a change in its usage assignment. The main body of the complex, built between 1916 and 1920, consists of two parallel identical 5 storey buildings, connected to each

other by means of transversal elements located every 120 m; the total length of the complex is about 556 m. At the ends of the main body are two ramps, built in 1925-1926 and of helical form, which allow vehicles to reach the flat roof, where there is a test track with banked curves.
In order to assess the structure reliability, an extensive campaign of investigation was planned and developed, this comprising, besides the search for and the analysis of the original drawings, and the survey made of the effective shape and dimensions of the structural elements:

- compression tests on concrete samples, which were cored from the main columns (a total number of 49 samples were tested);
- the measurement of ultrasonic pulse velocity in the column cores. These tests were performed on 5638 columns, the final result of each test being taken equal to the mean of two measurements, made near the bottom and near the top of each column;
- the measurement of the rebound Schmidt hammer index (in 3082 different positions);
- the measurement of the electrical potential in order to evaluate any corrosion of the reinforcement (a total number of 238 tests were developed);
- the evaluation of the depth of carbonation, by means of phenolphthalein tests;
- tensile tests on reinforcing bars cut off from the structure (22 specimens);
- compression tests on entire columns, cut out of the structure where some demolition was required by the architectural restructuring design (3 tests);
- load tests on beams and decks.

The preliminary structural analysis, made using the results of a first series of tests, led to the conclusion that the horizontal elements (beams and decks) should have a satisfactory bearing capacity, therefore requiring only the repair of local damage; but all columns located on the first and second levels seemed to be critical, and some of those on the third level too.
A more refined analysis was then performed in order to evaluate the failure probability of the columns. Taking into account the possibility that different contractors worked at the same time in different parts of the structure, the safety check was performed independently for each building portion delimited by two adjacent construction joints.
The main steps of the analysis carried out in order to verify the need for any upgrading were:

- the evaluation of the mechanical properties of the materials (i.e., of the probability density functions of the concrete compression strength and of the steel tensile strength);
- the safety check of the columns;
- the updating of the failure probabilities derived in the previous step, and according to the results of the direct compression test made on a column sample.

The main results of the analysis are summarized in what follows, with special reference to two different zones: the northern ramp and a zone of the main building, called Zone 1. The northern ramp was erected in 1925. It is helical in form, and is supported by columns located along the internal and external ramp perimeter, and the structure of the main building is very regular, and consists of span equal to 6 m in both directions.

3.1 Evaluation of the probability density functions of the strengths of concrete and steel

3.1.1 Concrete

A preliminary sensitivity analysis has shown that concrete compression strength is of major importance to the safety check of the columns. Therefore, to obtain the most accurate evaluation of the pdf of this variable, it is mandatory that proper account be taken of a number of available information items.

Assumption of the prior probability density function

The evaluation of the parameters of the prior pdf of the concrete strength is very difficult because no indications were found in the original design documents, nor were Italian Standards for reinforced concrete buildings available at the time of the construction. Therefore, the prior pdf must be derived only on the basis of the experimental results got from similar buildings of the same age. In this case, the measurements made on an industrial building erected in the same years in Venice were available [7], giving a mean value of the strength of 16 MPa. For safety, a mean value of 15 Mpa and a coefficient of variation of 0.5 were assumed. The prior pdf is thus (in Mpa):

$$f_c = LN(15;56.25)$$

Updating on the basis of core tests

Compression strength tests were performed on 49 cores, taken from the buildings at different levels. From the tests results, a mean value of 15.73 Mpa was obtained, together with a standard deviation of 5.82 MPa, corresponding to a coefficient of variation (c.o.v.) of 37%. The large value of the c.o.v. corroborates the assumption of poor homogeneity of concrete in different zones. Applying the updating procedure illustrated in Sec. 2.1, the posterior density function is:

$$f_c' = LN(15.72;35.58)$$

Correlation between the ultrasonic pulse velocity and concrete strength

From the 5638 ultrasonic pulse velocity tests, the 49 values obtained on the columns from which the cores were taken are considered. In Fig. 1 the measurements on cores of pulse velocity and of the compression strength are compared: the scatter appears to be quite large. Nevertheless, the data was fit to a correlation function between the ultrasonic pulse velocity and the concrete compression strength in the form:

$$f_c = c_1 \cdot \exp(c_2 \cdot V) \cdot \varepsilon$$

where: V is the ultrasonic pulse velocity; c_1 and c_2 are constants to be determined in order to fit experimental data; ε is a r.v. measuring the model uncertainty associated with the form of the correlation curve and with the scatter around the mean of the results obtained by the correlation.
The values of c_1 and c_2 corresponding to the best fit of experimental data, derived by means of a non-linear regression procedure, are equal to: 2.6792 and 0.0005, respectively. The above relationship is represented by the solid line in Fig. 1. The standard error of the correlation is about 0.29, so that the pdf of the r.v. ε can be taken equal to:

$$\varepsilon = LN\ (1;0.45)$$

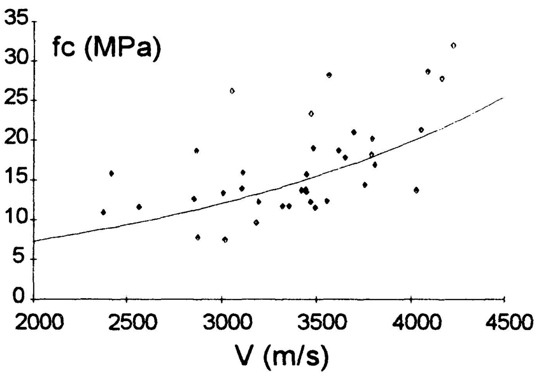

Fig. 1 Correlation between V and f_c

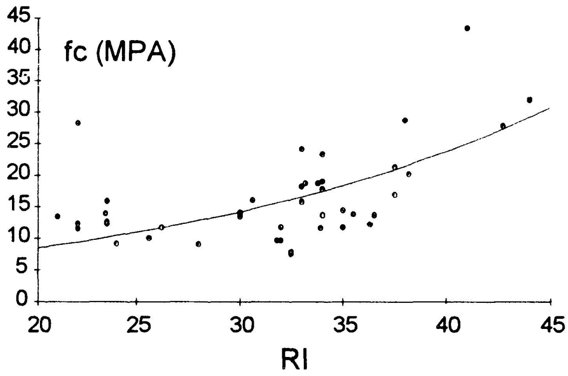

Fig. 2 Correlation between RI and f_c

Correlation between the rebound Schmidt hammer index and the concrete compression strength

The rebound Schmidt hammer test derives the concrete compression strength from the amount of rebound at the surface of the structural element. Many calibration tests are available to validate the results of this non destructive technique; however its considerable uncertainty owes mainly to the need to establish a relation between the Young's modulus of elasticity (conditioning the amount of rebound) and the concrete compression strength. Unfortunately, in the case of very old concretes, another source of uncertainty must be considered: in fact, the test refers exclusively to the surface of the structural element, where carbonation produces a local increase in strength. Consequently, the use of the correlation diagram accompanying the instrument would produce a serious overestimation of the strength. Therefore, the original correlation diagram was disregarded, and a new one was sought directly using the results on cores, as was done in the case of the ultrasonic pulse velocity tests.

Considering the rebound Schmidt hammer results on the same columns from which the cores were taken, the results of Fig. 2 are obtained. The same figure shows the best fit obtained with the relationship:

$$f_c = c_3 \cdot \exp(c_4 \cdot N) \cdot \varepsilon$$

where: N is the rebound Schmidt hammer index; c_3 and c_4 are constant coefficients, whose values, derived by means of a non-linear regression procedure, are equal to: 3.023 and 0.0516, respectively. The above relationship is represented by the solid line in Fig. 2. The standard error of the correlation is about 0.29, so that the pdf of the r.v. ε can be taken as:

$$\varepsilon = LN\ (1;0.45)$$

Correlation between the pdf of the ultrasonic pulse velocity and the rebound Schmidt hammer index and the pdf of the concrete strength

Considering the local values of the ultrasonic pulse velocity, the parameters of the concrete compression strength pdf are deduced as follows.
For the northern ramp, 38 measurements are avaliable, giving:

$$E[V] = 3872 m/s \qquad Var[V] = 44100 (m/s)^2$$

therefore, applying the procedure outlined in Sec. 2.1:

$$E[f_c] = 20.49 MPa \qquad Var[f_c] = 90.56 (Mpa)^2$$

For the 1st level of Zone 1, 176 measurements are available, giving analogously:

$$E[V] = 3507 m/s \qquad Var[V] = 71829 (m/s)^2$$

$$E[f_c] = 18.22 MPa \qquad Var[f_c] = 127.7 (MPa)^2$$

Considering the local values of the rebound Schmidt hammer index, the parameters of the concrete compression strength pdf are derived as follows.
For the northern ramp, 38 measurements are avaliable, giving:

$$E[RI] = 37.45 \qquad Var[RI] = 28.51$$

$$E[f_c] = 23.78 MPa \qquad Var[f_c] = 168.57 (Mpa)^2$$

For the 1st level of Zone 1, 38 measurements are available, giving:

$$E[RI] = 29.04 \qquad Var[RI] = 24.38 (m/s)^2$$

$$E[f_c] = 15.49 MPa \qquad Var[f_c] = 68.23 (MPa)^2$$

Combination of the results derived from ultrasonic pulse velocity and rebound hammer tests

The two resulting densities are then combined, a weighting being attributed to each of them, whose value is subjectively set on the basis of the degree of confidence given to the various tests. In this case, the parameter λ, representing the relative weight attributed to the first test method [(1 - λ) being the weigthing factor for the second one], is assumed equal to 0.6, in order to take into account the greater uncertainty associated to the rebound Schmidt hammer test, due to the effects of carbonation.
Consequently, the posterior pdf of the concrete compression strength is:

$$f_c = LN(21.11; 48.34)$$

for the northern ramp, and:

$$f_c = LN(16.56; 32.67)$$

for the 1st floor of Zone 1.

3.2 Steel

Assumption of the prior pdf

As already discussed for the concrete compression strength, no useful indications can be derived from the design documents or the Italian Standards. Therefore, and referring to the same industrial building in Venice, the following prior pdf is assumed for the steel tensile strength:

$$f_y = LN(320;6400)$$

Updating with results of tensile tests

Visual inspection of the reinforcing bars in the structural elements brought out the existence of three kinds of rebars: round section rebars; elliptical folded section rebars; small rectangular section rebars. To evaluate the mechanical properties of the reinforcing steel in the columns, 22 test were available, giving a mean strength value equal to 352.6 MPa and a standard deviation of 56.41 MPa. Applying the updating procedure, the posterior pdf is then:

$$f_y' = LN(347;19710)$$

3.3 Local verification of columns

In the reliability asessment of the main columns, the basic random variables are: the concrete compressive strength (f_c); the reinforcement yield strength (f_y); the cover thickness (c); the unintentional eccentricity of the live load (e); the section heigth (h) and width (b); the intensity of the permanent load (G); and the intensity of the live load (Q). The main characteristics of the input variables, evaluated for the most loaded columns, are reported in Table 1.

Table 1 *Parameters of the basic variables*

	TYPE	Northern ramp	Zone 1 1st level	Zone 1 2nd level
Cover thickness (mm)	*N*	*50 / 0.40*	*50 / 0.40*	*50 / 0.40*
Eccentricity (mm)	*N*	*80 / 0.50*	*50 / 0.50*	*50 / 0.50*
Section heigth (mm)	*N*	*600 / 0.10*	*600 / 0.10*	*600 / 0.10*
Section width (mm)	*N*	*800 / 0.10*	*600 / 0.10*	*600 / 0.10*
Permanent load (kN)	*LN*	*1367 / 0.05*	*1928 / 0.05*	*1594 / 0.05*
Variable load (kN)	*LN*	*690 / 0.15*	*810 / 0.15*	*594 / 0.15*

The amount of reinforcement in the columns section is equal to 3768 mm^2 in the northern ramp, and 1848 mm^2 in the Zone 1.
The limit state function is derived considering the ultimate limit state of the base section of the columns subjected to bending and compression.
Assuming for the materials the prior pdf's, the values of the safety index β and of the probability of failure P_f reported in Table 2 are obtained. If, instead, the posterior pdf's corresponding to updating according to core tests for concrete and tensile tests for steel are used, the values β' and P_f' are obtained.
A more precise evaluation can be performed using for concrete the local pdf resulting from the combination of ultrasonic pulse velocity and rebound hammer tests with core tests: the corresponding values of the safety index β'' and of the probability of failure P_f'' are reported in the same Table 2. This level of safety is satisfactory for the northern ramp, while it is doubtful for the second level of Zone 1, and insufficient for the first level of the same Zone. A direct updating of P_f on the basis of the results of destructive tests was then decided. Taking advantage of the necessity of demolishing one span to erect a staircase, three full-scale samples of column were tested until collapse: the ultimate resistance for the columns of interest resulted equal to 6768 kN. Characterizing this result as a normal r.v. (with a c.o.v. equal to 0.20, to account for measurements uncertainty, and, mostly, for differences between the tested specimen and the other columns), the direct updating procedure of Sec. 2.2 has been applied.

The corresponding "artificial" event (i.e., the comparison between the theoretical ultimate N_u and the measured N_{proof} normal force) forms a type (a) condition on the entire set of the vector of the r.v.s. The results of this updating are reported in the last column of Table 2.
The reliability of second level of Zone 1 resulted completely satisfactory, while it was decided to upgrade the most loaded columns of the first level of Zone 1.

Tabel 2 *Values of the safety index β and of the probability of failure P_f for the various updating*

	β / P_f	β'/ P_f'	β''/ P_f''	β'''/ P_f'''
northern ramp	$2.33 / .99 \cdot 10^{-2}$	$3.31 / .47 \cdot 10^{-3}$	$4.92 / .43 \cdot 10^{-6}$	$5.72 / .54 \cdot 10^{-8}$
zone 1 - 1st level	$1.67 / .54 \cdot 10^{-1}$	$2.37 / .88 \cdot 10^{-2}$	$2.97 / .15 \cdot 10^{-2}$	$3.90 / .48 \cdot 10^{-4}$
zone 1 - 2nd level	$2.17 / .15 \cdot 10^{-1}$	$3.10 / .95 \cdot 10^{-3}$	$4.29 / .91 \cdot 10^{-5}$	$4.77 / .94 \cdot 10^{-6}$

4. CONCLUSIONS

Bayesian Inference has proved to be a powerful procedure for improving knowledge of the properties of materials in existing structures, and of the bearing capacity of the structural system. The coupling of Bayesian Inference with FORM or SORM methods provides a very straigthforward process for directly updating the failure probability of a structure, taking advantage of load tests. The cost of in situ testing a structure and the increased complexity of the calculations are usually more than compensated for by the saving made possible by the improved knowledge of the capability of the structure. In the example case presented here, a very accurate in situ investigation has been performed, producing a large amount of data. Owing to the statistical processing within the rational framework of Bayesian Inference, and using the FORM method, the amount of strengthening required has been notably reduced.

ACKNOWLEDGMENT: The authors are grateful to the owner LINGOTTO S.r.l. and to the consulting firm FIAT ENGINEERING S.r.l., which co-ordinates the redesign process, for kindly having made available the test results and the design data on the Lingotto building in Turin.

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Safety Criteria for the Evaluation of Existing Structures

Critères de sécurité pour l'évaluation de structures existantes

Sicherheitskriterien für die Bewertung bestehender Tragwerke

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SUMMARY

Limit states criteria which conform to a single conservative safety level are appropriate for the design of new structures because the savings in adopting different safety levels is marginal for any project. The cost and disruption in not meeting such a safety level, however, can become a major obstacle for an existing structure. For this reason different safety levels are being introduced in Canada for the evaluation of existing structures based on a life safety criterion. The recommended safety differentiation allows more flexibility in practice but requires more professional judgement. The paper describes the derivation of safety criteria for the evaluation of bridges and buildings in Canada.

RÉSUMÉ

Les dimensionnements aux états limites, qui correspondent à un niveau de sécurité unitaire et conservateur doivent être considérés comme adéquats pour le calcul de nouvelles structures, étant donné que les possibilités d'économie sont relativement minimes pour différents degrés de sécurité. Toutefois pour des structures existantes, les coûts et l'interruption d'utilisation en vue d'atteindre un tel degré de sécurité peuvent s'avérer prohibitifs. Raison pour laquelle différents degrés de sécurité, qui se basent sur un critère de sécurité de la durée de vie, ont été introduits au Canada en vue de pouvoir apprécier les structures existantes. Cette différenciation permet davantage de flexibilité dans la pratique, mais exige un supplément d'aptitude dans l'appréciation professionnelle.

ZUSAMMENFASSUNG

Bemessungen nach Grenzzuständen, die einem einheitlichen konservativen Sicherheitsniveau entsprechen, sind für neue Bauwerke durchaus angemessen, da die Einsparungsmöglichkeiten durch Annahme unterschiedlicher Niveaus gering sind. Für bestehende Tragwerke können jedoch der Aufwand und die Nutzungsunterbrechung, um solch ein Sicherheitsniveau zu erreichen, prohibitiv werden. Für die Bewertung bestehender Tragwerke wurden deshalb in Kanada unterschiedliche Sicherheitsniveaus eingeführt, die auf einem Konzept der Lebenssicherheit fussen. Diese Differenzierung gestattet mehr Flexibilität in der Praxis, erfordert aber zusätzlich professionelle Urteilskraft.

1. INTRODUCTION

There are increasing pressures to preserve and maintain existing structures such as buildings and bridges for as long as possible with a minimum of structural intervention. The pressures derive primarily from the cost of upgrading but include also user disruption, energy conservation and heritage value. These pressures, along with the fact that the structure exists, has performed satisfactorily and has been inspected for defects means that the criteria for evaluation of existing structures for continued use need not be as conservative as for the design of new structures. The following describes the basis for minimum safety levels for the evaluation of existing structures under development in Canada.

2. SAFETY CRITERION

The following safety concepts can be applied to determine appropriate safety levels for civil engineering structures:

1. Probability of failure (P_f) or reliability index (β). In Canada a reliability index of 3.5 is used for the design of bridges. For buildings it varies but is approximately the same. An exception is connectors such as bolts and welds for which, β is of order 5.

2. Life safety, or probability of death or injury for persons exposed to structural hazards. This considers, in addition to the probability of failure, the likelihood of death or injury if failure occurs, as well as other factors such as the activity and number of people at risk.

3. Optimum hazard reduction. This concept applies to an inventory of existing structures that contain structural hazards. The objective is to gradually reduce these hazards in accordance with benefit in reduced risk vs. cost. Such an approach is being carried out to reduce the seismic hazards posed by existing buildings in Los Angeles.

4. Damage control. There may be life safety, economic and other reasons not only to prevent collapse but to control structural damage as well. For hospitals, for example, damage control becomes a life safety issue in the event of a disaster such as an earthquake or hurricane.

Each concept has its applications depending on the project under consideration. It is, however, possible to identify minimum safety levels for 'ordinary' structures based on life safety. These minimum safety levels must be adjusted upwards for evaluation of special structures such as hospitals, key bridges or communication towers, depending on the consequences of failure or damage. Also, based on life cycle considerations, it often becomes economical to follow current design criteria if structural upgrading is required.

The following life safety criterion is used to determine minimum safety levels for structural evaluation [1]:

$$P_f = \frac{TAK}{W\sqrt{n}} \tag{1}$$

where

P_f = target probability of failure based on life safety (this is a notional probability for setting technical criteria, not an actuarial one)

K = calibration factor based on experience with existing criteria

A = human activity factor which reflects what risk is acceptable in relation to other non-structural hazards associated with the activity (taken as 1 for buildings, 3 for bridges, 10 for certain work-related activities [1])

W = warning factor corresponding to the likelihood that, given failure or recognition of approaching failure, a person at risk will be killed or seriously injured (W=1.0 for impact with no warning)

$\sqrt{n}$ = importance factor based on the number of people likely to be at risk if failure occurs, essentially an aversion factor based on public reaction to high fatality hazards

T = assumed reference period

3. CALIBRATION TO DESIGN CRITERIA

It is well known that life-threatening structural collapses are relatively rare, furthermore most are due to human error or accidents not addressed by current design criteria. Therefore current design criteria, if correctly applied, provide a safe upper bound to the life safety criterion, Equation (1). This assumption can be used by considering the ratio of the target probability of failure for evaluation to the target probability of failure for design where, from Equation (1):

$$\frac{P_{fe}}{P_{fd}} = \frac{A_e}{A_d} \cdot \frac{W_d}{W_e} \cdot \frac{\sqrt{n_d}}{\sqrt{n_e}} \tag{2}$$

where the subscripts d and e refer to design and evaluation respectively.

Because of the logarithmic relationship between P_f and β, the ratio P_{fe}/P_{fd} can be approximated by an adjustment in target reliability index, i.e.

$$\Delta = \beta_d - \beta_e \tag{3}$$

where β_d and β_e are the target reliability indices corresponding to the target failure probabilities P_{fe} and P_{fd} determined from the standard normal distribution curve. For example, $\Delta = 0.5$ corresponds to P_{fe}/P_{fd} of approximately 1/5 for β_d in the range 2.5 to 3.5.

4. EVALUATION FACTORS FOR DETERMINING SAFETY LEVELS

If the ratios W_d/W_e, $\sqrt{n_d}/\sqrt{n_e}$ and A_e/A_d can be determined for evaluation as compared to design then the target reliability index β_e can be determined from Eqn. (3) and safety factors determined by current reliability techniques. The factor W, however, is not easy to assess in practice. Factors that can be assessed by the structural evaluator which affect W include the following:

- component behaviour: If a component fails gradually then failure is likely to be noticed before collapse takes place allowing time to avoid life-threatening consequences.

- system behaviour: If a component fails without collapse because of alternate paths of support (redundancy) then the risk to life is considerably reduced.

- inspection: Inspection affects the warning factor W by providing clues of approaching or potential failure in time to avoid life-threatening consequences.

These factors, along with risk category which is related to $\sqrt{n}$ and A, are listed in Table 1 along with a comment as to whether or not they are taken into account in current Canadian structural design codes for bridges and buildings.

Table 1 Structural Evaluation Factors Affecting Risk to Life

Evaluation Factor	Parameter in Eqn (2)	Factor Taken into Account by: Bridge Code[2]	Factor Taken into Account by: Building Code[3]
Component Behaviour	W	no	yes
System Behaviour	W	no	no*
Inspection	W	no	no
Risk Category	$\sqrt{n}$ and A	no	no**

* partly, for earthquake only
** only on the basis of building use and occupancy

5. APPLICATION TO BRIDGE EVALUATION

Minimum safety levels for bridge evaluation under traffic load have been developed based on the above approach [4] and incorporated in the Canadian bridge code [2]. The safety levels are expressed in terms of a target reliability index given in Table 2, adjusted as a function of the four evaluation factors in Table 1. The reliability index adjustment, Δ , is made up of contributions from each of the four evaluation factors. The maximum contribution for each factor is based partly on a consideration of the values of the life safety factors in Eqn (2) and partly on existing criteria used in other codes. A maximum Δ of 0.5 for component or system behaviour, for example, corresponds to an assumed likelihood of death/injury if failure occurs of approximately 1 in 5, or 1 in 25 for both together. A Δ of 0.5 is applied for supervised passage of an overloaded vehicle, because all other traffic is kept off the bridge, which reduces the factor $\sqrt{n_e}$ in Equation (2), and only the driver is at risk, which increases the factor A_e in Equation (2).

Table 2 Reliability Index, β_e, for Bridge Evaluation

$$\beta_e = 3.5 - [\Delta_C + \Delta_S + \Delta_I + \Delta_R] \geq 2.0$$

where β_e is based on a one-year time interval for all traffic categories except for supervised overload, where β_e is based on a single passage.

Adjustment for Component Behavior	Δ_C
Sudden loss of capacity with little or no warning	0.0
Sudden failure with little or no warning but retention of post failure capacity	0.25
Gradual failure with probable warning	0.5
Adjustment for System Behaviour	Δ_S
Element failure leads to total collapse	0.0
Element failure probably does not lead to total collapse	0.25
Element failure leads to local failure only	0.5
Adjustment for Inspection Level	Δ_I
Component not inspectable	-0.25
Component regularly inspected	0.0
Critical component inspected by evaluator	0.25
Adjustment for Risk Category	Δ_R
All traffic categories except supervised overload	0.0
Supervised overload	0.5

The total range of β_e in Table 2 is from 1.75 to 3.75, where the upper limit, 3.75, corresponds to a safety equivalent to that assumed for design [2]. The lower limit, which occurs only for supervised overload, represents an economic risk to the bridge authority (theoretically 1/25 times the loss if failure occurs); a lower limit of β_e=2.00 was therefore imposed. Most traffic networks have considerable flexibility if a bridge failure takes place but in some cases the effect of a bridge failure on the local economy can be severe. In such cases the lower limit for β_e should be increased.

The target reliability index in Table 2 was used to develop load and resistance factors for the evaluation of bridges in the Canadian bridge code [5].

6. APPLICATION TO BUILDING EVALUATION

The same basic approach has recently been applied to buildings [6]. Although the basis is the same as for bridges, the method was altered. The reason for this is that the confidence in reliability methods is much greater for bridges under traffic load than for a wide variety of buildings under a wide variety of loads, including earthquake. Instead of recommending reduced target safety indices for building evaluation it is more practical to recommend reduced load factors. These were determined by use of the following log normal relationship [6]

$$\alpha_e = \alpha_d \exp\left[-\Delta\sqrt{(1+V_R^2)(1+V_S^2)}\right] \quad [4]$$

where α_d is the design load factor and α_e is the evaluation load factor, Δ the target safety index adjustment. V_r and V_s are the coefficients of variation representing the uncertainties of resistance and load respectively. Based on assumptions for V_r and V_s given in Table 3, Figures 1-3 show the relationship between load factor and the target reliability index adjustment Δ. Based on Figures 1-3, Table 4 contains recommended load factors for building evaluation.

Table 3 Uncertainty Assumptions for Estimating Load Factors for Buildings

	Uncertainty
Load	V_S
Dead	0.1
Variable*	0.3
Earthquake	1.1
Resistance	V_R
Steel	0.1-0.15
Concrete	0.15-0.2
Masonry	0.2-0.3
Wood	0.3

* Occupancy, snow and wind loads

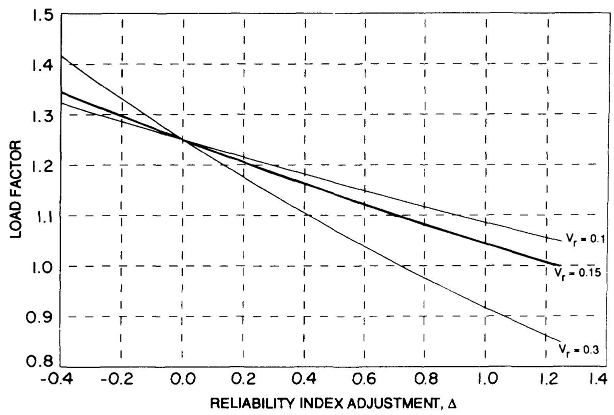

Fig. 1 Dead load factor (V_S=0.1)

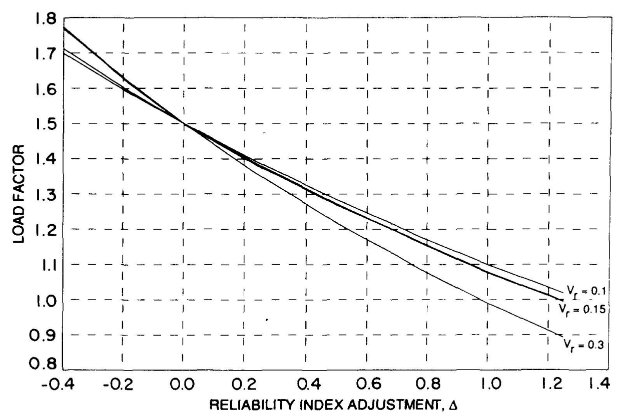

Fig. 2 Variable Load Factor (V_S=0.3)

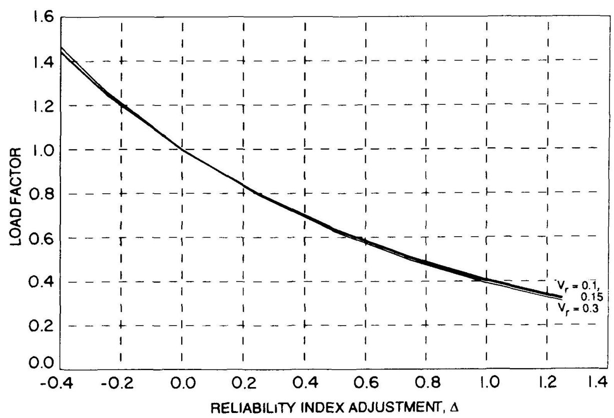

Fig. 3 Earthquake load factor (V_S=1.1)

Table 4 Load Factors for Building Evaluation

Adjustment to Design Safety Level $\Delta = (\Delta_s + \Delta_R + \Delta_P)$†	Load Factor for:		
	Dead Load*	Variable Loads	Earthquake
0	1.25 (0.85)	1.50	1.00
0.25	1.20 (0.88)	1.40	0.80
0.5	1.15 (0.91)	1.30	0.63
0.75	1.11 (0.93)	1.20	0.50
1.0 or more	1.08 (0.95)	1.10	0.40

† Adjustment for System Behaviour	Δ_S
-failure leads to collapse, likely to impact occupants	0.0
-failure is unlikely to lead to collapse, or unlikely to impact occupants	0.25
-failure is local only, very unlikely to impact occupants	0.5
† Adjustment for Risk Category	Δ_R
-high building importance or high occupancy exposed to failure	0.0
-normal occupancy exposed to failure	0.25
-low occupancy exposed to failure	0.5
† Adjustment for Past Performance	Δ_P
-no record of satisfactory past performance	0.0
-satisfactory past performance** or dead load measured***	0.25

* The value in the brackets applies when dead load resists failure

** Apply only to dead and variable load factors, age 50 years or more, no significant deterioration.

*** Apply to dead load factor only.

Two evaluation factors in Table 1 were not included in Table 4, namely 'component behaviour' because it is already taken into account in current design criteria, and 'inspection' because building structures are not inspected on a regular basis and therefore warning is not reliable. The risk category for occupancy in Table 4 (high, normal, low) can be estimated on the basis of floor area exposed to potential collapse if the failure occurs, occupant density and duration of occupancy (hours per week).

A new evaluation factor 'past performance' is included, however not because it affects the life safety criterion Equation (1), but because it reduces the uncertainty in estimating loads and resistance compared to design. Dead load parameters, for example, may be measured, and the corresponding reduction on uncertainty (V_S from 0.1 to 0.05) corresponds to a Δ of 0.25 [6]. More significant, however, is satisfactory past performance over many years under dead and variable loads such as wind and snow.

Successful past performance, however, is difficult to quantify in terms of reduced safety coefficients. Table 4 contains a conservative adjustment, Δ=0.25, the same as for measured dead load [5].

Besides the load factor adjustments contained in Table 4, there will also be adjustments in the resistance factors for components such as bolts and welds.

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Experimental Investigation of Traffic Load on Highway Bridges
Vérifications expérimentales de la charge mobile sur les ponts-routes
Experimentelle Untersuchungen der Verkehrslast auf Strassenbrücken

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SUMMARY

The dynamic response of two highway bridges subject to normal road traffic load has been investigated in order to obtain a file of statistical data as well as to identify their dynamic characteristics and to compare the behaviour of both bridges. A measuring and computing system was used for recording analog signals of the bridge deflection together with their subsequent digitization with statistical processing of the measured data by computer. This resulted in more representative data of dynamic and traffic parameters of bridges.

RÉSUMÉ

Le comportement sous effets dynamiques a été mesuré sur deux ponts-routes sous charges mobile normale, afin d'obtenir des données statistiques relatives aux charges, des renseignements sur le comportement aux vibrations propres et de pouvoir comparer ainsi les deux ouvrages. Pour ce faire, il a été fait appel à un système de mesure avec enregistrement analogique automatique des flèches; ces dernières, après avoir subi une conversion numérique, ont été soumises à un dépouillement statistique des données représentatives des paramètres et des sollicitations dynamiques des ponts.

ZUSAMMENFASSUNG

Das dynamische Verhalten zweier Autobahnbrücken unter normaler Verkehrsbelastung wurde gemessen, um statistische Belastungsdaten, aber auch Aufschluss über das Eigenschwingverhalten und einen Vergleich beider Brücken zu erhalten. Dabei wurde ein Messsystem mit automatischer Analogaufzeichnung der Durchbiegungen verwendet, die digitalisiert und einer statistichen Auswertung unterzogen wurden. Das Ergebnis sind repräsentative Daten der dynamischen Parameter und Beanspruchung der Brücken.

1. INTRODUCTION

From the point of view of service reliability of a bridge construction as well as an increase in traffic intensity on highways together with an effort to apply the knowledge of theory of reliability to the design of this type of structures we need to clarify many partial problems connected with the solution of the system *bridge-loading-environment*. At present the main task is to obtain a true picture of the load magnitude and its effects on the structure. For this reason it is clear that bridges which are subjected to considerable dead and moving load as well as to a secondary load deserve a maximum attenttion.

2. THE MEASURING AND COMPUTING SYSTEMS

A measuring system (Fig. 1) has been developed to provide measuring time variations of the bridge deflection, enconding the passing trucks into 8 + 2 categories (Table 1) and sensing their axles together with the directions of the drive. Both, analog signals (deflections) and discrete data (traffic parameters) are stored by an instrumentation tape recorder. The deflection sensing device consisting of a wire stretched by a spring and an inductive displacement transducer or a stretched resistance wire between the measurement point and the terrain are used to measure the bridge deflections [1].

A minicomputer system (Fig. 2) with a real-time operating software is exploited to digitize and preprocess the recorded deflections together with the discrete data on the vehicles. The results are digital records containing just a relevant part of the digitized deflection signal and a block of the data on the trucks which have evoked previous responses.

A Fortran IV programme computes dynamic parameters (Fig. 3) as they are defined by the Czechoslovak Standard 73 6209 [2]. The digital filtering is used to obtain a mean deflection curve and to select a pure dynamic portion of the signal. The computation results in the dynamic parameters for each relevant response of the bridge to the passing trucks together with the discrete data on the trucks.

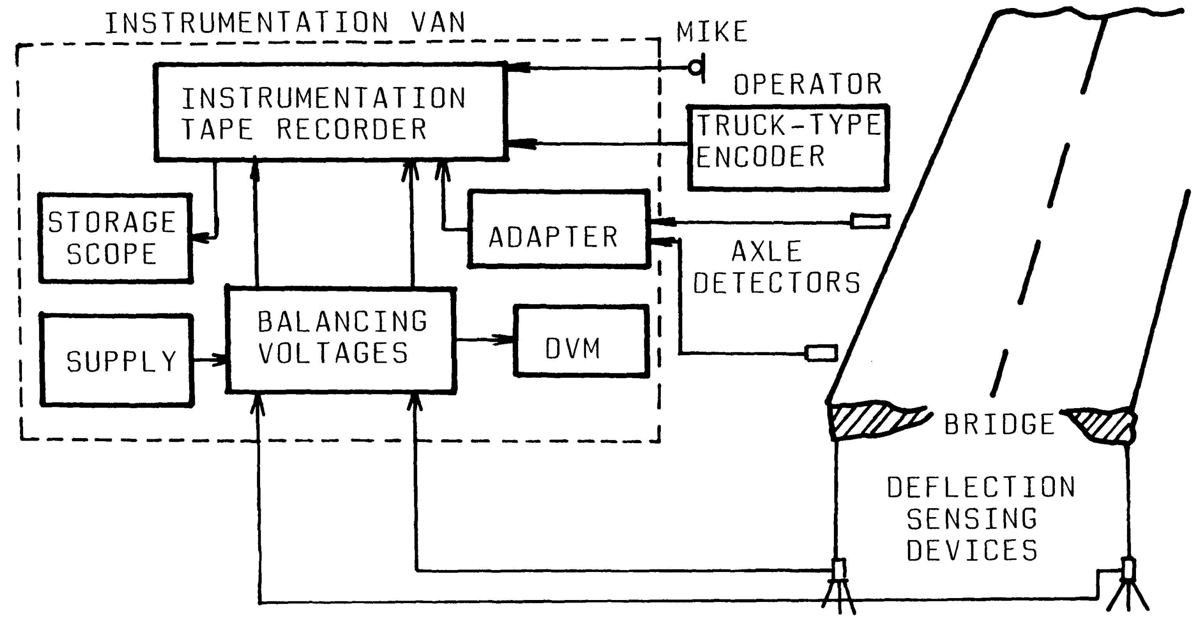


Fig. 1 The measuring system for monitoring bridge deflections and some traffic parameters

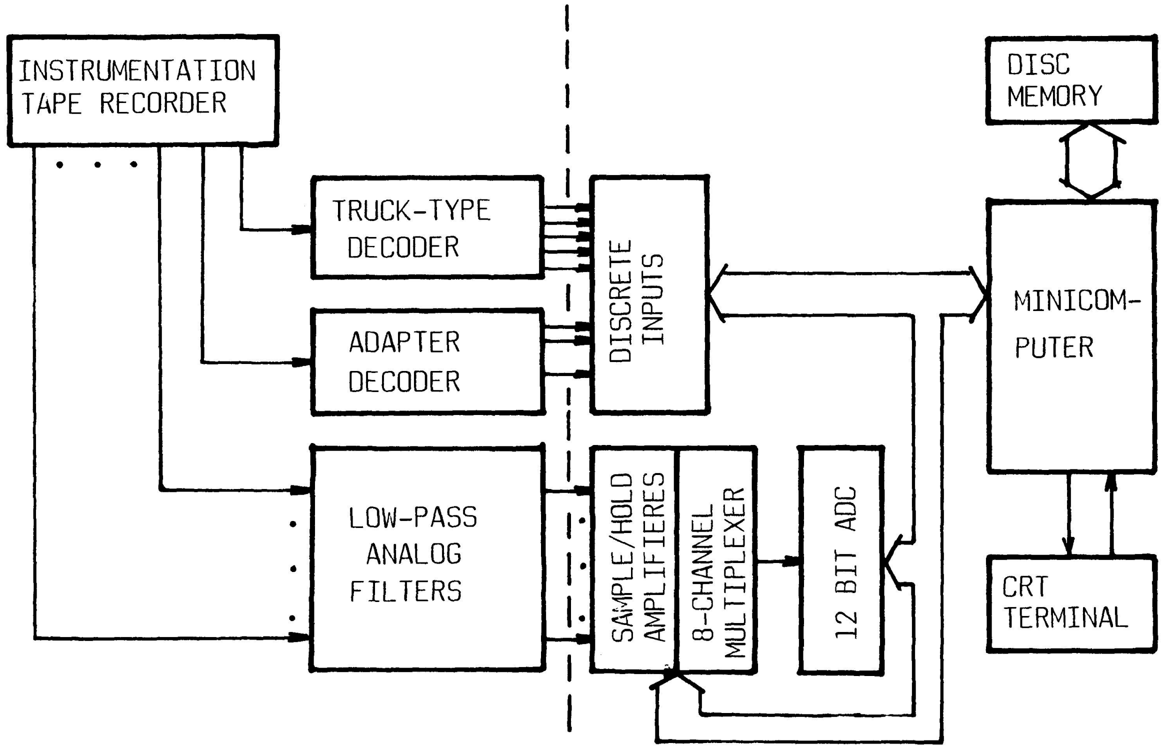


Fig. 2 The computing system for preprocessing the recorded analog and discrete signals

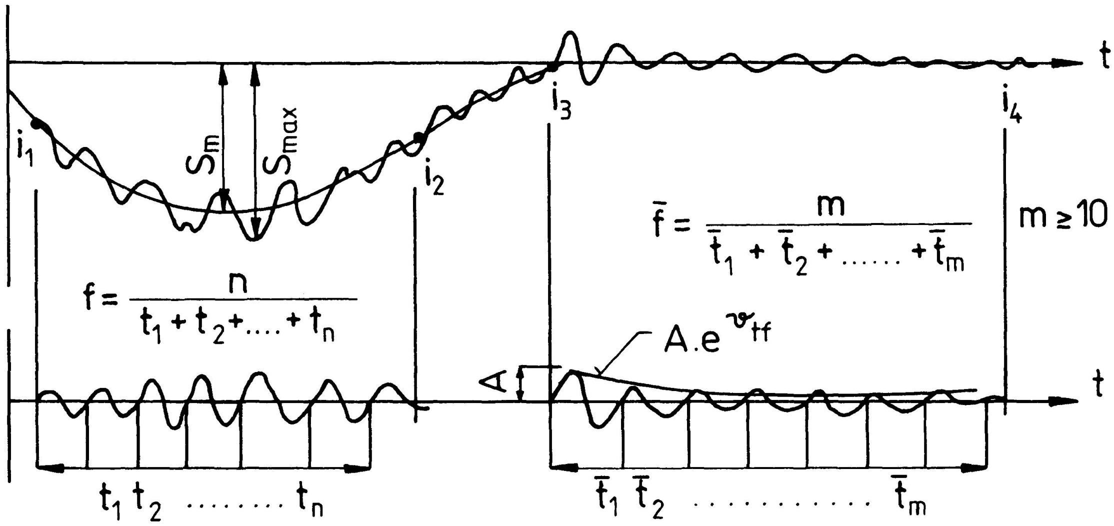


Fig. 3 Illustration of computing the natural frequencies f and $\bar{f}$, the damping ϑ and the dynamic coefficient S_{max}/S_m

A nonrecursive low-pass filter produces a mean deflection curve. The first and the second differences of this curve are continually evaluated as to locate the inflection points i_1, i_2 and the minimum S_m. At the interval $<i_1, i_2>$ vibrations of the loaded bridge are investigated and the S_{max} is located. To distinguish the natural frequency f from higher frequencies induced by dynamic wheel loads of the vehicles a nonrecursive band-pass filter is applied on the signal at the interval. The same filter is used to find out the natural frequency $\bar{f}$ of free decayed vibrations and the logarithmic decrement of damping ϑ. The programme processes also the encoded da ta about the vehicles, which exited the previously evaluated records of the bridge deflections. Speeds, axle bases and the categories are calculated and output as the traffic data.

3. THE LONG TERM OBSERVATION OF INFLUENCE OF THE TRAFFIC LOAD ON THE BRIDGE

The long term observation has been carried out on two one span bridges on the first class highway. Both bridges are of the same construction system. The structure is made of standard post-tensioned concrete (50 MPa) I-girders (Fig. 4). The former is of 26 m span and its construction height is 1.25 m. The latter is of 21.26 m span and is 1.1 m high. The substructures are created by massive abutment with the parallel wings. The expansion joints are placed above one abutment only.

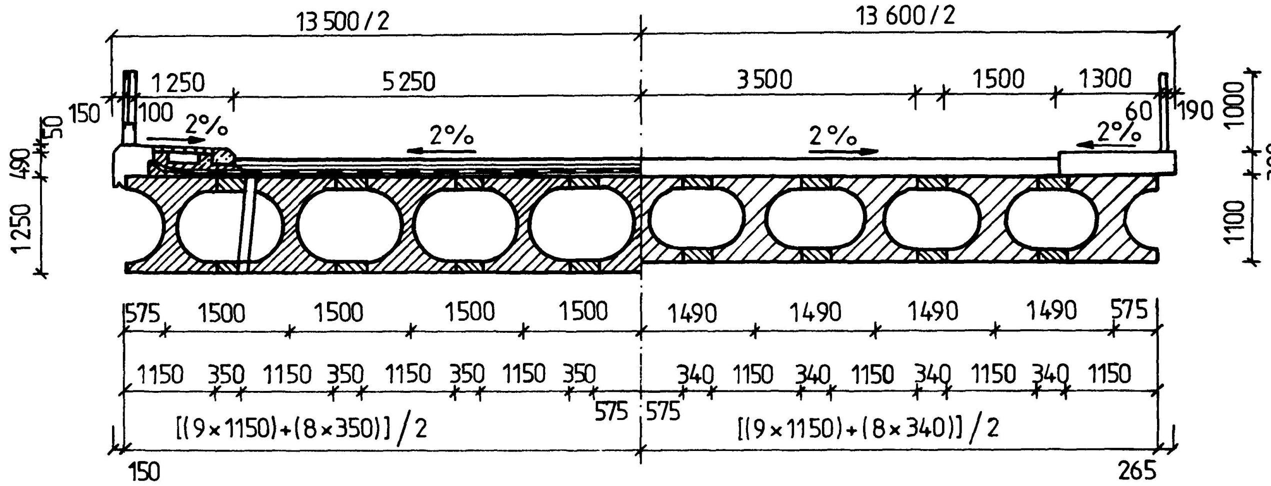


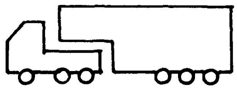
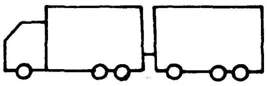
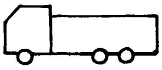
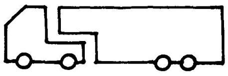
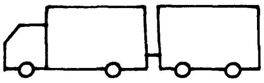
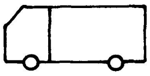
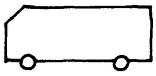
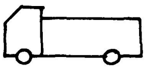
Fig. 4 Crossection of the bridge near H. Hričov and K. Lhota

The position of the bridges on the highway network is characterized by the transport intensity. We have recorded about 2.000 passing trucks on the first bridge and about 2.400 on the second one during the same period. The trucks have been encoded into 8 categories according to the number of axles, type of a truck, axle distances and bearing capacity, plus two extraordinary categories (Table 1). The operator observes the traffic and operates the recorder by the truck-type encoder. He selects the pushbutton corresponding with the category and the drive direction of the passing truck or with an extraordinary situation on the bridge.

3.1 Histograms and their statistical characterics

Having processed the whole group of the digitized deflection signals the obtained results were processed by the *Fortran programme* which compiled the measured values into the histograms of the deflection distribution for each category and drive direction of the trucks as well as for the whole group.

Then the statistical data of the histograms were computed and the aproximation by the following theoretical probability distribution was tried: *Weibull's, Gumbel's, Raleigh's, Exponential, Normal and Logarithmic-normal, Gama and Chi-model.* We have carried out the statistical testing based on the assumption that at least one of the theoretical probability distribution is realistic for distribution obtained from the measurement and that at least one of the introduced theoretical distribution would correspond to this

	SCHEME	VEHICLE TYPES	1st BRIDGE		2nd BRIDGE	
			THEORETICAL MODEL			
				No OF TRUCKS		
1		VOLVO T 813 T 138 +SEMI-TRAIL	WEIBULL´S	57	67	NORMAL
2		VOLVO T 813 T 138 TRAIL	GUMBEL´S	57	76	WEIBULL´S
3		T 111 T 813 T 138 T 148	GUMBEL´S	165	205	LOG-NORMAL
4		Š 706 Š 100 +SEMI-TRAIL	GUMBEL´S	178	210	WEIBULL´S
5		Š 706 Š 100 TRAILER	GUMBEL´S	195	265	NORMAL
6		Š 706 Š 100	GUMBEL´S	379	485	LOG-NORMAL
7		BUSES Š 706-RTO ŠL 11, ŠC 734 IKARUŠ	GUMBEL´S	153	217	LOG-NORMAL
8		V 35-ROMAN S5T, AVIA ROBUR, IFA	WEIBULL´S	371	410	FRECHET´S
9	UNIVERSAL GROUP	ALL VEHICLES DON´T INC LUDED IN GROUPS 1-8	GUMBEL´S	94	124	NORMAL
0	UNIQUE GROUP	PASSING AND MEETINGS 2 VEHICLES	GUMBEL´S	344	350	WEIBULL´S
	ALL PASSING TRUCKS		GUMBEL´S	1989	2409	WEIBULL´S

Tab. 1 The schemes of the 8 most occuring trucks plus 2 extraordinary categories and the corresponding optimal theoretical probability distribution models of the deflections for each category as well as for the whole group of the passing trucks

distribution. The parameters of the competent theoretical distribution for each group of deflection were calculated together with the significance test of each distribution [4].

Analysing the results achieved on the first bridge we have found out that the *Gumbel's* model, at the 5% significance level, is the most suitable in 72% and the one of *Weibull's* in 28% of all the histograms in which the measured deflections were divided according to the category of trucks as well as according to the both directions of drive. On the second bridge we have found out that the *Weibull's* model can be taken as an optimal theoretical one in 30% of all the histograms. For 30% of them the *Logaritmic-normal* model is the most suitable one as well as the normal one for other 30% of all the histograms. For the rest (10%) the *Frechet's* model is optimal.

3.2 Dynamic characteristics of both bridges

In Table 2 there are *natural frequencies* of the loaded and the unloaded bridges. It can be seen that the mean natural frequency of the second loaded bridge is higher approximately by 1.0 Hz than the frequency of the first one. This difference confirms the reality that with a growth of rigidity of construction its frequency grows up.

FREQUENCY (Hz)	1 st BRIDGE			2 nd BRIDGE		
	MIN	MEAN	MAX	MIN	MEAN	MAX
LOADED BRIDGE	1,000	4,700	12,500	3,900	5,667	8,820
UNLOADED BRIDGE	-	-	-	5,560	5,759	8,200

Tab. 2 The frequencies of vibrations of the bridges

Table 3 shows the values of the *logarithmic decrement of damping* (ϑ), the *dynamic coefficient* (δ) as well as the *dynamic increment* which have been measured according to the Czechoslovak Standard [2]. Here we would like to mention the fact that the dynamic coefficients have been greater by 3 to 10% in one direction of drive than in the opposite one. This difference of the dynamic influences is due to the expansion joint which lies on the abutments of both bridges in one direction only.

BRIDGE	LOGARITHMIC DECREMENT OF DAMPING (ϑ)			DYNAMIC COEFFICIENT (δ)			DYNAMIC INCREMENT		
	MIN	MEAN	MAX	MIN	MEAN	MAX	MIN	MEAN	MAX
FIRST	-	-	-	1,05	1,165	2,10	0,015	0,085	0,195
SECOND	-0,134	0,170	0,500	1,00	1,118	1,875	0,010	0,072	0,180

Tab. 3 Some dynamic characteristics of second bridge

The analysis of the correlation between the maximum deflections and the dynamic coefficients respectively, the dynamic increment has shown that in all casses on both bridges the negative respectively positive coefficient of correlation (for linear as well for logarithmic and exponential dependance) is very high and its values are very close to 1.0. The results have confirmed that with an increasing traffic load the dynamic coefficient decreases but the dynamic increment increaes.

4. CONCLUSION

In conclusion we can say that for further theoretical investigation of durability as well as a reliability analysis of a bridge construction the *Weibull's* or *the Gumbel's* theoretical model of probability distribution of the deflection could be considered as a response of the bridges to the vehicular loads. Further we can say that we have obtained average values of some dynamic characteristics of that type of the bridge from ample statistical data.

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Remaining Structural Capacity of Brick-built Piers and Abutments

Capacité portante résiduelle des piles et des culées en maçonnerie

Resttragfähigkeit von Pfeilern und Widerlagern aus dauerhaftem Mauerwerk

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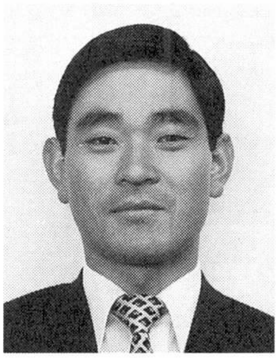

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SUMMARY
The strength of brick piers and abutments of bridges depends on the strength of joints rather than the strength of the bricks themselves. Attention was focused on elastic waves, particularly impulsive waves, as the strength of joints can probably be estimated by measuring the velocity of elastic waves propagating through brick structures. As a result of experiments, a relationship between the velocity of elastic waves propagating through brick structures and the flexural-tensile strength of joints has been derived, allowing a method for evaluating the safety of brick structures under substantial lateral seismic loads to be established. Finally the residual strength of several bridges under the loads of probable major earthquakes was evaluated.

RÉSUMÉ
La résistance des piles et des culées de ponts construites en maçonnerie dépend davantage de celle des joints que de celle des pierres elles-mêmes. Les vérifications se sont concentrées sur les mesures de la vitesse de propagation des ondes élastiques sous excitation par impulsions, en vue de déterminer l'état des joints. A partir d'échantillons de maçonnerie prélevés sur les piles de pont, il a été possible d'établir une corrélation entre la vitesse de propagation des ondes et la résistance à la flexion et à la traction des joints. Une méthode d'évaluation de la sécurité de la maçonnerie sous efforts transversaux dus aux tremblements de terre a pu être mise au point et appliquée à de nombreux ponts.

ZUSAMMENFASSUNG
Die Festigkeit von Mauerwerkspfeilern und -widerlagern hängt eher von der Festigkeit der Fugen als der Steine selbst ab. Die Untersuchungen konzentrierten sich deshalb auf die Messungen der Fortpflanzungsgeschwindigkeit elastischer Wellen unter Impulsanregung, um so den Zustand der Fugen zu ermitteln. Anhand von Mauerwerksproben aus Brückenpfeilern konnte eine Beziehung zwischen der Wellengeschwindigkeit und der Biegezugfestigkeit der Fugen hergeleitet werden. Damit konnte eine Methode zur Beurteilung der Sicherheit von Mauerwerk unter hoher Erdbebenquerbelastung entwickelt und auf mehrere Brücken angewendet werden.

1. INTRODUCTION

The number of piers and abutments on the JR lines has reached 132,000 (conventional lines), of which about 110,000 (83%) are made of brick or stone masonry units, or plain concrete [1]. The number of brick or stone masonry piersand abutments reaches about 32,000 (24%).

About 70% of these structures were built 40 or more years ago. Surprisingly, 90% of brick or stone masonry structures are 60 or more years old (see Fig. 1).

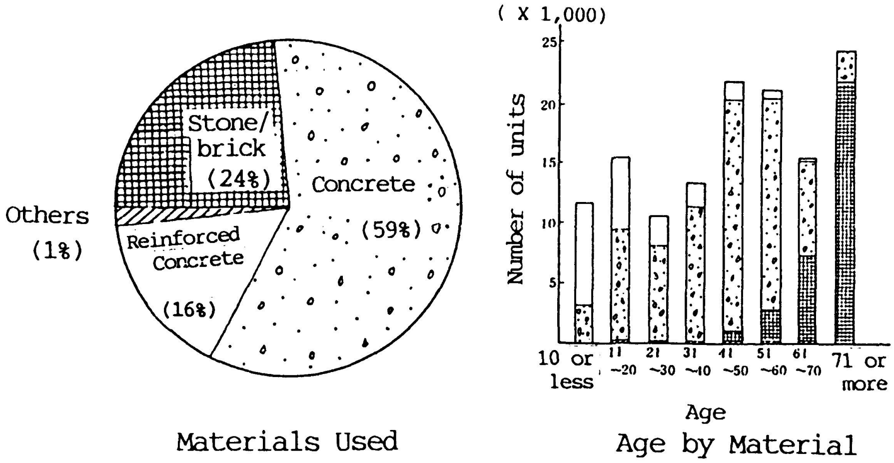


Fig. 1 State of Substructure(132,000 units)
(from Journal of Structurel Design Journal,No.9,February 1983)

Some of these old structures have already deteriorated because of years of use. Although they are being replaced with reinforced concrete structures, it takes huge amounts of cost and time to replace all of them. It is inevitable, therefore, to use the old structures until they are replaced.
Japan is one of the most seismically active countries in the world and has suffered many large earthquakes, which have resulted in substantial structural damages.

Recent damage-causing earthquakes include the Nankai Earthquake (M8.1, 1946), Fukui Earthquake (M7.3, 1948), Tokachi-oki Earthquake (M8.1, 1952), Boso-oki Earthquake (M7.5, 1953), Niigata Earthquake (M7.5, 1964), Tokachi-oki Earthquake (M7.9, 1968), Miyagiken-oki Earthquake (M7.4, 1978), Nihonkai Chubu Earthquake (M7.7, 1983), and Chibaken Toho-oki Earthquake (M6.2, 1987) [2].

Joints in brick piers and abutments are often in the deteriorated condition due to years of use. These deteriorated joints are vulnerable particularly to lateral loads like earthquake loads.

It is therefore important to evaluate the soundness, particularly earthquake resistance, of those old structures and take whatever necessary steps accordingly.

In this paper, data obtained from model experiments, which can be used in evaluating the earthquake resistance of brick piers and abutments, is reviewed. It is believed that the data will be found useful in maintaining those old structures.

2. FACT-FINDING SURVEY

2.1 Strengths of Brick Structures [3]

Block samples were taken from an old pier(built in 1889), and the shear strength of the samples was measured. Photo 1 shows a similar brick pier.

2.1.1 Shear Strength

Specimens used in the loading test are cubes measuring about 80cm x 80cm x 80cm, and their surfaces have been chiseled. The loading test was conducted in the form of a double shear test, as shown in Fig. 2. To introduce a stress of 0.19 MPa, which corresponds to the dead load (6374 kN), bearing plates (to cover overall surfaces) were installed on both sides of the specimens, and six ϕ 16mm steel bars were used for tensioning. Shear failures occurred at two joint faces, but the failures at the two faces did not occur simultaneously. Thus the first shear failure and the next one are referred to as the first failure and the second failure, respectively. The results of the shear strength test are shown in Table 1.

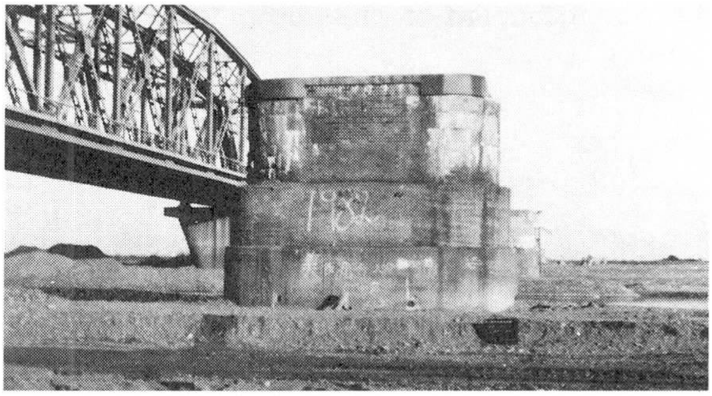


Photo 1 An Example of Brick Pier

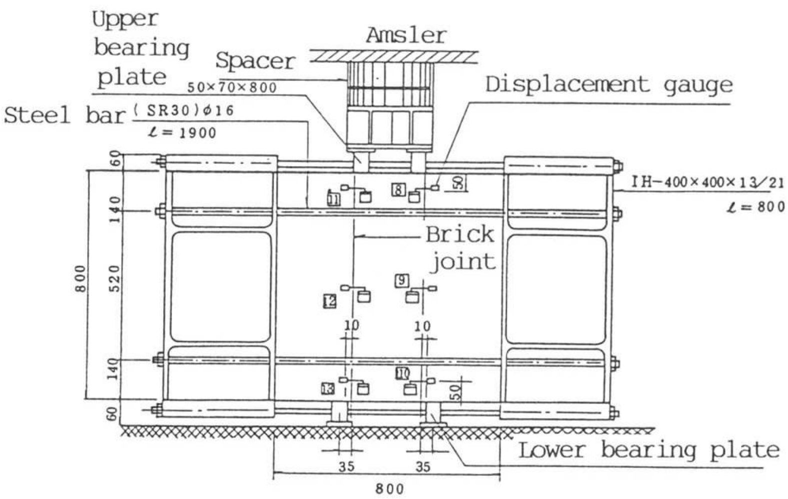


Fig. 2 Shear Test (unit: mm)

	Specimen 1		Specimen 2	
Stage of failure	First failure	Second failure	First failure	Second failure
Load 2P	1167 kN	1238 kN	1348 kN	1079 kN
Shear Stress	0.88 Mpa	0.93 Mpa	0.99 Mpa	0.86 Mpa

Table 1 Results of Shear Strength Test

2.2 Nondestructive Test Using Elastic Waves Propagating in Brick Structures

2.2.1 Configuration of the Setup for the Nondestructive Test

The strength of brick piers and abutments depends on the strength of joints rather than the strength of bricks, and there has been no method to estimate the strength of structures consisting of bricks.

As a tool for determining the compressive strength of concrete nondestructively, ultrasonic waves are in general use. The currently used ultrasonic wave methods, however, are hardly applicable to such partly hollow, mortar-jointed structures as brick structures because of considerable attenuation of sound waves. Therefore, a method using elastic waves (which in this paper refer to waves caused by hammering, though they generally refer to waves propagating through elastic bodies) with high levels of propagated energy caused by hammering was employed [4].

The configuration of the setup for elastic wave measurement is illustrated in Fig.3.

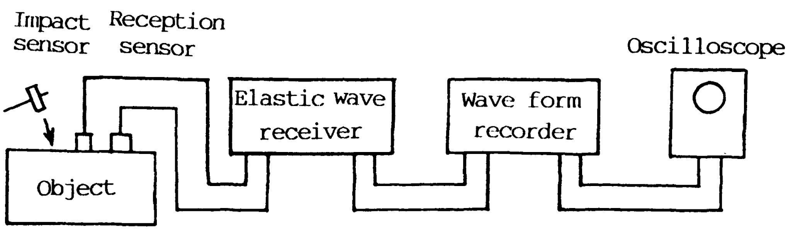


Fig. 3 Standard Configuration of Elastic Wave Measurement System [4]

Two sensors were used for the detection of elastic waves: an impact sensor and a reception sensor. The impact sensor detects surface vibration and converts it into signals representing the occurrence of waves. The reception sensor detects waves propagated through an elastic body. In the test, the signal for the initiation of waves and an elastic wave signal were chosen at the elastic wave receiver and stored in the wave form recorder. The recorded wave forms were then displayed on the oscilloscope.

2.2.2 Method for Measurement of Elastic Waves

The concepts of two standard methods for measuring elastic waves are shown in Fig. 4. Usually the transmission method is used to measure the velocity of elastic waves transmitted through media, and the reflection method is used to determine the locations and sizes of discontinuities in horizontal joints. Often these methods are used in combination.

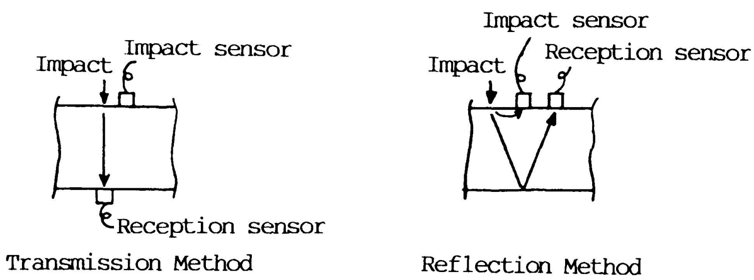


Fig. 4 Standard Measuring Method [4]

2.3 Nondestructive and Destructive Tests on Brick Specimens [5]

2.3.1 Relationship between Elastic Wave Velocity and Flexural-Tensile Strength

As shown in Fig. 5, specimens were taken from a pier of a bridge on a railway line that went out of operation, and were dressed manually. Using these specimens, a nondestructive test as shown in Fig. 6 and a flexural failure test as shown in Fig. 7 were conducted.

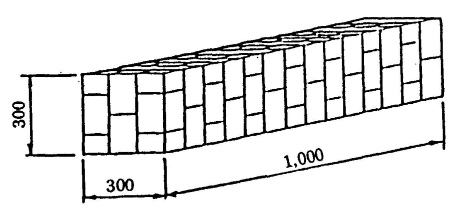
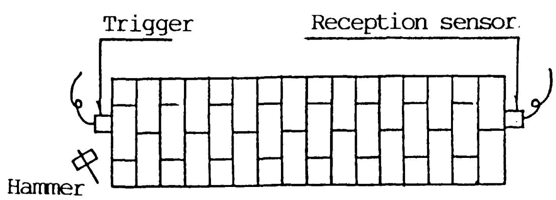


Fig. 5 Dimensions of Brick Specimen (unit: mm)



In measuring reflected waves, the trigger and the reception sensor are positioned at the corresponding points on the opposite faces.

Fig. 6 Nondestructive Test

Fig. 8 is a plot of the results of the tests, with the axis of ordinates measuring the flexural-tensile strength and the axis of abscissas the elastic wave velocity (where the dotted line represents a multiple regression formula). As a practical formula for the relationship between the elastic wave velocity and the flexural-tensile strength based on engineering judgments, the following equation is proposed:

$$\sigma_{ctu} = 2.65 \times 10^{-4}\ V \qquad (1)$$

where

σ_{ctu} :flexural-tensile strength(MPa)
V :velocity of elastic wave(m/sec)

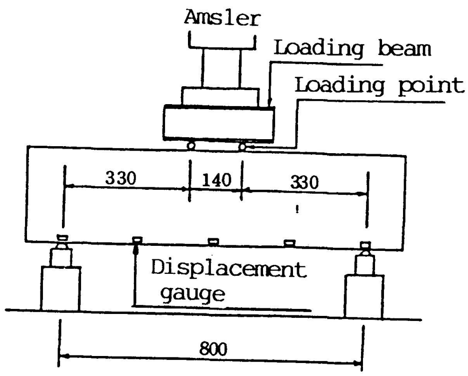


Fig. 7 Loading Device and Loading Points (unit: mm)

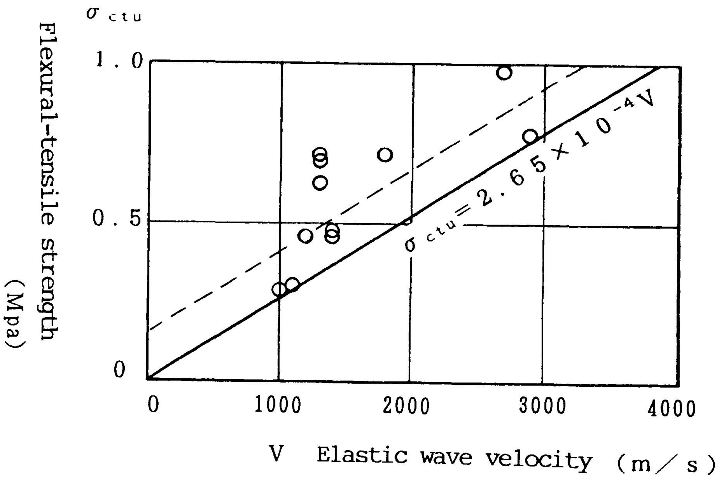


Fig. 8 Relationship between Elastic Wave Velocity and Flexural-Tensile Strength

2.3.2 Estimation of Residual Strength and the Locations of Defective Regions [4]

When the velocity of elastic waves in an actual pier or abutment is to be measured, transmission velocities are measured at multiple points as shown in Fig. 9 (a), and the average value of measurements is taken as the transmission velocity for the structure. Then, the flexural-tensile strength σ_{ctu} is calculated using Eq. (1).

The locations of defective regions can be determined by use of a time-distance curve as shown in Fig. 9(b). A time-distance curve shows the time elastic waves require to travel a certain distance. Since the gradient of the time-distance curve represents velocity, a greater gradient of the curve means a lower velocity. The lower velocity shown by the steep gradient in the section b-d in Fig. 9(b) suggests the possibility of disconnected joints.

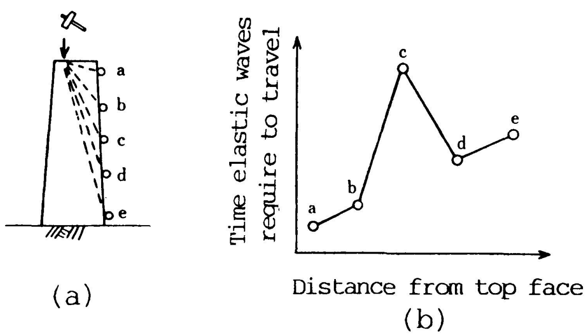


Fig. 9 Time-Distance Curve

2.3.3 Estimation of the Embedment Depth of Existing Structures

It is not unusual that design drawings of aged structures such as brick piers and abutments have been lost during the wars or for some other reasons. In such cases it is necessary to determine the embedment depth in order to evaluate the soundness of particular structures.

The embedment depth of a particular structure can be determined by estimating the velocity of elastic waves transmitted through the structure and finding bottom reflection from reflected waves [4]. Photo 2 and Photo 3 show examples of transmitted wave forms and reflected wave forms, respectively.

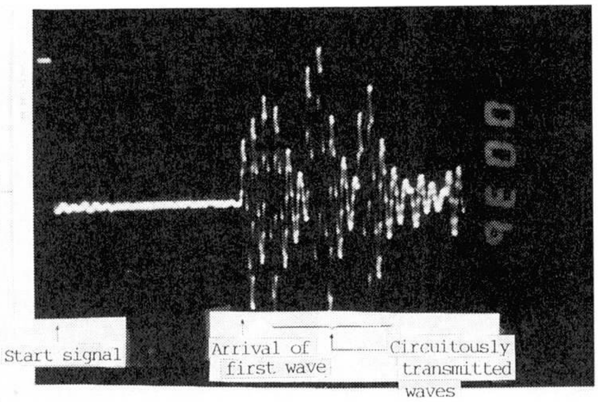


Photo 2 Transmitted Wave Forms

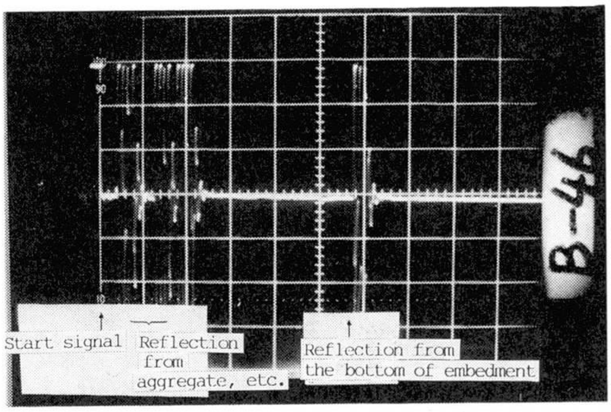


Photo 3 Reflected Wave Forms

3. JUDGMENT OF SOUNDNESS (MAINLY EARTHQUAKE RESISTANCE)

3.1 Failure Due to Lateral Seismic Loads or Other Lateral Loads

From data on brick piers damaged by earthquakes and the results of loading tests on abandoned brick piers, the failure process has been estimated as follows:

First, disconnection between joint mortar and bricks occurs where bond is weakest, resulting in decreases in the area of joint face (bonding surface). Thus, shear stresses acting on the joint face increases rapidly, causing slip along the horizontal plane. This failure process is illustrated in Fig. 10 [5].

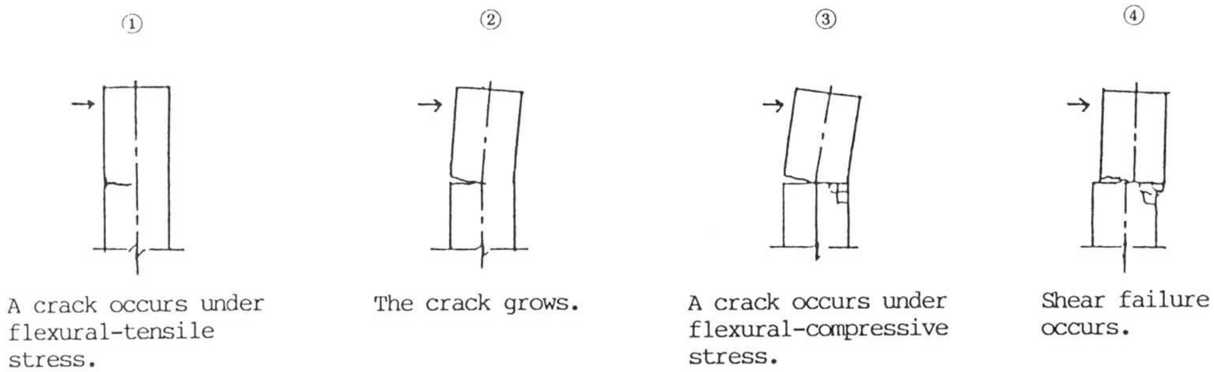


Fig.10 Failure Process [5]

3.2 Earthquake Resistance

If the flexural-tensile strength of a joint can be calculated using Eq. (1), earthquake resistance, that is, whether the joint is disconnected or not can be judged from such factors as bedrock acceleration, the type of ground, the natural period of a particular structure, and the acceleration magnification factor.

4. AN EXAMPLE EVALUATING THE EARTHQUAKE RESISTANCE OF AN ACTUAL PIER

4.1 Pier

The earthquake resistance of several piers has been evaluated. In this section, a pier (8P) of the former Fuji River Bridge on the Tokaido Line is considered.

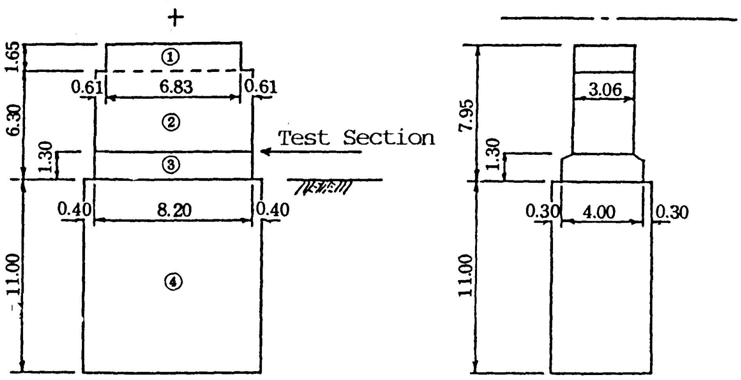


Fig.11 Fuji River Bridge on the Tokaido Line (out-of-use brick pier, 8P; unit:m) Superstructure:Theodore truss W=1433KN

4.2 Results of Earthquake Resistance Evaluation

Tables 2 and 3 show the results of the earthquake resistance evaluation. The shear strength used here is as per ACI's shear friction theory, and μ =1.0 is assumed.

Bridge name	Ground surface acceleration α(gal)	Natural period (sec)	Response acceleration at structure's center of gravity $\beta\alpha$(gal)	Response acceleration at center of gravity in test section $\beta'\alpha$(gal)	Equivalent seismic intensity $K=\beta'\alpha/g$	Average elastic wave velocity (m/s)
Former Fuji River bridge	221	0.3	508	566	0.578	2500

Table 2 Evaluation Data

Bridge name	Bending stress (Mpa)			Shear stress (Mpa)		
	Occurrence	Tolerance	Judgment	Occurrence	Tolerance	Judgment
Former Fuji River bridge	−0.69	−0.66	×	0.10	0.18	○

Table 3 Calculated Stresses

5. CONCLUSION

Findings from this study can be summarized as follows:

(1) A method for estimating the flexural-tensile strength of joints in brick structures like piers has been developed on the basis of the results of a nondestructive test using impulsive elastic waves.

(2) The embedment depths of existing structures can be estimated by use of the transmission method and the reflection method using impulsive elastic waves.

(3) The earthquake resistance of brick structures like bridge piers can be evaluated by the combined use of (1) and (2) above.

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