Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 57/1/57/2 (1989)

Rubrik: Presentation Session 4b: Assessment of structures

Expert System for Serviceability Rating of Concrete Bridges

Système expert pour la détermination de l'aptitude au service de ponts en béton

Expertensystem für die Unterhaltungsbeurteilung von Betonbrücken

Ayaho MIYAMOTO
Assoc. Professor
Kobe Univ.
Kobe, Japan

Ayaho Miyamoto, born 1949, received his Doctor of Engineering Degree from Kyoto Univ. in 1985. His recent research activities are in the areas of serviceability rating on concrete bridge and nonlinear dynamic (impact) analysis of reinforced concrete slab. He is a member of IABSE, JSCE, ACI and JCI.

Yuji YAMAGUCHI
Civil Engineer
Japan Highway Public Corp.
Gunma, Japan

Yuji Yamaguchi, born 1964, received his Master of Engineering Degree from Kobe Univ. in 1988. He is attached to the Takasaki Construction Office and has been engaged in investigation of systematization of bridge rating techniques.

Tuyoshi MAEDA
Chief Engineer
Hyogo Pref. Gov.
Kobe, Japan

Tuyoshi Maeda, born 1950, received his Master of Engineering Degree from Kobe Univ. in 1975. He is attached to the Road Construction Division and is now involved in technical investigation on repair, rehabilitation and replacement techniques of existing bridges.

Moriyoshi KUSHIDA
Chief Engineer
Sumiyoshi Factory of Kurimoto
Osaka, Japan

Moriyoshi Kushida, born 1954, received his Master of Engineering Degree from Kobe Univ. in 1979. He is attached to be the Structural Eng. Division which is now working on long span bridges, and also has great interest in assessment procedures for existing bridges.

SUMMARY

This paper describes a knowledge-based expert system for serviceability rating of concrete bridges. The present system applies the concepts of basic probability according to Dempster & Shafer's theory to deal with the subjective information related to bridge rating. The final results produced by this system are considered to be represented by five elements expressed by linguistic expressions with the fuzziness value which is the degree of subjective uncertainty.

RÉSUMÉ

Cet article décrit un système expert, de type base de données, pour la détermination de l'aptitude au service de ponts en béton. Le présent système applique les concepts des probabilités de base selon la Théorie de Dempster et Shafer pour tenir compte des informations subjectives relatives à l'évaluation du pont. Les resultats finaux obtenus avec ce système sont considérés comme étant définis pour cinq éléments exprimés par des expressions descriptives avec une valeur de divergence qui est le degré d'incertitude subjective.

ZUSAMMENFASSUNG

Dieser Beitrag beschreibt ein Expertensystem für die Unterhaltungsbeurteilung von Betonbrücken. Das vorliegende System verwendet die Konzepte der grundlegenden Wahrscheinlichkeit nach der Theorie von Dempster & Shafer zur Handhabung der mit der Brückenbewertung zusammenhängenden Information. Für die durch dieses System erhaltenen Endergebnisse wird angenommen, dass sie mit fünf Elementen charakterisiert werden, die durch sprachliche Begriffe, zusammen mit dem Verschwommenheitswert, der Grad der subjektiven Ungewissheit ist, ausgedrückt werden können.

1. INTRODUCTION

In this paper, an expert system for serviceability rating of concrete bridges (Bridge Rating Expert System) is developed based on a combination of several components which are the knowledge base including the subjective information related to the rating, the inference engine, the data reference module, the calculation module, the explanation module, the knowledge acquisition module and the I/O module. The computer system and main language which is used in the expert system are the PC-9801VX41 personal computer made by NEC Corporation, Japan and PROLOG and C languages, respectively. For the construction of the knowledge base including the subjective information related to the rating, it is an unavoidable problem in dealing with subjective informations which cannot be allotted binary codes such as true or false. As a remedy to this problem, a concept of the basic probability according to the Dempster & Shafer's theory is introduced in the present system. The upper probabilities in the Dempster & Shafer's theory to introduce experiences and knowledge accumulated into the knowledge base are obtained through questionnaires sent out to bridge experts. The results of the rating at the final stage produced by this system are considered to be represented by five elements expressed by the linguistic expressions "safe" "slightly safe" "moderate" "slightly danger" "danger" with the fuzziness value which is the degree of subjective uncertainty. A few concrete bridges on which field data have been collected are analyzed to demonstrate the applicability of this expert system. Through the application to the deteriorated reinforced concrete bridge girders and slabs, reasonable results are obtained by inference with the expert system.

2. SYSTEM DESCRIPTION

The Bridge Rating Expert System is a newly developed microcomputer knowledge based system which is capable of various inference and judgment. The expert system consists of seven main components: the knowledge base system, the inference engine, the data reference module, the calculation module, the explanation module, the knowledge acquisition module and the I/O module.

To develop a practical knowledge-based expert system for serviceability rating of concrete bridges,it is necessary not only to establish a diagnostic process model that can capture most of the available information about bridge rating but also have a rule in dealing with subjective information of bridge engineers such as professional experience, knowledge on bridge rating, etc. In order to construct a diagnostic process model in the knowledge processor of the inference engine, the relations among causes of deterioration of structural serviceability (judgment factors) are represented by a global hierarchical form which has serviceability for slabs and main girders, respectively as the final goal. The hierarchy structure consists of 11 sub goals, 23 goals and 34 basic factors for slabs and 10 sub goals, 17 goals and 30 basic factors for main girders. On the other hand, in order to develop a rule in dealing with subjective information of bridge engineers, a concept of the basic probability according to the Dempster & Shafer's theory is introduced in the knowledge base of the Bridge Rating Expert System. The upper probabilities in the Dempster & Shafer's theory[1] to introduce experiences and knowledge accumulated into the knowledge base are obtained through questionnaires consisting more than 400 questions concerning both slab and girder sent out to bridge experts[2]. The knowledge base consists of general facts, a set of production rules for storing the empirical knowledge and a series of knowledge fields[3]. In determining the value of the above-mentioned basic probabilities, $m(\{x\})$,it is deemed effective to base on opinions extracted from questionnaires sent out to bridge rating experts as the bridge engineer's knowledge is considered to be transferred to the knowledge base of the expert system. Considering the case when a group of bridge experts make a diagnosis on a structure, the scattering of individual diagnosis may be regarded as the fuzziness of diagnosis by the group, which may be measured quantitatively by the standard deviation in the case of numerical estimation of the specified factor of a target structure. The questionnaire consists of a series of more than 400 questions which corresponded to the hierarchy structure of rating process for both slab and main girder. The results of bridge rating are considered to be represented by five elements expressed by the linguistic expressions "safe", "slightly safe", "moderate", "slightly danger" and "danger", each of which is symbolized by a,b,c, d and e. The 15 kinds of basic probabilities can be obtained by solving the equations which were formed based on the properties of basic probability. In the rating process of structural serviceability conformed to the hierarchy structure, the combination of some basic probabilities retrieved from the series of knowledge fields are performed in each level of goal and sub goal by use of the Dempster's rule of combination[1]. And, the rating at the final stage will be performed by selecting the element a_i which corresponds to the maximum estimated value $M(a_i)$ given by the following equation and then the judgment is given on the screen display of the system:

$$M(a_i)=\sum_{a_i \in A_k} \frac{m(A_k)}{N(A_k)} \quad (i=1,2,\cdots n) \tag{1}$$

where, $m(A_k)$ is the basic probability for the set A_k and $N(A_k)$ is the number of elements in a set A_k.
Furthermore, since it may be considered that the degree of fuzziness is larger when a large mass of basic probability is able to move in a wider range, the fuzziness, F, of the assessment will be given by the following equation:

$$\begin{aligned} F&=\sum_{A_k} m(A_k)\cdot s(A_k)=\sum_{A_k} m(A_k)\cdot[\{N(A_k)-1\}\cdot dx] \\ &=\sum_{A_k} m(A_k)\cdot[\{N(A_k)-1\}/(n-1)] \end{aligned} \tag{2}$$

where, $s(A_k)$ is the allotted movable distance for the basic probability of a set A_k and $dx=1/(n-1)$ is the distance between adjacent elements on the abscissa.

Both forward and backward reasoning are used as the inference engine in the present expert system. The flow of reasoning in the inference engine of the expert system is shown in Fig.1[3]. The inference is performed separately on the slab and the main girder of a target bridge aiming at the diagnosis of the serviceability as a final goal along the flow of Fig.1. Therefore, two kinds of knowledge-base system are prepared for slabs and main girders, and are read immediately before diagnosis starts. In the flow of inferences shown in Fig.1, the forward reasoning process will continue until the arrival at the data item(basic factor) stage, for which the advanced inferences are difficult to perform. For example, an answer of "yes" or "no" for the deposition of free lime in reinforced concrete bridges halts any further inference. For such items(basic factors), suitable basic probabilities are assigned as an opinion from a series of knowledge fields and are joined

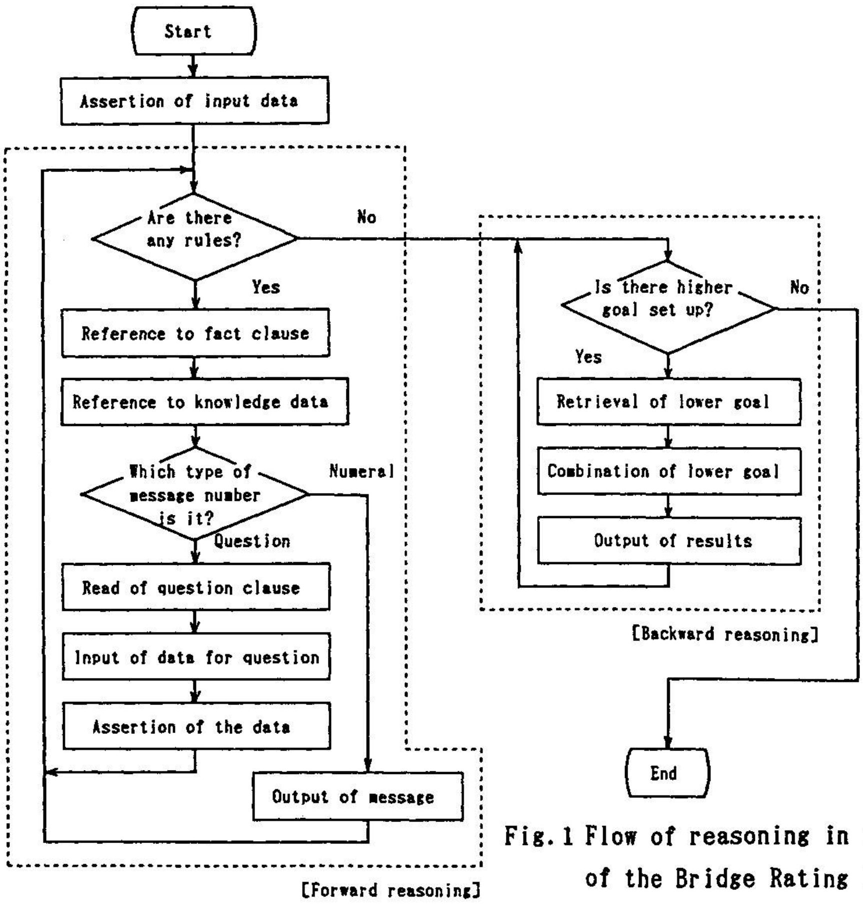


Fig.1 Flow of reasoning in inference engine of the Bridge Rating Expert System

together at each goal. When all data reaches this state, forward reasoning will be followed by backward reasoning. The basic probability is given in a set of production rules for storing the empirical knowledge according to the results of questionnaires or to the subjective judgment on them. During backward reasoning, the lower sub goal, which is necessary for inference of the higher sub goals pre-set previously, is retrieved, and the assigned basic probabilities are calculated and combined, and next asserted as a new fact clause. At the same time, using the new basic probabilities obtained from the higher sub goal, the estimated values for "safe", "slightly safe", "moderate", "slightly danger" and "danger" with the fuzziness value which is the degree of subjective uncertainty are calculated and picked out as outputs. Finally, the serviceability of a target bridge, which is set as a final goal, is diagnosed basing on the combination of the two highest sub goals, namely the "durability" and the "load carrying capability", and is picked out.

3.APPLICATION OF EXPERT SYSTEM TO ACTUAL BRIDGE RATING

The Bridge Rating Expert System is verified for its effectiveness through the field testing on three kinds of reinforced concrete T-beam bridges[4]. Three national highway bridges, Sakurabashi(constructed in 1933), Maenobashi(constructed in 1931) and Taitabashi(constructed in 1950), were selected for verification of the inference results because these bridges were about 40 and over 50 years old which is considered to be the design service life for concrete bridges.

The Bridge Rating Expert System is used to diagnose the three bridges described above. Table 1 shows an example of a dialog between the expert system and a user extracted from the intermediate stage of the diagnosis of reinforced concrete T-beams(main girders) in Taitabashi bridge. The first question produced by the expert system side to the user concerns the present state of cracks caused in main girders. In the case of Taitabashi bridge, the answer is chosen as "flexural crack", "corrosion crack", "bond crack" according to the observed eminent crack modes in the bridge. Generally speaking, the so-called menu format was adopted where the user selects an answer from prepared multiple-choice suggestions. The

Table 1 An example of dialog between the Bridge Rating Expert System and user (for main girder of Taitabashi bridge)

Question and explanation from the Bridge Rating Expert System	Answer from user
What kind of cracks are there in main girders?	Frexural crack Corrosion crack Bond crack
[C: Vertical cracks are inferred as caused by bending moment]	
What level is the bending cracks?	3rd stage; a few cracks
What is the maximum crack width?	1.0 mm
[C: Cracks over 0.3mm wide are recommended to be repaired]	
Are there any water leakage & free lime near the cracks?	Occurred considerably
Are there any spalling of cover concrete near the cracks?	Occurred slightly
What degree of reinforcement corrosion is there near the cracks?	Severely corroded
What level is the corrosion cracks?	3rd stage; a few cracks
[C: Horizontal cracks parallel to longitudinal direction are inferred as caused by volume expansion of steel corrosion]	
What is the maximum crack width?	0.5 mm
[C: Cracks over 0.3mm width are recommended to be repaired]	
Are there any water leakage & free lime near the cracks?	Occurred considerably
Are there any spalling of cover concrete near the cracks?	Occurred moderately
What degree of reinforcement corrosion is there near the cracks?	No exposure of steel
Are there any rust deposition?	Nothing
What level is the bond cracks?	3rd stage; a few cracks
[C: Small diagonal cracks along reinforcement sometimes occur when steel ratio is relatively large and round bars are used]	
What is the maximum crack width?	0.5 mm
[C: Cracks over 0.3mm width are recommended to be repaired]	
Are there any water leakage & free lime near the cracks?	Occurred considerably
Are there any spalling of cover concrete near the cracks?	Occurred moderately
What degree of reinforcement corrosion is there near the cracks?	No exposure of reinforcing bars
Are there any rust deposition?	Nothing

Table 2(a) Inference results for Sakurabashi bridge

	Judgement factor	safe	slightly safe	moderate	slightly danger	danger	fuzziness
Main girder	Design	0.132	0.313	0.437	0.115	0.003	0.466
	Execution of work	0.049	0.445	0.478	0.028	0.000	0.245
	Service condition	0.345	0.549	0.105	0.002	0.000	0.159
	Flexural crack	0.000	0.000	0.030	0.890	0.081	0.008
	Shear crack	0.000	0.000	0.000	0.081	0.919	0.002
	Corrosion crack	0.000	0.000	0.008	0.748	0.244	0.034
	Whole damage	0.000	0.000	0.000	0.929	0.071	0.000
	Load carrying capa.	0.000	0.000	0.000	1.000	0.000	0.000
	Durability	0.000	0.000	0.000	1.000	0.000	0.000
	Serviceability	0.000	0.000	0.000	1.000	0.000	0.000

Table 2(b) Inference results for Maenobashi bridge

	Judgement factor	safe	slightly safe	moderate	slightly danger	danger	fuzziness
Slab	Design	0.032	0.395	0.523	0.049	0.000	0.113
	Execution of work	0.248	0.248	0.248	0.248	0.008	0.760
	Road condition	0.993	0.007	0.000	0.000	0.000	0.003
	Service condition	0.985	0.015	0.000	0.000	0.000	0.003
	The worst slab	0.026	0.459	0.486	0.029	0.000	0.019
	Crack along haunch	0.277	0.581	0.131	0.011	0.000	0.285
	Crack at slab center	0.056	0.319	0.458	0.167	0.000	0.221
	Whole damage	0.007	0.634	0.357	0.001	0.000	0.006
	Load carrying capa.	0.000	0.442	0.558	0.000	0.000	0.001
	Durability	0.808	0.192	0.000	0.000	0.000	0.001
	Serviceability	0.001	0.999	0.000	0.000	0.000	0.000
Main girder	Design	0.132	0.313	0.437	0.115	0.003	0.466
	Execution of work	0.248	0.248	0.248	0.248	0.008	0.760
	Service condition	0.626	0.357	0.018	0.000	0.000	0.196
	Flexural crack	0.138	0.683	0.176	0.003	0.000	0.084
	Corrosion crack	0.001	0.093	0.599	0.306	0.000	0.000
	Whole damage	0.002	0.397	0.594	0.007	0.000	0.022
	Load carrying capa.	0.001	0.675	0.324	0.000	0.000	0.007
	Durability	0.001	0.789	0.210	0.000	0.000	0.003
	Serviceability	0.000	0.000	0.883	0.117	0.000	0.000

Table 2(c) Inference results for Taitabashi bridge

	Judgement factor	safe	slightly safe	moderate	slightly danger	danger	fuzziness
Slab	Design	0.007	0.317	0.605	0.071	0.001	0.068
	Execution of work	0.407	0.495	0.092	0.006	0.000	0.241
	Road condition	0.058	0.199	0.421	0.321	0.001	0.448
	Service condition	0.865	0.134	0.002	0.000	0.000	0.015
	The worst slab	0.000	0.000	0.001	0.515	0.484	0.003
	Crack along haunch	0.002	0.123	0.815	0.060	0.000	0.076
	Crack near support	0.000	0.007	0.173	0.794	0.026	0.068
	Crack at slab center	0.000	0.000	0.001	0.528	0.471	0.004
	Whole damage of slab	0.000	0.000	0.000	1.000	0.000	0.000
	Load carrying capa.	0.000	0.000	0.006	0.994	0.000	0.000
	Durability	0.000	0.000	1.000	0.000	0.000	0.000
	Serviceability	0.000	0.000	1.000	0.000	0.000	0.000
Main girder	Design	0.264	0.479	0.196	0.060	0.002	0.421
	Execution of work	0.049	0.445	0.478	0.028	0.000	0.245
	Service condition	0.511	0.455	0.034	0.000	0.000	0.178
	Flexural crack	0.000	0.000	0.000	0.009	0.991	0.001
	Corrosion crack	0.000	0.000	0.007	0.832	0.161	0.006
	Bond crack	0.000	0.000	0.078	0.915	0.007	0.020
	Whole damage	0.000	0.000	0.000	0.959	0.041	0.000
	Load carrying capa.	0.000	0.000	0.000	1.000	0.000	0.000
	Durability	0.000	0.000	0.000	1.000	0.000	0.000
	Serviceability	0.000	0.000	0.000	1.000	0.000	0.000

following question is on the flexural cracks on which the observation from the most severely cracked girder was chosen as input. The feature of the cracks pointed out in this case are generally unidirectionally spread out, which leads to the answer "3rd stage" out of a choice of 8 stages presented in a menu format. For the input of a maximum crack width of "1.0mm", which surpasses well above the allowable limit, the system recommends that the cracks be repaired. In the following step, the target of questions is directed to the "condition of cracks along the flexural crack", and answers concerning the severe deterioration around the bottom and both side surfaces are required: "Are there any water leak and free lime deposited? " or "Are there any spalling of cover concrete?". The answers for these are "considerably occurred" and "slightly occurred", respectively. Based on the answer for level of spalling, a further question is produced by the expert system: "What degree of reinforcement corrosion is there". By answering "severely corroded",the questions on the flexural cracks comes to an end. In the next steps, the target of questions is moved forward from "corrosion crack" to " bond crack", and the answers are requested to be prepared on the same manner as that of flexural crack. When all questions are filled up the data(basic factors), and the assigned basic probabilities are combined, the inference results with the inferred causes at the final goal and each sub goal are listed on the screen display through the forward and backward reasoning as shown in Table 2(a)-(c). From these tables, the "slab serviceability" as the final goal inferred from the "load carrying capability" and the "durability" is estimated to be support of the element of "slightly safe" for Maenobashi bridge and "moderate" for Taitabashi bridge. On the other hand, the "girder serviceability" is estimated to be support of the element of "slightly danger" for Sakurabashi bridge, "moderate" for Maenobashi bridge and "slightly danger" for Taitabashi bridge. To illustrate further, we investigate and analyze the estimated values at the sub goals(judgment factors) where the items related to the deterioration of serviceability along the rating process for main girder are as follow: The estimated results for the "flexural crack", "shear crack" and "corrosion crack" in Sakurabashi bridge are support of the element of "slightly danger" and "danger". Then, such estimation affects those for the "whole damage of main girder(element value =0.93)", and the "load carrying capability" and the "durability", which are the highest sub goals and the "girder serviceability" which is the final goal are estimated to be support of the element of "slightly danger(element value =1.0)" without "fuzziness" (see Table 2(a)). On the contrary, for Maenobashi bridge, the estimated results for all judgment factors except for "service condition" have a tendency to support the element of "slightly safe" and "moderate". Then, the "load carrying capability" and the "durability" are estimated to be support of the element of "slightly safe"(see Table 2(b)). Finally, for Taitabashi bridge, the judgment factors except for "design", "execution of work" and "service condition" are estimated to be support of the element of "slightly danger" and "danger". Because such estimation affects those for the abovementioned three factors, both the "load carrying capability" and the "durability" are estimated to be support of the element of "slightly danger (element value =1.0)" without "fuzziness"(see Table 2(c)). These conclusions coincide well with the results obtained through the field testing[4].

4. CONCLUDING REMARKS

By introducing the expert system and constructing the knowledge-base system of experiences and knowledge of experts through questionnaires to them, the systematization of the bridge serviceability diagnosis which is comparatively easy to modify and to renew is shown possible. Through the application to actual bridges, reasonable results were obtained by inference with the system. The certification of the present system will be continued by accumulating data on actual bridges.

REFERENCES

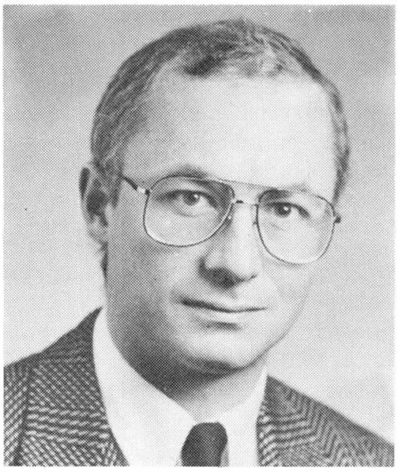
1. ISHIZUKA M., Probability Theory of Dempster and Shafer. Journal of the JSEC, Sept. 1983.
2. MIYAMOTO A., NISHIMURA A., YAMAGUCHI Y. and HONMA I., Utilization of Questionnaire in the Development of Rating System for Concrete Bridges. Proc. of JCI, 10-3, June 1988.
3. MIYAMOTO A., Diagnosis Techniques for Concrete Structures. Concrete Journal, 26-7, July 1988.
4. MIYAMOTO A. and FUJII M., Safety Evaluation and Verification by Field Testing of Concrete Bridges. Proc. of ASCE Congress '89, May 1989.

A Strategy for Structures Suffering Fatigue Cracking

Procédure d'évaluation des structures sujettes à la fissuration par fatigue

Eine Strategie für ermüdungsgefährdete Tragwerke

Ian F. C. SMITH
Research Associate
Swiss Fed. Inst. of Technology
Lausanne, Switzerland



Dr. Smith received engineering degrees from Cambridge University, U. K. and the University of Waterloo, Canada. Presently at ICOM, he is performing research into structural engineering applications of expert systems, and into various topics of fatigue and fracture in steel construction. He is also Technical Secretary of the ECCS committee for fatigue.

Manfred A. HIRT
Professor
Swiss Fed. Inst. of Technology
Lausanne, Switzerland

A graduate of ETH Zurich, he received his doctorate from Lehigh University, Bethlehem, USA. He has worked with consultants in Zurich and New York City before joining the Swiss Federal Institute of Technology in Lausanne. He is Chairman of the Swiss committee on loads and the ECCS committee for fatigue design of steel structures.

SUMMARY

The strategy presented in this paper provides a rational tool for evaluating critical details in structures sensitive to fatigue cracking. Approximate and qualitative information, incorporating factors concerning safety, loading, strength, and maintenance history are combined using a simple rating system, thereby creating a framework for decision making.

RÉSUMÉ

La procédure présentée dans cette contribution représente un outil rationnel permettant d'évaluer les éléments critiques d'une structure sujette à la fissuration par fatigue. Les données approximatives et qualitatives concernant, entre autres, la sécurité les charges, la résistance et les travaux de maintenance effectués, sont combinées selon un simple système de répartition en classes. Cette banque de données constitue un outil de décision important pour le choix des mesures à prendre.

ZUSAMMENFASSUNG

Die in diesem Beitrag vorgestellte Strategie ist ein Hilfsmittel zur Einschätzung von kritischen Komponenten in ermüdungsgefährdeten Tragwerken. Annähernde und qualitative Auskünfte, betreffend unter anderem die Sicherheit, die Belastung, die Festigkeit und den bisherigen Unterhalt, werden anhand eines einfachen Einteilungsverfahrens kombiniert, um eine Informationsbasis zur Festlegung von Massnahmen zu schaffen.

1. INTRODUCTION

If fatigue cracks are discovered in a complex structure, development of a rational strategy is essential. It is likely that further inspection will reveal more cracks, and that continued use of the structure will result in crack growth at other locations. Replacement of the structure is rarely desirable due to costs and the inconvenience of interrupting service. On the other hand, if nothing is done, a critical situation may occur; such cracking usually results in reduced safety levels and, at best, increased maintenance costs.

Identification of the causes of fatigue cracking and evaluation of the possibilities of solving the problem at its source are priorities. Clearly, elimination of all causes of fatigue cracking is the best solution. However, all causes are rarely identified and, furthermore, those which are may not be easily eliminated.

The most critical locations may need modifications in order to reduce the risks of cracking. Critical locations should also be inspected more carefully using appropriate inspection technology. Unfortunately, much important information related to factors such as dynamic loading and previous load histories may not be available to the investigator. Consequently, detailed quantitative analyses are often not possible. As a result, indentifying critical locations becomes difficult.

This paper presents a strategy already applied successfully to the evaluation of structures in service. The strategy identifies critical locations in the structure using approximate and qualitative information. Thus, decisions can be made regarding modifications and subsequent inspection tasks.

2. CONCEPT OF RATING CRITERIA

This concept involves rating fatigue-critical details according to six criteria. The first task requires indentification of fatigue-critical details on elements which are subjected to fatigue loading. Typically, these elements are located close to the points of introduction of fatigue loading. In some structures, particularly those subjected to deformation-induced stresses, indentification of such elements requires observation of the structure in service as well as estimates of possible strain directions. Fatigue-critical details on these elements are defined as those details which are vulnerable to fatigue cracking if fatigue stresses were sufficiently high. Since an element may contain several areas of stress concentration, the total number of fatigue-critical details is likely to be several times greater than the number of elements.

A list of the criteria used to rate fatigue-critical details is given in Figure 1. The most important criterion is the rating associated with the CONSEQUENCES OF FAILURE. The events following element failure due to cracking are estimated for each fatigue-critical detail. Situations where cracking at a detail could lead to catastrophic collapse of all or parts of the structure are given a rating of one. A rating of two is given to those details which could cause a local failure if cracking occurred. Local failure is defined as any failure which would compromise the use of the structure in service. Those details which, if failed, would cause cracking in adjacent elements are given a rating of three. Lastly, a rating of four is intended for those details on elements in highly redundant structures where removal of the element would not affect the performance of the structure. These ratings could be lowered to account for details sensitive to rapid fracture.

Evaluation of the LOADING criterion requires careful study. Although all factors which contribute to fatigue loading can rarely be determined exactly, there is usually enough information available in order to classify structural loca-

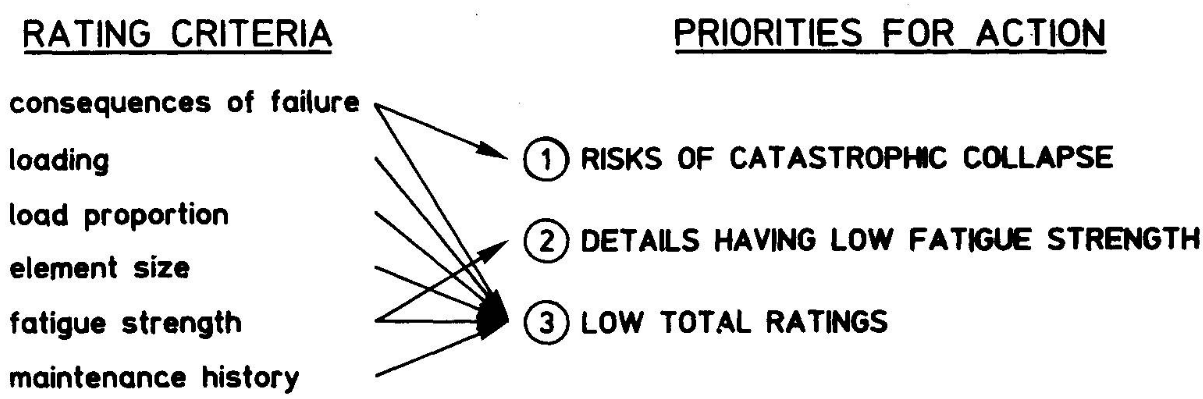


FIGURE 1 : Rating criteria and the use of ratings for setting priorities.

tions in terms of four orders of severity. Load models may provide satisfactory information for quasi-static loading. Correction factors for dynamic effects are often more difficult to estimate, especially if there is noticeable structural movement during fatigue loading. Data from strain gauge or accelerometer measurements, as well as identification of locations where wear or fretting damage is most pronounced, provide important information on dynamic effects. Where total loading effects are estimated to be greatest, details on elements at these locations are given a loading rating of one. Other details are assigned ratings of two, three or four depending upon relative estimates of loading severity.

Estimates of loading severity do not provide sufficient indication of stresses at a given detail. Therefore two additional criteria are employed in order to estimate fatigue stresses. The first criterion is LOAD PROPORTION. If a detail is on an element which carries directly the load, fatigue stresses are likely to be higher than in a detail on an element which is more remote from the point of load application. The second additional criterion for fatigue stresses is ELEMENT SIZE. Other limit states such as deflection requirements, can create situations where elements subjected to similar loading are very different in size. Clearly, larger elements would be subjected to lower fatigue stresses in such situations. As for previous criteria, details are rated from one to four - the most severe cases are given a rating of one and the best a rating of four.

The FATIGUE STRENGTH of a detail determines whether or not fatigue cracks will grow when fatigue loading is applied. Since design recommendations exist for classifying fatigue strength, e.g. [1], a rating using more than four classifications is possible. However, considering the resolution which is possible for other criteria, four classifications are maintained. The best rating of four is reserved for details having good fatigue strengths, for example see Figure 2. The worst rating, one, is reserved for those details which should be avoided in dynamically loaded structures. Intermediate ratings of two and three may be fixed through reference to fatigue-design recommendations.

The final criterion is used to account for the MAINTENANCE HISTORY of the structure. Consultation of maintenance records and interviews with those responsible for structural modifications may reveal valuable information. Again, details on the structure are rated into four groups. Since characteristics of maintenance histories change greatly from one structure to another, it is difficult to provide a general rating framework.

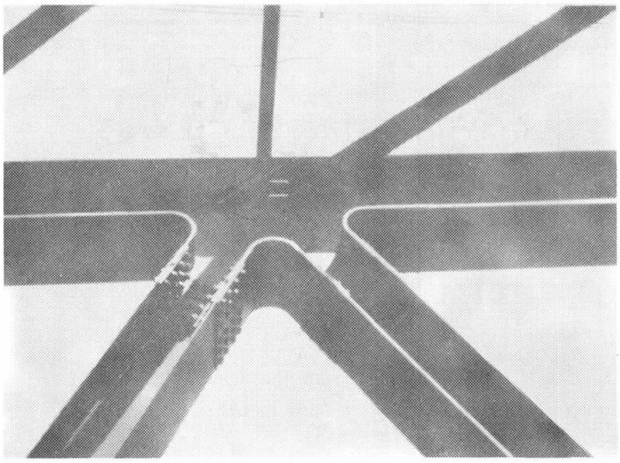


FIGURE 2 : An example of a fatigue-resistant detail.

An example shall be used to illustrate the use of this criterion. A structure, in service for many years and modified several times, was rated according to the following considerations. A rating of one was given to those details where fatigue cracking had been observed and no remedial action had been carried out. A rating of one was also given to locations where remedial action had created a more severe condition, for example, elements containing poorly executed repair welds. A rating of two was assigned to those details where cracking had been observed and remedial action was taken, but it was thought that this action was not going to be successful. A rating of three corresponded to those areas where problems had been rectified satisfactory and no further cracking was expected. Finally a rating of four was given to all those locations where no problems had been observed and none were expected.

The six criteria, consequences of failure, loading, load proportion, element size, fatigue strength and maintenance history, enable six ratings between one and four to be assigned to each fatigue-critical detail. Combination of the ratings requires an estimate of their relative importance. A simple addition of the ratings provides a first estimate of overall severity. If fatigue strength is equally important as fatigue stresses, a combination of the three ratings relating to fatigue stresses into one rating number could be considered. Then, addition is performed to obtain total ratings. Total ratings are used to set priorities, described below, for modification and inspection.

3. PRIORITIES FOR ACTION

Although the total ratings are useful for setting priorities for repair and other modifications, their use, in exclusion of other considerations, is not recommended. An example of priorities is given in Figure 1. Situations where catastrophic collapse is possible are top priorities for action. Secondly, details having very low fatigue strengths should not be present in fatigue loaded structures; modification of all details having a fatigue-strength rating of one is the second priority.

The third priority in Figure 1 employs total ratings. For a given detail, the aim should be to increase its total rating. Ratings for individual criteria provide guidance for each case. For example, the best solution for one detail may involve a reduction of dynamic effects whereas other locations may benefit more by element replacement using more appropriate sections, or by an additional element to share the load. Note, however, that some structural alterations only transfer the problem to other locations. Each case requires individual consideration and not all cases will require modification; a minimum allowable value for total ratings should be fixed whereby details with total ratings above this value do not require action.

If used under appropriate conditions, several improvement methods, such as peening fillet-weld toes and grinding butt-weld reinforcement, are available to increase fatigue strength [2]. Hammer-peening methods, see Figure 3, usually provide the best improvement most economically for details having low fatigue strengths [3]. If the quality of the improvement can be assured, such methods provide useful alternatives to detail modification.

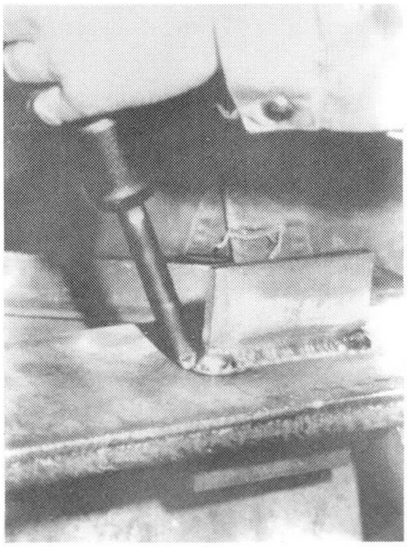


FIGURE 3 : Improvement methods, particularly hammer peening, may increase fatigue strength.

4. USING RATINGS FOR SUBSEQUENT INSPECTION TASKS

The discovery of fatigue cracking in a structure necessitates the development of a rational inspection strategy. Ratings associated with the details should be revised whenever changes are made since they are useful for maintaining appropriate inspection priorities. Repairs should be monitored to ensure that no further cracking will appear. Although ratings are useful guides, they are not likely to identify and place successfully every possible crack location in its correct priority. Therefore, the entire structure should be inspected periodically. A reduction in the inspection effort should be considered only after a decline in the incidence of fatigue cracking has been observed over some time. Finally, an analysis of subsequent crack locations gives an indication of the usefulness of the strategy.

4. FINAL REMARKS

Using this strategy, it is possible to make rational decisions quickly using information which would be incomplete for a detailed structural analysis. Such strategies, if formulated carefully, can increase structural reliability and decrease operating costs. Most structures do not need to be replaced. Unless fatigue cracking is beyond reasonable control, consideration of replacement should be preceded by an attempt to implement a strategy which evaluates the factors discussed in this paper. Finally, this strategy could be extended to cover the evaluation of existing structures, e.g. [4], even when no cracking has been discovered.

ACKNOWLEDGEMENTS

The authors are grateful to several working commissions of national and international standards organizations as well as to many private industries for contributing to the development of ideas. Also, the staff at ICOM are thanked for their help in the preparation of this paper.

REFERENCES

[1] EUROPEAN CONVENTION FOR CONSTRUCTIONAL STEELWORK. Recommendations for the fatigue design of steel structures. ECCS, Brussels, 1985, (Publication no. 43).

[2] SMITH, I.F.C. and HIRT M.A. A review of fatigue strength improvement methods. Canadian Journal of Civil Engineering, Ottawa, vol. 12, no. 1, 1985, pp. 166-183.

[3] BREMEN, U., SMITH, I.F.C. and HIRT, M.A. Comparison of crack-growth behaviour in a welded joint improved by residual-stress methods. International Conference on Fatigue of Welded Constructions, Brighton, UK, 7-9 April, 1987. Cambridge, UK, The Welding Institute.

[4] HIRT, M.A. Remaining fatigue life of bridges. Zurich, IABSE Report, vol. 38, 1982, pp. 113-129.

Bridge Management System for 10,000 Bridges in Thailand

Système de gestion pour 10'000 ponts en Thaïlande

Brückenüberwachungs- und Unterhaltungssystem für 10'000 Brücken in Thailand

Anders Borregård SØRENSEN
COWIconsult
Virum, Denmark

Per CLAUSEN
Danish Road Directorate
Copenhagen, Denmark

SUMMARY

The bridge Management and Maintenance System (BMMS) will introduce measures for:
— Inspection and reporting on the condition and load bearing capacity (rating) of the Department of Highway's approx. 10,000 bridges
— Ranking and budgeting of maintenance, repair, strengthening and replacement works in accordance with the current condition and rating results
— Standardized repair and strengthening of the bridges
— Rational administration of heavy transports.

RÉSUMÉ

Le système de gestion et de maintenance de ponts comprendra les points suivants:
— Inspection et rapport sur l'état et la capacité portante pour env. 10'000 ponts de la Direction des Routes principales
— Classement prioritaire et inscription au budget des travaux de maintenance, de réparation, de renforcement et de remplacement conformément aux résultats de l'inspection de l'état actuel et de la capacité portante
— Réparations standardisées et renforcement des ponts
— Administration rationelle des transports lourds.

ZUSAMMENFASSUNG

Das Programm für Ueberwachung und Unterhaltung von Brücken beinhaltet folgende Massnahmen:
— Inspektion und Bericht über Zustand und Tragfähigkeit der ca. 10'000 Brücken der Hauptstrassenabteilung
— Bewertung und Veranschlagung von Unterhaltung, Reparatur, Verstärkung und Ersatz in Uebereinstimmung mit dem aktuellen Zustand und den Berechnungsresultaten
— Standardisierte Reparatur und Verstärkung der Brücken
— Rationelle Abwicklung von schweren Transporten.

INTRODUCTION

Department of Highways is responsible for the maintenance of approximately 10,000 bridges on the routes that form the overall road network of Thailand. In the past, only the seriously damaged bridges were reported - if found - and subsequent budgets for the repair works were given. This situation caused DoH to hold a budget reserve each year, and the unforeseen amount varied from year to year. To manage the maintenance budget in the most effective way and to keep the bridges in good condition, DoH decided to set up a Bridge Management and Maintenance system.

A country wide bridge management and maintenance organization has been established and the future members have been involved in the system design from the very beginning in order to secure its acceptance in the Central Administration as well as in the districts. Fig. 1 shows the main activities in the system:

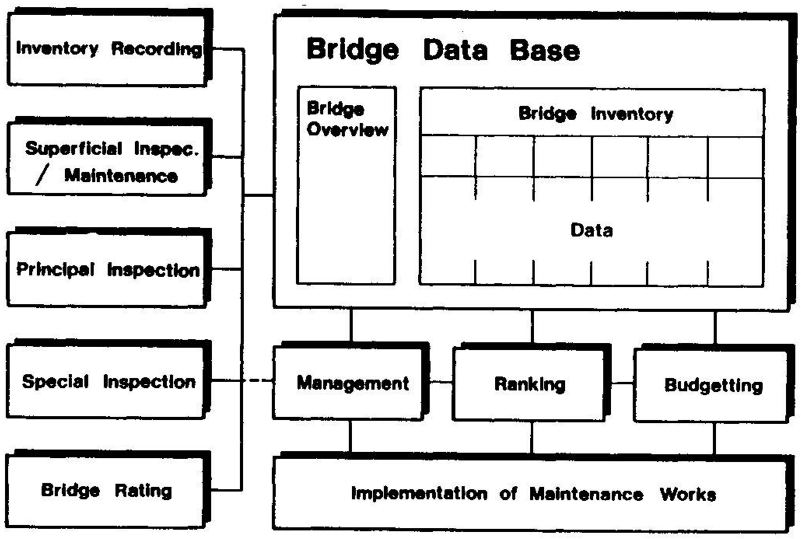


Fig. 1: Main activities covered by the system.

CONCEPT OF THE SYSTEM

General

The BMMS provides manuals for all the activities shown in Fig. 1, and it is run on personal computers.

All bridge information has been categorized and the updated combination of administrative, geometry and condition data is treated as the representatives for an element.

An element can be either the bridge as a whole or a specific part of the bridge. Each single element thus represents - and is limited to - such part of the bridge where unique information is needed. The user can select among elements as shown in Fig. 2, but he must always aim at using a minimum number of elements:

If the bridge is of uniform condition all over, only one element "Bridge" need to be used to represent the bridge.

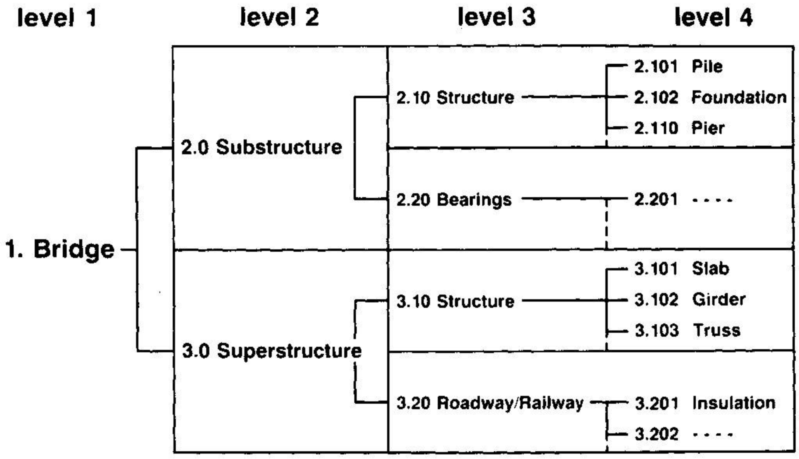


Fig. 2: Hierarchic structure shown with selected elements.

The System Modules

1. The Inventory Module

From April 1988 to February 1989 a full inventory recording of all the DoH bridges in Thailand took place. With all bridge data in a database the Inventory Module facilitates predefined output forms corresponding to routine inquiries. A typical predefined form is shown in Fig. 3.

Further, the user may create individual reports comprising selected data.

DEPARTMENT OF HIGHWAYS - BRIDGE OVERVIEW, IN GENERAL - THIS DATE: 31/08/02

Area Code: 610 Nakhon Ratchasima Division Rev. Date: 31-06-01
Route No.: ALL , Control Section No.: ALL , km: -
km: - BK

Route No.	Ctrl Sect No.	Structure No.	P S	km km BK	Name	F c n	Con/ Chan	Superstructure - design - material
2	0702	16	P	350.077	Ban Sida-Phon Huaisokkabuong	1	2506	-slab -reinforced concrete
2	0702	19	P	352.630	Ban Sida-Phon Huaiphailon	1	2506	-slab -reinforced concrete
2	0702	26	P	357.056	Ban Sida-Phon Huaisokkhong	1	2506	-slab -reinforced concrete
2	0702	28	P	357.519	Ban Sida-Phon Huailuk	1	2506	-slab -reinforced concrete

Fig. 3: Bridge overview

2. The Inspection and Bearing Capacity Module

There are three types of inspections: Superficial Inspection, Principal Inspection and Special Inspection.

- **Superficial Inspection** is carried out by the local road personnel with short intervals. They clean the structural members and repair minor damages in accordance with manuals. The maintenance level for each bridge depends on the importance of the bridge.

- **Principal Inspection** is carried out by local well trained engineers, who typically deal with about 500 - 1000 bridges. Inspection intervals are normally 3 years with a range of 1 to 6 years depending on condition of the bridge.

  The inspector evaluates damages and gives a condition mark to the elements - general or detailed - required to describe the condition of the bridge. Only significant damages may be registered if required for documentation. Further he estimates the remaining life and corresponding repair costs of each element, and recommends the time for the next Principal Inspection.

- **Special Inspection** is a detailed investigation of a bridge structure or parts of it, when the condition or load bearing capacity has reached an unacceptable level. This inspection is either initiated on basis of the recommendations by the Principal Inspector or on basis of the ranking list. Laboratory tests will normally be required. The findings in connection with an inspection will normally lead to alternative proposals for rehabilitation with corresponding budgets.

```
DEPARTMENT OF HIGHWAYS      - SPECIAL INSPECTION REPORT -     THIS DATE: 31/08/02
---------------------------------------------------------------------------------
Route No.:     2 , Control Section No.: 0702 , Structure No.:     19
Stationing: km 352.630 and km          BK
---------------------------------------------------------------------------------

Repair Proposal No.: 1

Superficial Inspection/Maintenance:
        Maintenance level (I minimum, II normal, III high) :    II
        Yearly budget  (1,000 Baht)                        :     5

Repair Works in 1,000 Baht:

Year     Element                          Method          Estimated repair costs
       Code    Name                                             (1,000 Baht)
---------------------------------------------------------------------------------
2531  BRIDGE  Bridge                      Repair                   850
2535  ABUWAL  Abutment Walls              Strenghtening            250
2541  EXPJOT  Expansion Joints            Exchange                  60
2560  BRIDGE  Bridge                      Exchange                3000
```

Fig. 4: Special Inspection report

The system comprises **Bearing Capacity** programmes for a continuous rating of the bridges based on the inventory data, including the inspectors' reports on current conditions, and for rating of actual trucks.

The bridge rating programme refers to a Standard Rating Truck and expresses the load bearing capacity in terms of this truck - the result is given as a bridge class. If the condition has deteriorated, the material strength is normally affected, and thus the actual bearing capacity of the bridge shall reflect a deficiency in material strenghts.

A similar vehicle rating programme expresses the effects of any actual vehicle in terms of the same Standard Rating Truck as mentioned above - the result is given as a vehicle class.

By a simple comparison of the vehicle class and the bridge class it is checked whether a given truck or specific truck type can pass the bridge.

Actual traffic along routes is analysed, and each bridge is given a rating mark that reflects the bridge class in relation to the maximum vehicle class on the route that passes the bridge.

```
-----------------------------------------------------------------
DEPARTMENT OF HIGHWAYS     - BRIDGE RATING -     THIS DATE:31/08/07
-----------------------------------------------------------------
Route No.:     2 , Control Section No.: 0702 , Structure No.: 16
Stationing: km 350.077 and km          BK
-----------------------------------------------------------------

Rating Document
---------------

Element: 3.10 Carrying Superstructure
Last Principal/Special Inspection: 31-08-02
Element's Condition Mark: 1
Remarks:

Load Case 1:

- Load factor on Dead Load       = 1.10
- Load factor on Standard Truck = 1.25

Inventory Rating Class: 133 %
Operating Rating Class: 104 %

Rating Mark: 0
```

Fig. 5: Results from the bridge rating

3. The Ranking and Budgeting Module

The condition marks and the rating mark of each bridge or it's elements form the basis of the ranking of bridges in need of repair. The importance of each element for the function and safety of the bridge is included in an element weight model, and also the importance of the route is considered before a final ranking point is calculated for the bridge. The rank of each element thus reflects:

- the current condition
- the bearing capacity in relation to the current traffic
- the importance of the element for the function of the bridge
- the importance of the route that passes the bridge

```
--------------------------------------------------------------------------
DEPARTMENT OF HIGHWAYS     - RANKING OF STRUCTURES -     REPORT DATE : 31/09/12
--------------------------------------------------------------------------

Ranking Points have been calculated  Year/Month/Day: 31-09-08

Structures are reported for the following unit(s) only:
 - Code: 610 Nakhon Ratchasima Division
```

Route No.	Control Section No.	Structure No.	Km	Km BK	No	Ranking Points Total Pr	Cond. Pc	Bear Cap. Pb	Cond. Mark BRIDGE
219	0500	14	14.961		3	49.2	24.6	24.6	3
2057	0100	27	16.117		10	11.0	12.3	0.0	2
2149	0101	28	11.933		43	5.5	6.1	0.0	2

Fig. 6: Ranking of bridges in need of repair

Alternative maintenance plans are considered for the bridges in the top of the ranking list for which maintenance works can be carried out within available budgets. Initial selection among alternative maintenance plans are normally based on net present value of the maintenance plan with possible considerations of traffic costs imposed on users by the work programme (detours etc.).

Alternative maintenance plans are only required for the immediate maintenance works and for accurate short term budgets. The estimates of replacement costs from the Principal Inspectors are generally used for the long term budgets where no detailed proposal is yet required.

The investment schedule for the selected maintenance alternatives or inspection estimates are summarized for all bridges and compared with available budgets.

When discrepancies between available funds and estimated maintenance costs are found, the user indicates from which period and to which periods activities shall be moved. The system then directs the identification of bridges where alternative maintenance activities exists in the "move to" period instead of the previously selected ones in the "move from" period.

DEPARTMENT OF HIGHWAYS - BUDGET - REPORT DATE : 31/09/12

The budget has been accepted Year/Month/Day: 31-10-01

Budget Report for the following unit(s) only:
- Code: 610 Nakhon Ratchasima Division

Year Cost in 1,000 Baht		2531	2532	2533	2534	2535	2536- 2546	2547- 2567	2568- 2593
Route No.	Contol Section No.								
2	0702	0.9	0.2	0.3	0.0	0.3	2	4	2
2	0800	0.0	0.0	0.0	0.0	0.0	1	3	6
2	0901	0.0	0.0	0.0	1.0	0.0	0	0	2
23	.0101	0.0	0.4	0.0	0.0	0.2	3	0	0
23	0102	0.0	0.0	0.0	0.5	0.0	0	1	4
23	0103	0.0	0.0	2.4	0.3	0.0	1	0	0
202	0500	1.0	0.5	0.0	0.0	0.0	0	0	0
207	0202	0.0	0.7	0.0	0.0	0.0	0	0	1
207	0300	0.0	0.0	0.0	0.0	0.0	0	0	0
219	0500	2.8	0.0	0.5	0.0	1.0	3	2	3

Fig. 7: Final budget for a Field Division.
* The Thai year 2531 corresponds to the Gregorian year 1988.

Conclusion

The majority of the 10,000 bridges have now been inspected, and it proves that the Bridge Management System will lead to:

- consistantly updated and objective information on each bridge
- improved basis for budgeting and maintenance planning by objective ranking of the bridges
- savings from more reliable and flexible budgeting due to the minimization of unexpected repair works
- savings from a more rational administration of the bridge network with a minimized need for inspections and data collection.

Re-evaluation of Structural Load Carrying Capacities

Détermination de la capacité portante actuelle

Ermittlung des vorhandenen Tragwiderstandes

Erik Yding ANDERSEN
Civil Engineer
COWI consult
Virum, Denmark

Erik Yding Andersen, born 1951, received his Civil engineering degree and Ph. D. at the Technical University of Denmark. For eight years he was involved in experimental research on silos. He is now involved in experimental investigations on structures and assessment of structural reliability.

SUMMARY

Inclusion of information on measured structural characteristics in probabilistic reliability analyses for re-evaluation purposes are briefly described. Application of the methods are illustrated by a case with re-evaluation of the reliability of a road bridge. Actual material strengths and test load results are used. The formal failure probabilities are further used for risk analyses where the total costs associated with various rehabilitation strategies are assessed, in order to select the best solution.

RÉSUMÉ

Les données sur les grandeurs caractéristiques mesurées sont inclues dans une évaluation probabiliste de l'état de la structure. L'application de cette méthode à un pont routier est illustrée à l'aide des résistances effectives des matériaux et des résultats d'essais de charge. Les probabilités de rupture sont employées avec les coûts globaux de différentes stratégies de maintenance, dans une analyse de risques pour le choix de la meilleure solution.

ZUSAMMENFASSUNG

Informationen über die gemessenen charakteristischen Grössen werden in einer probabilistischen Ermittlung des Bauwerkszustandes berücksichtigt. Die Anwendung der Methode wird anhand einer Strassenbrücke gezeigt, wobei wirkliche Baustoffestigkeiten und Resultate von Belastungsversuchen verwendet werden. Die Versagenswahrscheinlichkeiten werden für eine Risikoanalyse zusammen mit den Gesamtkosten verschiedener Instandstellungsstrategien zur Wahl der besten Lösung weiterverwendet.

1. INTRODUCTION

Increasing demands or progressing deterioration have resulted in a number of existing structures being found structurally insufficient when their load carrying capacity is assessed by current design methods.

As test loading of structures have often revealed that the load carrying capacities can be considerably larger than calculated, a wish for supplementary investigations arises in the aforementioned situation in order to utilize an existing structure best possible.

Load and resistance factor design methods are often inadequate for the inclusion of such supplementary information in the re-evaluation procedures mainly because no measure is given for the importance of deviations from the stated rules.

Probability based limit state analyses offers such possibilities primarily because a probability of limit state exceedance is calculated. The methods allows additional informations on the structures or structural parameters to be included (updating), whereby posterior evaluations are obtained.

The probability of failure in combination with the estimated costs of failure can be used to evaluate the risk associated with various maintenance strategies. The risk may be used as a decision parameter when priority must be given between various strategies.

2. PROBABILISTIC RELIABILITY ANALYSIS

Reliability models with distribution functions and parameters describing the random variation of the physical characteristics and the uncertainty of the calculation models are used to establish a reliability index - β.

The reliability index β is defined as $\beta = -\Phi^{-1}(p_f)$, where Φ is the inverse normal distribution function and p_f the probability of limit state exceedance (failure). It is calculated by the First Order Reliability Method (FORM) [3].

The probability of limit state exceedance or failure, p_f, may be expressed as

$$p_f = P\ (\ M(\underline{x}) < 0\ )$$

where the limit state function $M(\underline{x})$ of the basic variables, $\underline{x}$, is positive for all acceptable states of the structure (safe or intact), and negative for all unacceptable states (**failure** or unsafe).

The reliability depends upon the available information on basic parameters and upon the calculation model applied. The reliability is consequently not a physical property of the structure, but an evaluation variable applicable for decisions concerning the structure.

2.1 Improved basis for evaluation (UPDATING)

2.1.1 Updated basic variables and improved calculation models.

An improved evaluation of the reliability can be achieved by use of improved calculation models or by use of observed values of basic parameters, obtained either during construction or at a later stage. The observations are combined with the originally available information (prior), from which an updated probability distribution function is obtained by use of Bayesian statistical models [2],[5].

2.1.2 Updating of system reliability

When a structure has resisted external loads either through normal operation or during test loading, this is information on the load carrying capacity not available at the design stage. It may be used for updating of the reliability. However, the information relates not to a single basic parameter but to the entire structural system.

The failure probability for ordinary loads to be carried in the future can thus be expressed as

$$p_f = P(\text{ failure for L given safe for Q }) = P(L > R \mid R > Q)$$

$$= \frac{P(\text{ failure for L **and** safe for Q })}{P(\text{ safe for Q })} = \frac{P(L > R \cap R > Q)}{P(R > Q)}$$

where

L is Ordinary loads
Q is load(s) resisted by the structure (load history or test loading)
R is the resistance of the structure

The last formulation is made in order to facilitate calculations.

2.2 Risk analysis

Economic considerations incorporating the probability of failure may indicate whether it is reasonable to continue the use of the unaltered structure or precautions should be taken to improve the reliability [2],[4].

The formal failure probability p_f is used for the expected average frequency of failure for the structure, and the total expected cost, G, associated with a specific decision for the bridge is thus

$$G = c(p_f) + (d + c_{new}) \cdot p_f$$

$c(p_f)$ is construction costs for a new structure or costs of strengthening works and a decreasing function of the failure probability ($c(p_f)=0$ for an unaltered bridge).
d covers user losses, personal injuries, material damage at failure, the cost of demolition and any additional costs due to the aversion against larger catastrophic events.
c_{new} cost of a new structure as a substitute for the failed.

The optimal reliability level corresponds to the value p_f^*, which gives the minimum total expected cost G^*. The total cost G^* may be used as a decision parameter.

Only failures due to stochastic variations in the basic parameters for the bridge are considered, whereas gross errors are omitted [2]. It will lead to a conservative evaluation of the optimal failure probability p_f^*.

A considerable problem in the above mentioned calculations is the determination of the failure cost, d, mainly due to the difficulty of evaluating the capital costs of personal injuries and fatalities and aversions due to large catastrophic events [1],[4].

3. CASE STUDY: SMALL ROAD BRIDGE

The application of the principles outlined in the preceding chapter have been illustrated by an example covering a former road bridge Figure 1.

The bridge was test loaded before demolition and concrete cores and samples of the main reinforcement were taken for strength tests.

The variable load was considered to be a fixed parameter without any uncertainty due to insufficient stochastic informations on extreme traffic load effects in short to medium span bridges.

3.1 Structural calculational model

A grid of beam elements was used for the calculations. Bending and torsional effects were considered, but not shear. Elastic and plastic models are used and with and without of torsional stiffness of the beam elements.

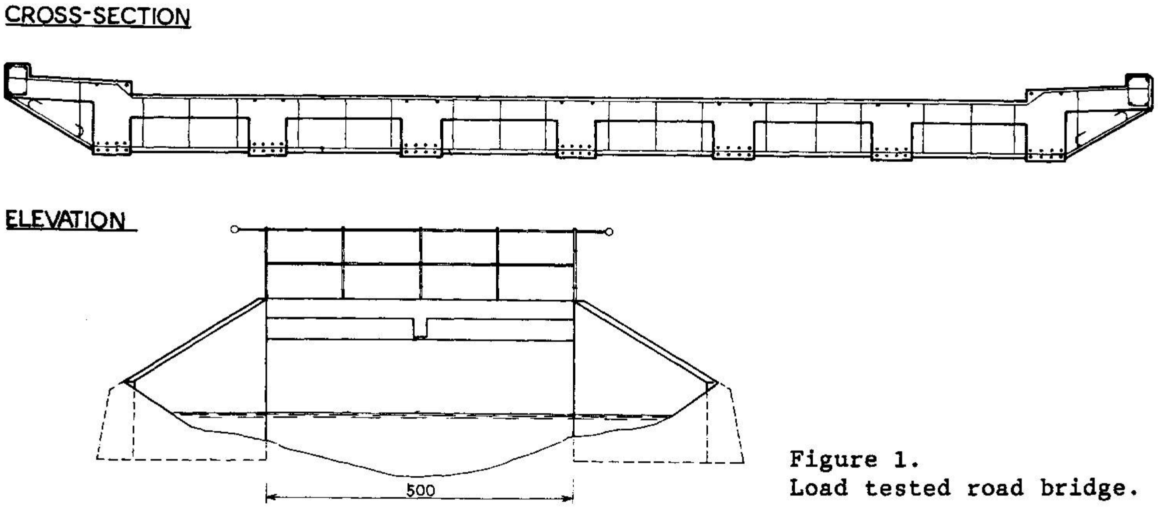


Figure 1.
Load tested road bridge.

One position of the traffic load and the corresponding internal forces were investigated.

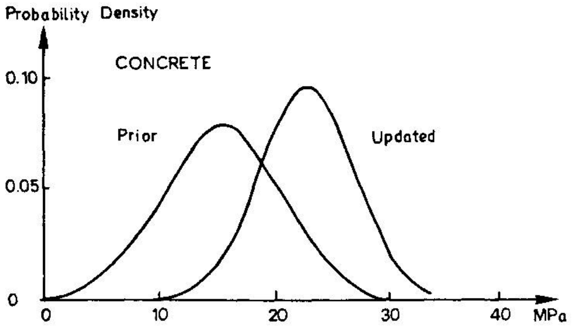
The influence of the calculation model on the reliability is shown in Figure 3.

Consideration of the correlation of material strengths between different cross sections has a significant influence, just like the distribution type for the basic parameters [2].

The possibilities for improving the evaluation of the structural reliability by improved calculation models are good, but it is not possible to require specific reliability levels for structures without specification of basic assumptions.

3.2 Updating material strengths

Results from laboratory tests with concrete and steel samples have been used to update the probability density functions for concrete and reinforcement respectively as shown in Figure 2.



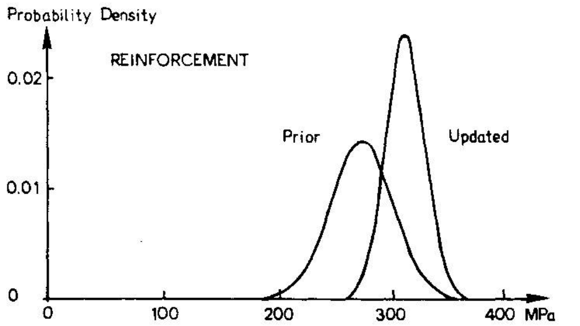


Figure 2: Probability density functions before (prior) and after (updated/posterior) observed material strengths are included.

The reliability index β is shown in Figure 4 for prior and updated strengths applied separately and in combination.

The reliability is increased considerably when both concrete and reinforcement strengths are updated. When only one material strength is updated the effect depends upon the load level. The reason is that both normal and overreinforced cross sections are considered to be possible failure modes for identical load levels and cross sections. This phenomenon requires special attention [2].

A possible way to utilize the results is to allow permissible loads to be increased to a level for which the prior and updated reliability are the same.

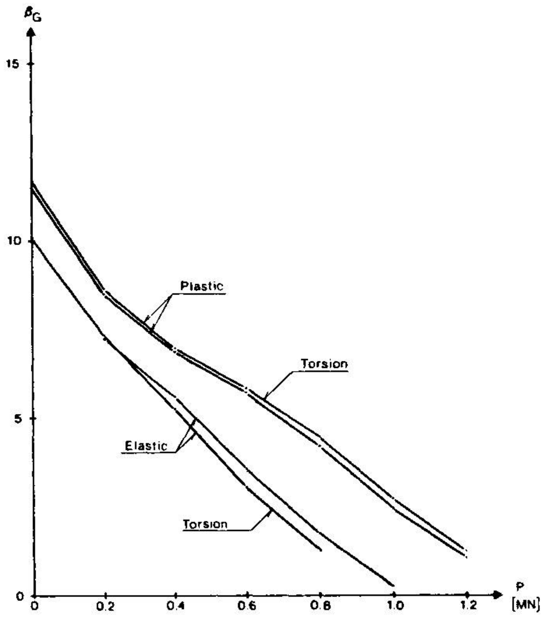


Figure 3: Reliability index β plotted against the variable load P
- 4 structural calculation models
- prior material strengths
- P deterministic

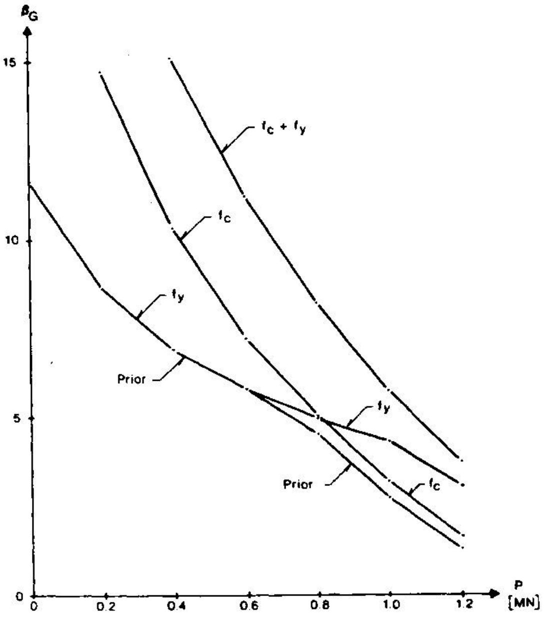


Figure 4: Reliability index β versus variable load P
- separate and combined updating of strengths f_c for concrete and f_y for reinforcement
- plastic model incl torsion
- deterministic load P, (D(P)=0)

3.3 Test loading

The effect on the reliability for test loading is shown in Figure 5. The test load Q is applied to the bridge in the same configuration as the variable load P. The test load Q is measured and thus considered deterministic. The variable load P has a constant mean value and six levels of the standard deviation for P, D(P).

A significant improvement of the reliability is obtained when the test load Q exceeds the mean value of the variable load P. For small standard deviations of the variable load, D(P), the benefit from a test loading is significant, but the effect is decreasing rapidly for increasing standard deviations, D(P). The required test load may be prohibitively high and it appears to be cheaper and more efficient to update the reliability through updating of material strengths and structural geometry in the present situation.

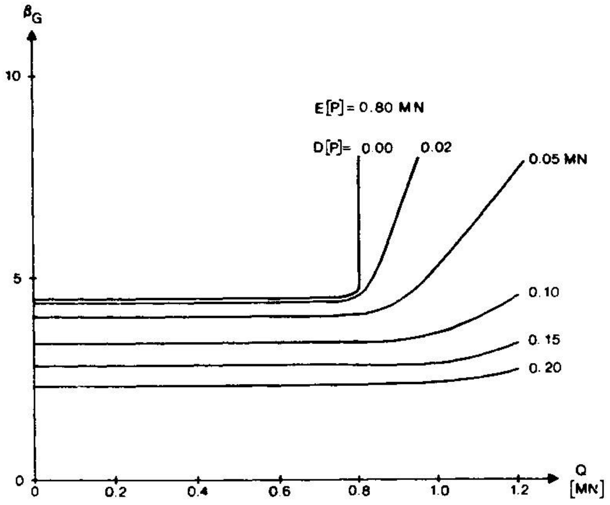
The reliability level is estimated on the basis of the random fluctuations in the basic parameters, but gross errors are not considered. As gross errors may be difficult or impossible to detect by other means than by test loadings they may still be relevant in certain cases.

3.4 Economic analysis

The principles of the risk analysis are used for analyzing three alternative decisions for the imagined future use of the bridge:

- continued use of the unaltered bridge (NTH)
- strengthening of the existing bridge (STR)
- demolition of the existing bridge and reconstruction (NEW)

For each load level the minimum total expected costs G_{NTH}, G^*_{STR} and G^*_{NEW} were determined. The minimum total costs are plotted in Figure 6.



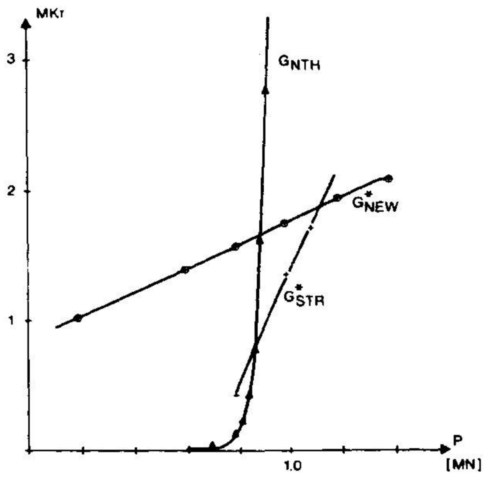


Figure 5: Reliability index β against the test load Q
- E(P) = 0.8 MN, Plastic model incl. torsion, prior strengths,
- six standard deviations D(P).

Figure 6: Minimum of expected total costs, G, for
- unaltered bridge : G_{NTH}
- strengthened bridge (FOR): G^*_{STH}
- new bridge (NY) : G^*_{NEW}

For the considered alternative decisions, the expected cost will be a minimum if the bridge is used unaltered for low load levels and is replaced by a new bridge for high load levels, whereas strengthening will be preferable for intermediate load levels.

REFERENCES

1. DITLEVSEN O., Fundamental Postulate in Structural Safety, Journ. Eng. Mech., vol. 109, no. 4, August 1983.
2. DANISH ROAD DIRECTORATE, Structural Reliability of Existing Bridges, Report no. 1, Case Study Madum Bridge, Bridge Department, 1986.
3. MADSEN H.O., KRENK S. and LIND N.C., Methods of Structural Safety, Prentice Hall Inc., New Jersey, 1986.
4. DITLEVSEN O. and ARNBJERG-NIELSEN T., Decision rules in re-evaluation of existing structures, Proc. Int. Symp. Re-evaluation of concrete structures, Reliability and Load Carrying Capacity, june 1988, to be published.
5. DITLEVSEN O., Uncertainty Modeling, with Application to Multidimensional Civil Engineering Systems, MGraw-Hill Int., 1981.

Assessment of a Bow-String Bridge 50 Years Old in Pavia

Evaluation d'un pont de type bow-string de 50 ans à Pavie

Beurteilung einer 50-jährigen Stahlbeton-Stabbogenbrücke in Pavia

Giorgio MACCHI
Prof. of Struct. Eng.
University of Pavia
Pavia, Italy

Author of several papers on prestressed concrete, reinforced concrete and masonry construction, with particular emphasis on plastic behaviour of statically indeterminate structures and nonlinear analysis. Chairman of CEB Commission II (Structural Analysis) and member of CEB Board.

Gaetano NOCCA
Consulting Engineer
Pavia, Italy

Gaetano Nocca, born 1937, obtained his civil engineering degree at the University of Genova. Since then he works in the field of design of bridges and other heavy civil engineering structures. He is partner of Teikos Consulting Engineers, Milan.

SUMMARY

In situ and laboratory measurements on a reinforced concrete bridge severely deteriorated by concrete carbonation provided data for a cost/benefit analysis. Diagnostic studies provide useful data for the improvement of the degradation models and realistic criteria for design and construction of durable bridges.

RÉSUMÉ

Les essais in situ et en laboratoire sur un pont en béton armé sérieusement endommagé par la carbonatation du béton a fourni des données pour une analyse prix/bénéfices. Les études diagnostiques fournissent des données précieuses pour l'amélioration des modèles de dégradation et des critères réalistes pour le projet et la construction de ponts durables.

ZUSAMMENFASSUNG

Messungen an einer stark geschädigten Stahlbetonbrücke und im Labor ergaben die für eine Kosten-Nutzen-Analyse erforderlichen Daten. Diagnostische Studien führen zu verbesserten Modellen bezüglich Schädigungsvorgängen und zu realistischeren Bemessungs- und Konstruktionskriterien.

1. INTRODUCTION

The criteria for design and construction of durable bridges shall be derived from appropriate models of the deterioration processes (chemical and mechanical), but may take advantage of the observation of the damages to existing structures.
In the limited field of reinforced concrete and prestressed concrete, the main reasons of degradation have been identified in:
- concrete carbonation and consequent steel corrosion
- action of de-icing salts and steel corrosion
- fatigue effects of loading on concrete and steel.

Such phenomena are qualitatively known [4], but quantitative models are still vague or totally lacking, so that their application to the design of new structures is not yet reliable. On the other hand, the large number of badly damaged existing bridges [1] constitutes a large set of specimens for study, and the current numerous works of diagnosis and assessment are precious opportunities for the acquisition of new knowledge.
In several cases the pathological state of the bridge is so advanced that demolition and reconstruction have been already decided; the structure is then an ideal laboratory , where tests on the residual life may be performed.
The Structural Mechanics Department of the University of Pavia and the Structural Engineering Department of the University of Rome are performing along such lines a joint theoretical and experimental research on the viaduct Pecora Vecchia of the highway Bologna-Florence, which shall be replaced; the research is focused on the synergetic effects of mechanical and chemical agents [2].
The present report is dealing with a previous experience of diagnostic studies on a r.c. bridge in Pavia, which is now being replaced. Further laboratory tests on parts of the structure could bring additional informations on the studies of degradation processes.

2. THE BRIDGE

Built in 1932, the bridge (Fig.1) has two spans of 38 m, with a bow-string statical scheme, a popular structure in the 30ies.
A first reason of deficiency was the design load, considerably less than the present design load, and than the load of vehicles frequently present on the National roads. The design of the superstructure was based on an axle of 140 kN, when the present Code imposes a load of 360 kN on an area of 2.6 by 1.0 m, and a dynamic factor 1.34. Tandem axles of 260 kN are indeed very frequent today.
If the deficiencies were obvious in the grid of the superstructure, they were not evident for the main resisting elements of the bridge, because of the great preminence of permanent loads. However, a dramatic spalling of concrete and corrosion of reinforcement (Fig.2) led to the need of a deep diagnosis for the assessment of the residual carrying capacity and for decisions about its possible repair or necessity of replacement.
The conclusions of the tests and of the analyses led to establish the loads temporarily admissible on the bridge (100 kN vehicles) and provided the necessary data for a cost/benefit analysis. The data, combined with town planning considerations, led the Authorities to the decision of replacing the bridge with a new one.

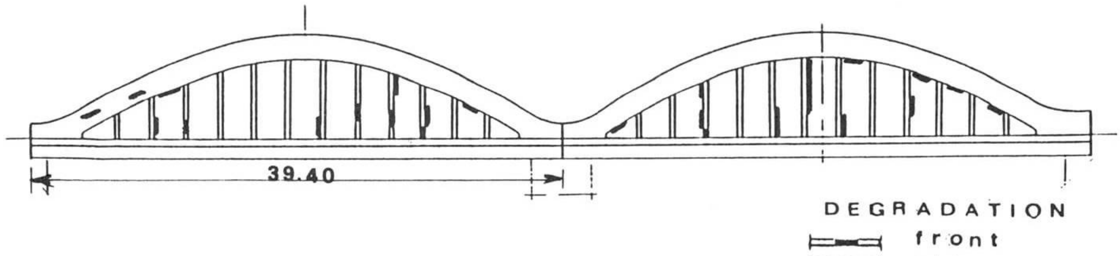


Fig.1. The bridge in Pavia and deteriorated parts in elevation

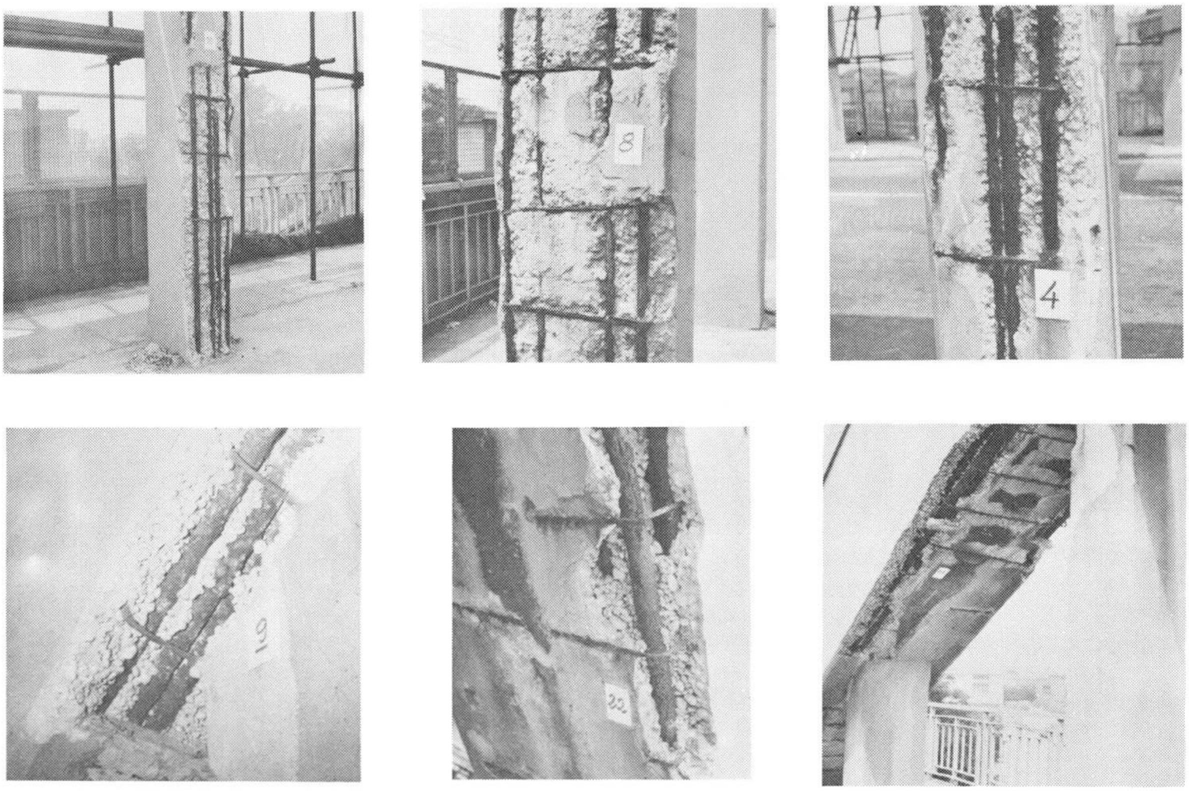


Fig.2. Corrosion of reinforcement and spalling of concrete on various elements of the bridge

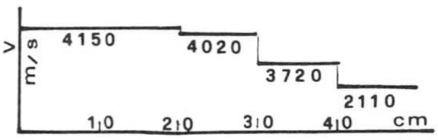


Fig.3. Decrease of ultrasonic pulse velocity for increasing path length (detecting intermediate cracks

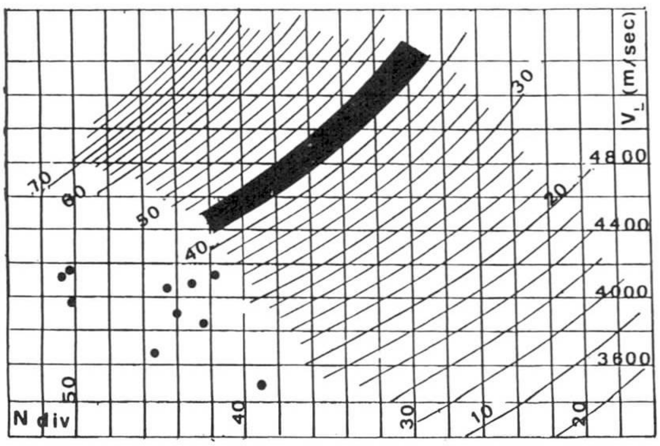


Fig.4. Concrete strength from hammer rebound index (sclerometer Schmidt)and ultrasonic pulse velocity (m/sec.). Shaded area is drilled core strength.

3. IN SITU AND LABORATORY MEASUREMENTS

3.1. Concrete

The strength and quality of concrete are needed not only for the assessment of the structure, but also for its correlation with the degradation phenomena, mainly carbonation and chloride penetration.
Non-destructive testing shall combine hammer rebound (Schmidt sclerometer) with ultrasonic pulse velocity (the RILEM SONREB method, Fig.4), because the first procedure is sensitive to superficial hardening due to carbonation, and the second one is sensitive to microcracking.
The detection of cracks and the elimination of their effects on measurements has been obtained by measuring the speed on different lengths on the same line (Fig.3); the highest values of speed have been retained as a property of the uncracked material.
The values of non-destructive tests are compared with the strength obtained on drilled cores, in the diagram SONREB (Fig.4).
The product of the correction factors may be roughly considered equal to 1.
The experimental points are plotted on the standard chart (the measurements in the damaged areas have been excluded); the band 42 to 46 MPa of the drilled core strength has been shaded.
It clearly appears that all the experimental points fall out of the area calibrated for new concretes. It means that the hammer rebound is measuring a skin hardness which is not correlated in the usual way with E modulus and strength, and this is due to the carbonation of concrete.
On the other hand, the ultrasonic velocity seems to underestimate the strength, perhaps due to a microstructural damaging of the old concrete. We cannot state anything of this kind at present; nevertheless, a warning is necessary: the use of non-destructive testing requires a special calibration for old structures.

3.2 Carbonation depth

The observed depth of carbonation was variable, but by far larger than expected according to the existing knowledge.
The measured values are plotted in Fig.5, and compared with the curve given in [10] for the development of carbonation with time.
Such unexpected level of the phenomenon cannot be associated with a bad quality of concrete; the strength was in fact exceptionally high for the time.
The question should be answered by further research on old structures and diffusivity in old concretes.

3.3 Corrosion of reinforcement

The dramatic spalling of concrete, mainly in vertical elements, as shown by Fig.2, is in complete agreement with the description of the phenomenon given in literature: the expansion of steel due to oxidation creates a pressure causing the spalling of the concrete cover; and oxidation is due to the destruction of the passivation layer caused by the carbonation of concrete.
At the time of the diagnosis the corrosion was already in progress on most reinforcing bars of the vertical tendons (all bars being disposed near to the external surface).
A reduction of 40% of the initial section of one bar has been observed locally; this value cannot obviously be generalized, but it caused a serious concern for the safety of the structure, because the progress of corrosion process in time cannot be estimated at present with reliable models.

3.4. Structural analysis

In the design of old structures the analysis has been often oversimplified;the modern methods of analysis can provide sometimes useful informations in the

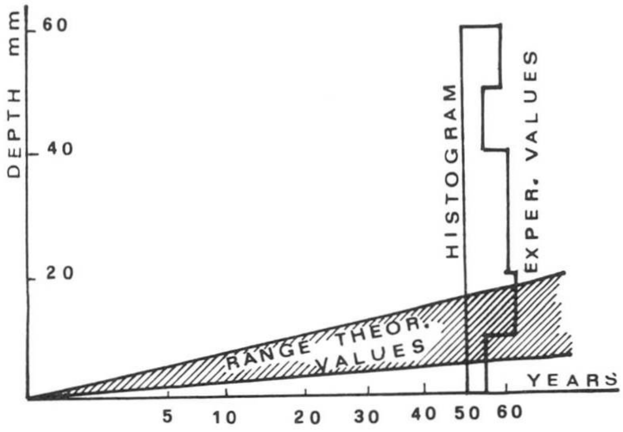


Fig.5. Carbonation depth vs time; plots indicate measured values.

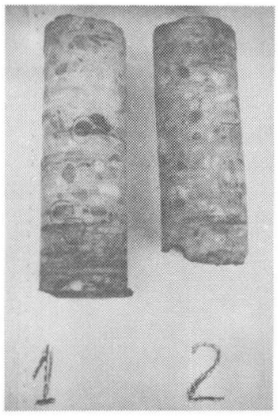


Fig.6. Drilled concrete cores.

diagnosis.
The 45 degree skewness of the bridge was not considered in the original analysis; therefore, the accurate analysis showed that conservative assumptions were done for the main resisting elements (as arches and ties), but bending and torsional moments were largely underestimated in the grid elements of the deck. Some elements needed urgent strengthening, but the entire superstructure was not in measure of carrying the present traffic loads, even in case of no degradation.
However, the analysis alone may be insufficient and ignore essential stresses which can appear under loading tests; it has been the case of the vertical tendons.

3.5. Loading tests.
In fact, the loading tests performed under the reduced load of vehicles of 100 kN (which was indicated as permissible load by the analysis) reserved a surprise: strain gauges applied on little degradated parts of the vertical tendons showed that the vehicles caused local stresses in the reinforcement of 170 MPa, several times the stresses anticipated by the analysis.
Such overstress can be explained if the redistribution of internal forces is considered between highly degradated tendons and the others: the formers lose the contribution of concrete and the corresponding stiffness, so that loads are transferred to the still rigid undamaged tendons. Such uneven stress distribution has been observed during the tests; the resulting stress range in steel was an unexpected danger of fatigue failure.

4. SUGGESTIONS FOR DURABLE BRIDGES.

The diagnosis of the old bridge has been the occasion for re-considering the criteria of design of r.c. and prestressed concrete bridges.

- Concrete.
The phenomena observed on the old concrete (both in the highly damaged elements and in the still well preserved parts) suggest to modify the criteria of design in the direction of an increase of durability and strength of concrete without increasing the stresses in service.

The equation: " strength = durability " has not been confirmed by our tests on the old concrete, which showed high carbonation depth in spite of the (apparently) high strength. However, extensive tests on modern high strength concretes show that they have much less porosity than ordinary strength concrete, and therefore less expected carbonation depth; moreover, less microcracking under short term and long term loading is a guarantee against progressive degradation and fatigue failure.
This concept implies to stop , at the same time, the constant increase of the stress level which has accompanied so far the improvement of the concrete quality.
- Cover.
The decrease of concrete porosity cannot be expected to reduce the carbonation to such an extent to justify the present values of cover; if carbonation depths higher than 40 mm (and locally 60 mm) have been observed in the 52 year old bridge, in spite of an overall good quality of construction, it would be wise to ensure a cover of at least 40 mm to a great percentage of the steel area of the main structural elements. We are perhaps forced to use different criteria for slabs, but envisage other kinds of protection!
- Diameter of bars.
The large diameter bars used in the 30ies provided a substancially safe life to the bridge in the essential elements (arch and tie). We should no more encourage the use of small diameters. Large diameter bars provide higher durability also in prestressed concrete bridges.
- Minimum thickness of resisting members.
Thin members have been used in the past, in the research of light and economic structures. Now thin girders and slabs are the most deteriorated parts of bridges both for the chemical attack and the fatigue action.
Minimum thickness has to be increased, and thick slabs should , when possible,substitute the conventional grids or box girders.

5. ACKNOWLEDGEMENTS

The support of the Municipality of Pavia and of the Italian Ministry of Education (MPI) are gratefully acknowledged.

6. REFERENCES

1. THUL H., Maintenance of Reinforced and Prestressed Concrete Bridges in the Federal Republic of Germany. IABSE Symp. Maintenance Repair and Rehabilitation of Bridges, Washington DC, 1982.
2. MACCHI G., RADOGNA E.F., MATERAZZI A.L. The synergetic effects of environment actions and fatigue. IABSE Symp. Lisbon 1989.
3. MACCHI G.,RADOGNA E.F., MAGENES G., MATERAZZI A.L. Probabilistic analysis of safety of bridges versus fatigue. CEB Bulletin n.185, 1988.
4. CEB Bulletin n.166. Draft CEB Guide to durable concrete structures.
5. KAVYRCHINE M. Inspection and Monitoring. IABSE Symp. Strengthening of Building Structures. Diagnosis and Therapy. Venice 1983.
6. TASSIOS T.P. Prolegomena to a generalised Design Space. CEB Bulletin n.152 1984.
7. SENTLER L. Service life predictions of concrete structures. Report TVBK 3018. Lund 1983.
8. GOTFREDSEN H.H., The decisions of non repair. CEB Bulletin n.152, 1984.
9. DE SITTER W.R., Costs for service life optimization. The "law of fives". CEB Bulletin n. 152. 1984.
10. SIEMES A.J.M., VROUWENVELDER A.C.W.M., VAN DER BEUKEL A., Durability of buildings: a reliability analysis. Heron, n. 3, 1985.
11. MARTIN H., RAUSEN A., SCHIESSL P., Carbonation of concrete made with different types of cement. Coll.Behaviour of Concrete Struct. Liege 1975

Beständigkeitsprobleme an der Ölandsbrücke in Schweden

Durability Problems at the Öland Bridge, Sweden

Problème de durabilité du pont d'Öland en Suède

Werner VON OLNHAUSEN
Brückenchef
Reichsamt für Strassenwessen
Borlänge, Schweden

Werner von Olnhausen, geb. 1928, erhielt seinen Dipl.-Ing. Grad 1954 an der Techn. Hochschule Hannover, BRD. Danach hat er in Schweden mit Entwurf, Konstruktion und Normung im Brücken- und Ingenieurbau in Industrie, Verwaltung und als beratender Ingenieur gearbeitet.

Clas-Göran RYDÉN
Bauingenieur
Reichsamt für Strassenwesen
Borlänge, Schweden

Clas-Göran Rydén, geb. 1957, M. Sc. 1982, Tekn. lic. 1986 an der Techn. Hochschule Luleå, arbeitet hauptsächlich mit der Entwicklung von Methoden zur Feststellung und Verhinderung von Stahlkorrosion im Beton.

ZUSAMMENFASSUNG

Der Artikel beschreibt die an den Pfeilern der Ölandsbrücke auftretenden Korrosionsschäden sowie Testausführungen von Sanierungsmassnahmen.

SUMMARY

The article describes corrosion damage at the piers of the Öland bridge and measurements for rehabilitation.

RÉSUMÉ

L'article décrit les dommages causés par la corrosion sur les piles du pont d'Öland, ainsi que les mesures d'assainissement étudiées.

ALLGEMEINES. ÜBERBAU UND UNTERBAU.

Die 1967-1972 gebaute Brücke zwischen dem schwedischen Festland und der im Südosten vorgelagerten Insel Öland ist mit 6.072 m Länge immer noch Europas längste Brücke. Sie besteht aus einem Hochbrückenteil, Fig. 1, mit 6x130 m + 2x60 m Spannweite in 36 m Höhe über der Schiffahrtsrinne, der als Spannbetonkastenkonstruktion im Freivorbau ausgeführt ist, und aus zwei Anschlussbrücken, Fig. 2, mit 23x35 m bzw. 124x35 m Spannweite, die als kontinuierliche Stahlbetonbalkenbrücken mit Hilfe von zwei Vorbauwagen am Platz fachweise betoniert wurden. Die Brückenbreite ist 13 m. Die Anschlussbrücke auf der Ölandseite folgt auf etwa 3,5 km Länge bei nur geringfügiger Steigung (1,7 %) der Wasseroberfläche, wobei der tiefste Punkt des Überbaus am Ölandwiderlager auf Kote +3,5 m liegt. Bei Dimensionierung und Ausführung des Überbaus wurde grosse Mühe auf gute Beständigkeit verwandt, um den erwarteten Frost-Tausalzbeanspruchungen widerstehen zu können. Überall wurde Beton K 40, in der Fahrbahn mit Luftzusatz, verwendet. Die Brückenoberfläche erhielt Gussasfaltisolierung, freiliegende Betonflächen wurden mit Silanpreparaten behandelt. Wasserdichte Fugen wurden mit fast 600 m Abstand eingebaut.Der Uberbau der Brücke hat sich bisher gut bewährt.

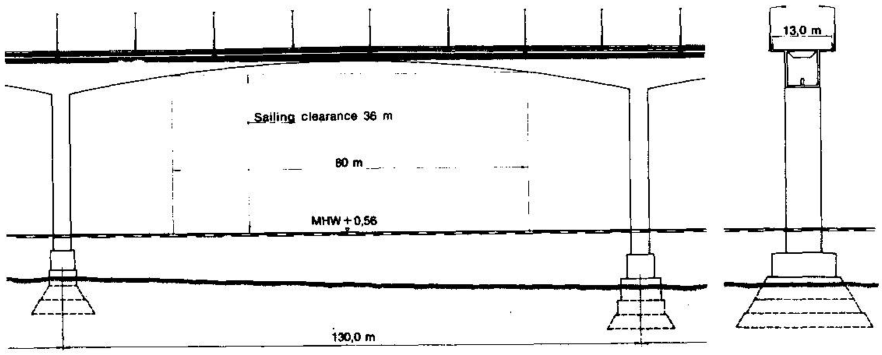


Fig. 1 Hochbrücke.

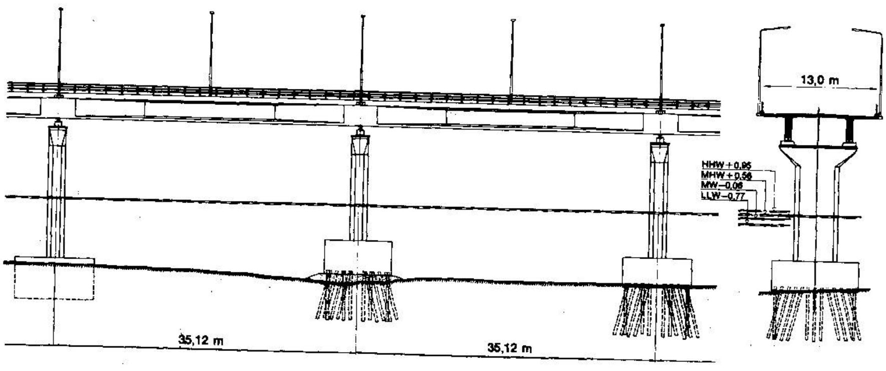


Fig. 2 Anschlussbrücken.

Der Unterbau besteht aus innerhalb von Spundwänden unter Wasser gegossenen Bodenplatten, die direkt auf dem Sand oder auf Pfahlrosten aus Betonschlagpfählen ruhen. Die Pfeilerschäfte sind im Bereich der niedrigen Anschlussbrücken massiv mit reichlichem Betonquerschnitt c:a 2 m x 5 m, ausgeführt, der nach Auspumpen der Spundkammern im Trocknen gegossen wurde. Bewehrung liegt nur an den Aussenflächen. Sie ist für einen beachtlichen Eisdruck bemessen. Oberhalb der Pfeilerschäfte liegen Hammerkopfbalken, die für sich betoniert und schon nach kurzer Zeit mit dem Rüstungswagen belastet wurden. Der Baufortschritt war ein Fach in 14 Tagen mit jedem Wagen. Die Produktion der Anschlussbrücken wurde äusserst rationell betrieben, wobei allerdings die Qualitätsfragen beim Unterbau wenig Beachtung fanden. Die Ausführung entsprach jedoch der zum Bauzeitpunkt üblichen Sorgfalt für Bauteile ohne besondere Anforderungen. Die Pfeiler stehen im Wasser der Ostsee, die im Brückenbereich einen Salzgehalt von nur 0,3-0,4 % hat. Die Einwirkung dieses Milieus auf den Beton wurde stark unterschätzt. Die Specifikation verlangt nur Beton mit K 30 als Druckfestigkeit, Luftzusatz im Bereich der Wasserlinie und im übrigen die allgemeinen Forderungen der damals ganz neuen schwedischen Betonbestimmungen von 1965. Es ist heute - zurückschauend - leicht, eine Reihe von Faktoren anzugeben, die für die Beständigkeit der Pfeiler nachteilig waren. Folgende Liste ist sicher nicht vollständig:

- Die Pfeileroberkante hätte etwa 1 m höher verlegt werden müssen, um die Spritzwasserwirkung zu reduzieren.
- Die Betonüberdeckung sollte 45 mm statt 30 mm sein.
- Die Hammerkopfbewehrung hätte mit geringeren Dimensionen und besserer Verteilung (Scheibenbewehrung statt Balkenbewehrung) ein günstigeres Rissbild ergeben.
- Der Betonzuschlag - besonders für Lieferungen von der Ölandseite - enthält in gewissem Umfang poröse Bestandteile.
- Der Zementgehalt ist mit 270 kg/m^3 zu gering um dichten Beton zu garantieren. Der Wasser/Zementfaktor liegt nahe 0,7, was für die Festigkeit ausreicht, aber hohe Porosität zur Folge hat.
- Als Anmachwasser kam Seewasser zur Verwendung, wodurch ein Grundgehalt an Chlorid von etwa 0,2 % entstand.
- Die Steighöhe beim Betongiessen lag bei etwa 2 m/Std, was schlechtes Verdichten und Setzungen herbeiführte. Zum Verdichten der grossen Betonmenge von 20 m^3/Std standen nur 1-2 Vibratorstäbe zur Verfügung. Schlechte Verdichtung führte zu Densitätswerten bis herab auf 2,1 t/m^3.
- Weder Kühlung noch Formisolierung wurden eingesetzt um die Temperaturgradiente zu begrenzen, die vermutlich bis auf 40°C anstieg. Die Nachbehandlung des Betons beschränkte sich auf Feuchthalten bis zum Ausschalen, das schon nach kurzer Zeit (etwa 2 Tagen) stattfand.
- Frühbelastung der Hammerköpfe führte zu verstärkter Rissbildung.

2. AUFTRETENDE PFEILERSCHÄDEN

Um 1980 wurden in einer Reihe von Pfeilerköpfen Querrisse in den Hammerbalken festgestellt. Die Bewehrung ist hier aus der Konsolwirkung der Auflager hochbeansprucht, aber nach einer einfachen Fachwerkstheorie ausreichend bemessen. Ein Mangel besteht in der Konzentration der Bewehrung an den oberen Rand. Der Hammerbalken wirkt als Scheibe und müsste einen Teil der Bewehrung auf das obere Drittel verteilt erhalten. In diesem Bereich sind die Risse am grössten. Die Pfeilerköpfe der niedrigen Pfeiler bis etwa 1 km vom Ölandswiderlager enthielten Chloridkonzentrationen bis zu 5 % der Zementmenge im Bereich der Bewehrung, die auf Spritzwasser vom Wellenschlag zurückzuführen waren. Das Chlorid reicherte sich auf hohe Werte an durch abwechselndes Überspülen und Austrocknen, trotz des geringen Salzgehalts im Seewasser. Die Oberflächenbewehrung auf den Seiten der Pfeilerschäfte war zum Teil stark korrodiert. Besonders im Spritzwasserbereich und in der Wasserlinie war die Deckschicht an vielen Stellen lose und durch die sich volummässig ausdehnenden Korrosionsprodukte der Bewehrung (Rotrost) abgesprengt, so wie das bei Oberflächenkorrosion unter Zugang von Sauerstoff zu erwarten ist. Zu unserer Überraschung setzte die Korrosion fort auch in Pfeilerteilen weit unter der Wasserlinie, wo mit Sicherheit kein Sauerstoff mehr vorhanden ist. Wir fanden einige grobe Bewehrungstäbe mit etwa 25 % Materialverlust am Übergang der Pfeilerschäfte in die Bodenplatte bei etwa 6 m Wassertiefe. Die Betondeckschicht war hier nicht abgesprengt, zeigte auch keine "Boom"-Tendenzen mit dem Betonhammer. Die Korrosionsform war vom Lochfrasstyp, das Korrosionsprodukt war schwarz, zeigte nur geringe Volumausdehnung, teilweise war die Staboberfläche noch intakt, während das Innere des Stabes stark angegriffen war. Elektropotentialmessung gab an der Betonoberfläche eindeutige Indikation des arbeitenden Korrosionsprocesses, d.h. eines vollausgebildeten galvanischen Elements, wo das Eisen in Anwesenheit von Chloridjonen im Seewasser in Lösung geht und in Jonenform die undichte Deckschicht passiert. In einigen Fällen zeigte der Beton an der Oberfläche schwarze Flecken, meist dort wo die Deckschicht dünn war. Auch dieser Korrosionstyp - chloridinitiierte Korrosion - ist an und für sich wohlbekannt, jedoch noch nicht so sehr im Betonbau. Es gibt heute aber bereits Experten, die behaupten, dass man für Beton im Seewasser auf die Dauer immer Korrosionsschäden der genannten Art erhält, wenn man nicht den wirksamen galvanischen Elementen durch einen gegengerichteten Schutzstrom entgegenarbeitet (kathodischer Schutz). Demnach wäre es ausgeschlossen bei starker Chloridbelastung im Wasser einen zuverlässigen Korrosionsschutz allein durch eine hochwertige Betondeckschicht zu erreichen.

Viele dauerhafte Seebauten widersprechen dieser Behauptung. Wir sind deshalb in der schwedischen Brückenverwaltung der Meinung, dass die Probleme an der Ölandsbrücke mit betontechnischen Mitteln zu lösen sind, d.h. dass wir die Bewehrung in dichten hochwertigen Beton einbetten müssen. Andererseits sind die Neuentwicklungen auf dem Gebiet des kathodischen Schutzes so wichtig, dass wir uns über die neuen Möglichkeiten informieren und die zugehörigen Techniken studieren und prüfen müssen. 1988 wurden 4 Probepfeiler saniert, wobei nicht nur hochwertiger Beton, sondern auch verschiedene Kathodische Schutzsysteme zur Anwendung kamen. Weiterhin wurde die eingebohrte Bewehrung am Übergang zur Bodenplatte mit Epoxibehandlung eingebaut.

3. SANIERUNGSMASSNAHMEN

Die vorgesehenen Sanierungsmassnahmen können im Rahmen des begrenzten Umfanges dieses Artikels nicht im Einzelnen beschrieben werden. Für die Mehrzahl der Pfeiler, Fig. 3, wird der Pfeilerschaft mit einer bis 2 m unter die Wasserfläche herabreichenden angedrückten Arbeitskammer versehen. Der chloridverseuchte Beton wird mittels Waterjet-Strahlen bis etwa 10 cm hinter der Bewehrung abgearbeitet. Die Bewehrung wird erneuert, mit einem Anodennetz versehen und mit hochwertigem Beton eingegossen. Im Bereich von -2 m bis zur Bodenplatte erhält die Bewehrung einen Kathodenschutz mittels im Seewasser aussen an den Pfeilern angebrachten Magnetitanoden für aufgedrückten Schutzstrom.

Der Hammerkopf, Fig. 4, wird an der Oberfläche auf die gleiche Weise saniert und wird ausserdem durch 4 eingebohrte horizontale Ge-Wi-Stäbe Ø 50 in Brückenquerrichtung c:a 70 cm unter Pfeileroberfläche abgesichert. Der Pfeilerbereich oberhalb c:a 6 m über der Wasserfläche wird mit Spritzbeton saniert. Für einige Pfeiler mit starker Bewehrungskorrosion unter Wasser, wird der gesamte Pfeilerschaft mit einer selbsttragenden c:a 40 cm dicken bewehrten Betonschale aus hochwertigem Beton versehen, wobei die geschädigten Pfeiler ohne Abarbeiten eingegossen werden.

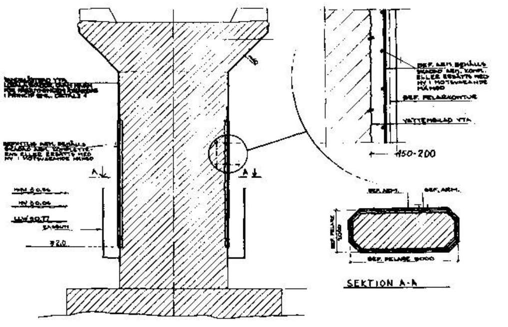


Fig. 3 Sanierung des Pfeileroberteile.

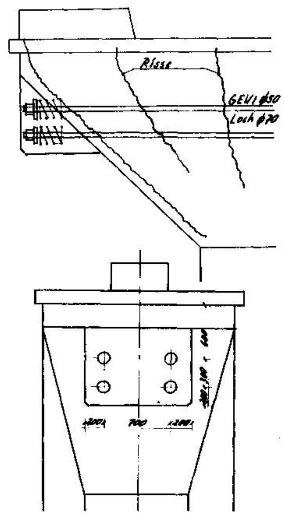


Fig. 4 Rissverstärkung.

4. KATHODISCHER SCHUTZ

4.1 Korrosionsschäden

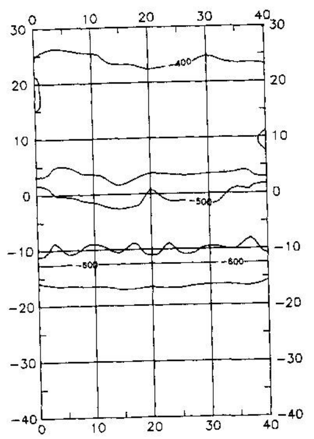


Fig. 5 Elektropotentialwerte für Pfeiler.

Wie oben angedeutet wurden elektrokemische Potentialmessungen durchgeführt, um den Umfang der Bewehrungskorrosion klarzulegen. Fig. 5 zeigt ein Beispiel des Resultats von solchen Messungen. Negative Potentialwerte von unter -350 mV im Verhältnis zu einer $Cu/CuSO_4$-Elektrode wurden bis zum Niveau 3 m über der Wasserlinie an allen getesteten Pfeilern gefunden. Daraus kann man schliessen, dass Bewehrungskorrosion in diesem Bereich im Gange ist. Potentialmessungen geben jedoch keine Unterlage zur Beurteilung des Umfanges der Korrosionsschäden. Die Indikationen der Potentialmessungen wurden durch Freilegen der Bewehrung bestätigt. Die Bewehrungskorrosion war umfangreich bis 3-4 m über der Wasserlinie. Das Aussehen der Korrosionsschäden und deren Lage deuteten auf 3 verschiedene Arten von galvanischen Korrosionszellen in diesem Bereich, siehe Fig. 6.

Elektropotentialmessungen unter der Wasserlinie ergaben Messwerte von -500 bis -650 mV. Dies kann man so deuten, dass 1.) die passive Schicht auf der Bewehrungsoberfläche geschädigt ist und dass 2.) der Zutritt von Sauerstoff geringen Einfluss hat. Das Korrosionsrisiko unter der Wasserlinie wird hauptsächlich vom elektrischen Leitungsvermögen des Betons bestimmt. Das Leitungsvermögen erwies sich bei Messungen als hoch, weshalb Bewehrungskorrosion unter Wasser möglich scheint. Beim Freilegen der Bewehrung wurden auch tatsächlich Korrosionsschäden unter Wasser festgestellt. Fig. 7 zeigt ein Modell, das diese Korrosionsangriffe als eine klassische galvanische Zelle mit grosser physischen Ausbreitung erklärt.

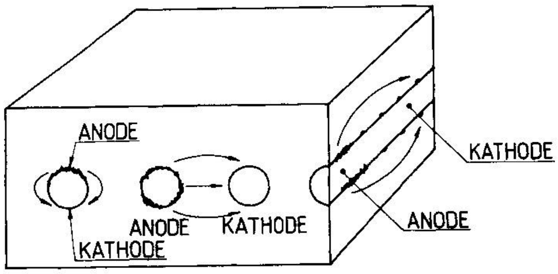


Fig. 6 Verschiedene Korrosionszellen. Der galvanische Strom fliesst in Richtung der Pfeile.

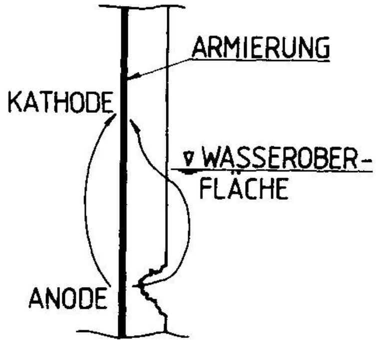


Fig. 7 Galvanische Korrosion unter der Wasserfläche.

4.2 Kathodischer Schutz über Wasser

An den Pfeilern, die 1988 repariert wurden, sind als Versuch Anordnungen für kathodischen Schutz angebracht. Zwei verschiedene Anodenmaterialien, Kabel aus elektrisch leitendem Plast (Raychem Ferex) bzw. Streckmetall aus Titan (Eltec Elgard) wurden verwendet. Jedes Anodensystem ist in verschiedene Kreise unterteilt um das Anpassen an variierende Feuchtigkeiten, die in den verschiedenen Pfeilerteilen vorliegen, zu ermöglichen. Das Anodenmaterial wurde auf der freigelegten Bewehrung an den verschiedenen Pfeilern mit Hilfe von speciellen Montierungsdetails und Distanzklötzchen angebracht. Dieses Verfahren bereitet grössere Schwierigkeiten als das normale Anbringen von Anoden auf einer fertigen Betonfläche. Viel Mühe wurde darauf verwandt die Anoden so zu befestigen, dass sie beim Betonieren ihre Lage beibehielten.

4.3 Kathodischer Schutz unter Wasser

Schon früher waren an der Ölandsbrücke Versuche mit kathodischem Schutz der Bewehrung unter Wasser mittels Opferanoden ausgeführt worden. Opferanoden aus Zink gaben einen ausreichenden Schutz, während Aluminium schlechter funktionierte, vermutlich war der Salzgehalt des Wassers zu gering. In den Probepfeilern von 1988 wurden für den Schutz unter Wasser Systeme mit aufgedrücktem Strom installiert. Die Anoden bestehen aus gesintertem Magnetit (Bergsöe Anti Corrosion) und werden in unmittelbarer Nähe der Pfeiler plaziert um Läckstromkorrosion an den bewehrten Betonpfählen, auf denen die unbewehrten Unterwasserbodenplatten ruhen, zu vermeiden.

4.4 Betrieb des kathodischen Schutzes

Probepfeiler wurden mit bis zu 12 Stück eingegossenen Referenzzellen versehen, um die laufende Überwachung der Funktion des Kathodenschutzes zu ermöglichen. Die Referenzzellen sind vom Silber/Silberchloridtyp, mit komplettierenden Referenzzellen vom Grafittyp. Kommende Untersuchungen und Proben sollen uns Wegleitung für den künftigen Betrieb des Kathodenschutzsystems geben.

Examining, Evaluating and Strengthening Masonry Constructions

Examen, évaluation et consolidation de constructions en maçonnerie

Untersuchung, Beurteilung und Sanierung von Mauerwerkskonstruktionen

Andreas KOLBITSCH
Dr. techn.
ÖGEB
Vienna, Austria

Born in 1956, Andreas Kolbitsch received his diploma as civil engineer in 1979 and his Dr. techn. degree in 1981 from the Technical University of Vienna. Until 1986 he worked at this University (Institute for building and industrial construction) as assistant and since then as lecturer. Since 1987 he is secretary of ÖGEB (Austrian Society for Maintenance of Buildings).

SUMMARY

Methods for examining, evaluating and strengthening masonry constructions are investigated in various research projects by the österreichische Gesellschaft zur Erhaltung von Bauten (Austrian Society for the Preservation of Buildings). The pratical results of these investigations are presented in this paper.

RÉSUMÉ

Les méthodes d'analyse, d'évaluation et d'assainissement des maçonneries font l'objet de différents projets de recherches menés par la Société autrichienne pour la conservation de bâtiments (österreichische Gesellschaft zur Erhaltung von Bauten). Les résultats pratiques de ces analyses sont exposés.

ZUSAMMENFASSUNG

Methoden zur Untersuchung, Beurteilung und Sanierung von Mauerwerkskonstruktionen werden im Rahmen der österreichischen Gesellschaft zur Erhaltung von Bauten in einigen Forschungsschwerpunkten behandelt. Im vorliegenden Beitrag werden die praktischen Ergebnisse dieser Untersuchungen präsentiert.

1. BASIC PROBLEMS

In the course of several investigations within the scope of the "Österreichische Gesellschaft zur Erhaltung von Bauten" (Austrian Society for Restoration of Buildings) it turned out that evaluating and restoring of masonry constructions has a cosiderable impact on the sanitizing costs for the whole building.

Among several others the following main aspects have to be considered:

- Analyses and repair of damages caused by humidity
- Evaluation of the actual loadbearing capacity
- Measures for consolidation and reinforcement

Several working groups are dealing with these problems. The results of their investigations are presented within this summary.

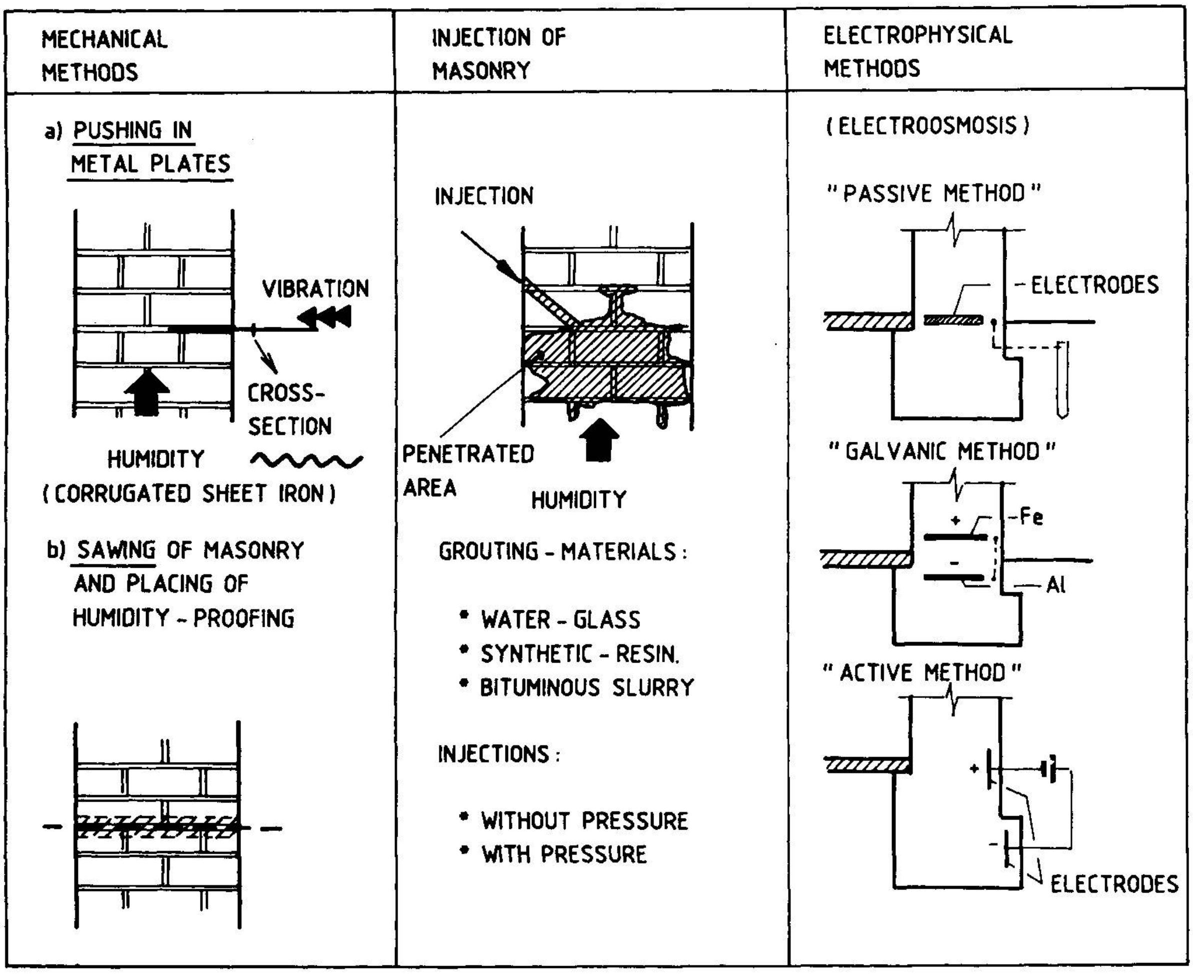


Fig.1 Measures against humidity caused damages

2. DAMAGES CAUSED BY HUMIDITY

Although the influence of humidity on the loadbearing capacity of masonry structures can be neglected in most cases, it turned out that repairing these damages causes a considerable part of overall costs of building consolidation.

In many cases the ground floor of masonry buildings cannot be used for apartments or offices because of damages caused by humidity.

Since the beginning of this century many methods have been invented to insert humidity proofings or water stoppers either chemically or electro- physically. In practical use only a few of them have shown acceptable results.

Within the last 10 years most of these measures have been improved. (Mainly chemical and electro-osmotic methods). Fig. 1 gives a principal outline on these methods. It has to be pointed out that mechanical methods are state of the art, whereas the other methods are still subject to further investigations dealing primarily with the lifecycle- costs of these measures inseparably associated with the durability of the components.

In most cases economic reasons are essential in selecting a particular method.

3. CONSIDERATION OF THE ACTUAL LOADBEARING CAPACITY

At present the characteristic values of actual compressive strength of masonry can be achieved in three different ways, as shown in Fig.2.

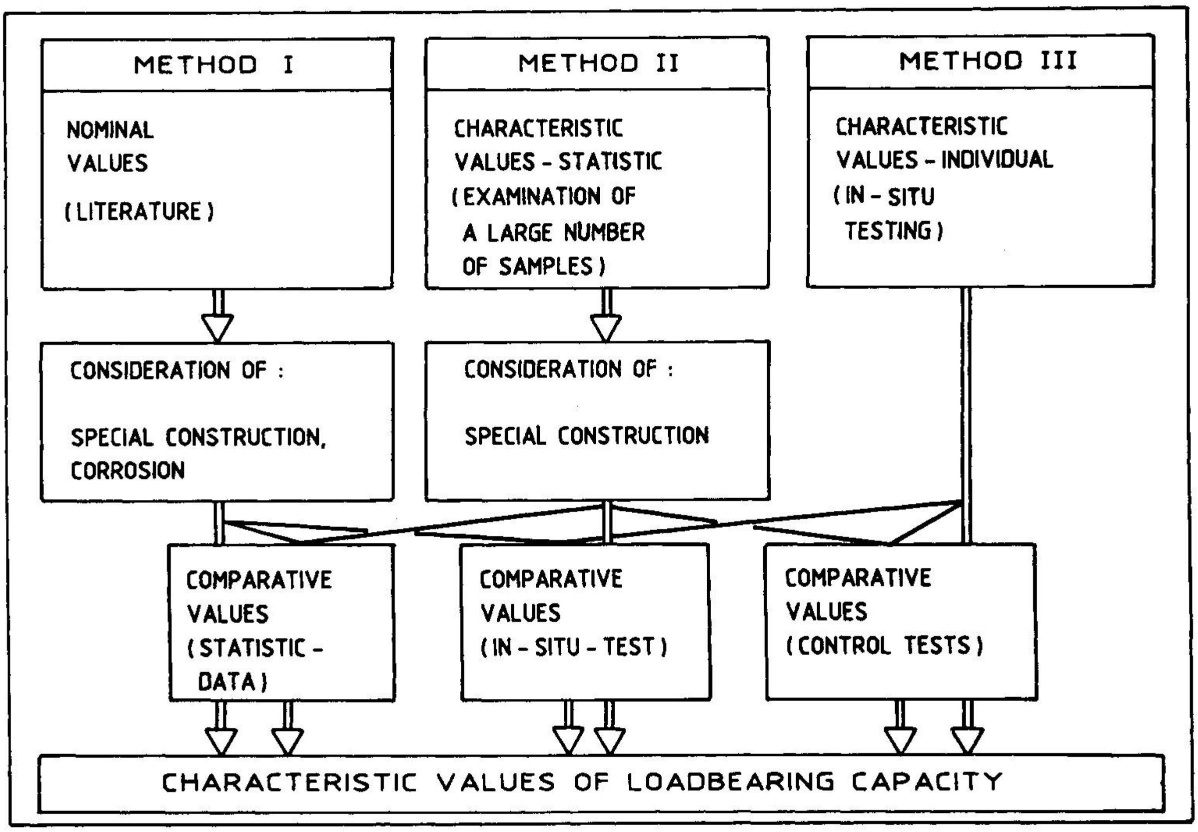


Fig.2 Consideration of actual loadbearing capacity

In any case it is necessary to examine the mechanical uniformity of masonry structures over large areas by non-destructive methods (e.g. Schmidt-Hammer). At the same time random samples should be taken in order to gain secured values for compressive strength.

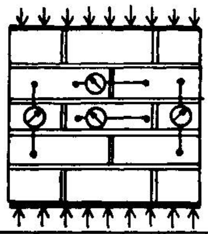
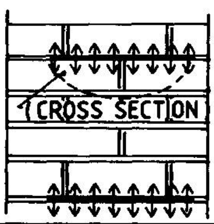
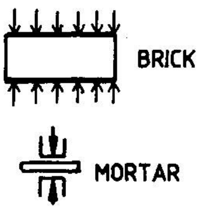
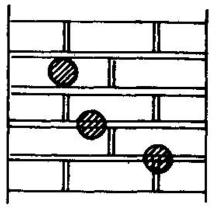
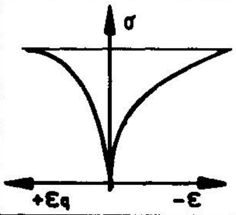
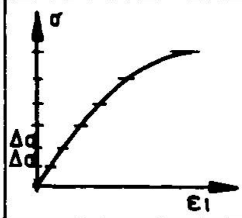
TESTING METHOD				
NAME	(RILEM) WALLETTE TESTING	INSERTING FLAT JACKS	TESTING OF COMPONENTS	TESTING OF DRILLING CORES
RESULTS, (EVALUATION)			SEE FIG. 4	SEE FIG. 5
LIMITS	TAKING OF SAMPLE	EVALUATING THE INFLUENCE OF MASONRY BOND	TAKING OF MORTAR SAMPLES	CONVERSION OF RESULTS TO COMPRESSIVE STRENGTH

Fig.3 Testing methods (loadbearing capacity)

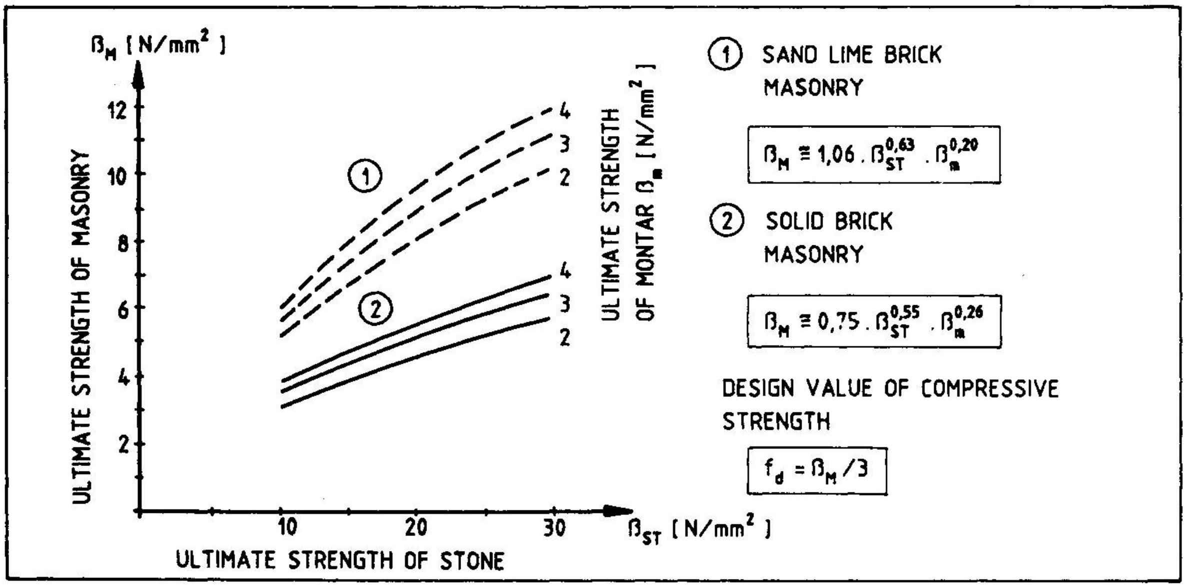


Fig.4 Testing of components - superposition formulas

The testing methods are shown in Fig. 3. Taking and Testing of wallettes (specimen of several layers of brick masonry) has considerable drawbacks connected with extraction and transport of the specimen to a laboratory. (The wallettes should be at least five layers high, so that the results can be compared to those of RILEM-test, on which most national standards are based.)

The other methods are subject to recent investigations. Testing of components (brick and mortar) - as shown in Fig. 3 - is based on the knowledge of superposition formulas as pointed out in Fig. 4.

Testing of drilling cores was first researched by Berger [2]. At present the correlation of the shown test methods is analysed in various research projects by ÖGEB.

Fig. 4 evaluates the superposition formula for pure sand lime brick or brick masonry and mortars with ultimate strengths of 2, 3 and 4 N/mm^2. Using the proposed formulas superposition curves have been drawn.
The design value of compressive strength of masonry is estimated as 1/3 of ultimate strength.

Fig. 5 shows the prininciples for evaluating compressive strength of pure brick constructions by testing drilling cores (diameter 5 to 6cm).In some cases this method poses problems during the extraction of joint drilling cores. After superposition of the shear stress and transverse stress for pure brick and joint drilling cores the results have to be converted to the compressive strength of the masonry. In order to find acceptable conversion factors, several tests are recently investigated.

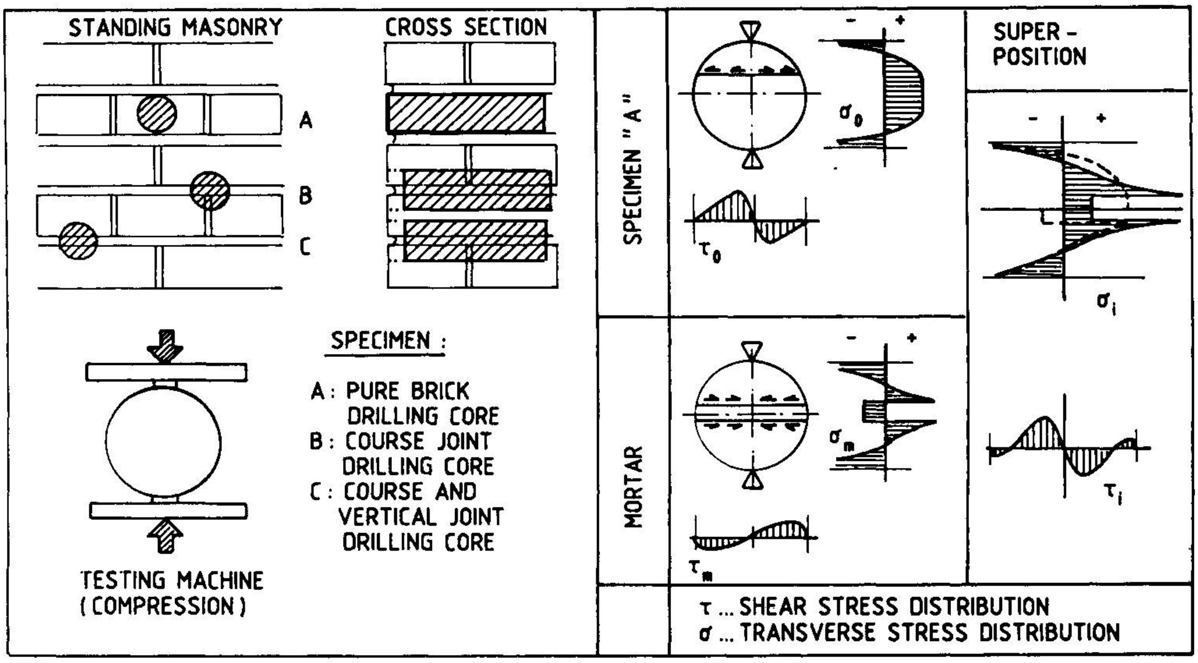


Fig.5 Testing drilling cores

4. STRENGTHENING OF MASONRY

Techniques of strengthening existing masonry structures are investigated within a special working committee. Five methods have been analysed so far. (See Fig. 6)

Each measure has both advantages and drawbacks in technical as well as in economic respect. Some of these are pointed out in Fig. 6.

In most cases the first two methods are put into practice. Reinforcing or prestressing local areas is used for historical buildings, where economic aspects play a minor role.

Evaluating shrinkage and creep of old masonry structures turned out to be the main problem in the application of these methods. Another problem is connected with the protection of reinforcing rods and prestressing cables against corrosion [5].

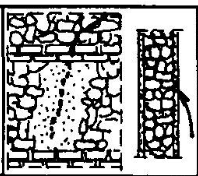
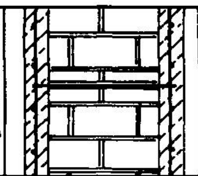
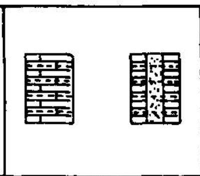
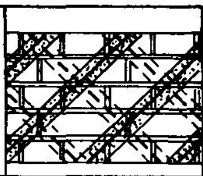
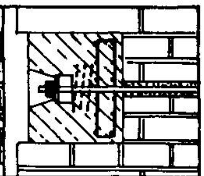
STRENGTHENING METHOD					
NAME	INJECTION	JACKETING (SHOTCRETE)	INSERTING REINFORCEMENT RODS	REINFORCING	PRESTRESSING
FIELD OF APPLICATION	LOCAL FAILURE	INCREASING SHEAR RESISTANCE, STRENGTH	SANDWICH MASONRY	LOCAL STRENGTHENING	REHABILITATION AFTER DEFORMATIONS
LIMITS	AREA OF APPLICATION	PLASTERING OF SURFACE	SANDWICH MASONRY	ACCESSIBILITY OF DRILLING LOCATION	CREEP AND SHRINKAGE
NECESSARY INVESTIGATIONS	CHARACTERISTIC VALUES OF MATERIALS (MORTAR, BRICK), GEOMETRICAL PARAMETERS CHEMICAL ANALYSIS OF MATERIALS				

Fig.6 Techniques of strengthening existing masonry structures

REFERENCES

1. WENZEL, F., Verbesserung von Mauerwerk durch Zementinjektion bzw. durch Vernadelung und Zementinjektion. Forschungsarbeit, Institut für Tragkonstruktionen der Universität Karlsruhe, 1981.

2. BERGER, F., Zur nachträglichen Bestimmung der Tragfähigkeit von zentrisch gedrücktem Ziegelmauerwerk. Erhalten historisch bedeutsamer Bauwerke, Sonderforschungsbereich 315, Universität Karlsruhe, Jahrbuch 1986, Verlag Ernst u. Sohn, Berlin, 1987.

3. TASSIOS, T.P., Physial and Mathematical Models for Re-design of Damaged Structures. In: IVBH Reports Volume 45, IVBH ETH-Hönggerberg, Zürich 1983.

4. PAUSER, A., et al., Probleme der Tragkonstruktion, Bauphysik, Technologie und Verfahrenstechnik bei der Erneuerung von Altbauten. Schriftenreihe der FGW, Heft 103, Wien, 1986.

5. WENZEL, F., MAUS, H., Nachuntersuchungen an ingenieurmäßig sanierten Mauerwerksbauten. Erhalten historisch bedeutsamer Bauwerke, Sonderforschungsbereich 315, Universität Karlsruhe, Jahrbuch 1986, Verlag Ernst u. Sohn, Berlin, 1987.

6. KOLBITSCH, A., Beton in der Althaussanierung. In: Zement und Beton, Heft 1/1988.