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Life Expectancy Studies of Reinforced Concrete Using Microcomputer

Étude de la durée de vie du béton armé par simulation

Studie zur Lebenserwartung von Stahlbeton mit Hilfe von Simulationen

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SUMMARY

The paper describes how a systematic stochastic life-expectancy and financial analysis can be performed by exploiting the combination of microcomputers and Monte Carlo simulation. The use of the program "Venturer" developed by the author, is illustrated with an example of a footbridge.

RÉSUMÉ

L'article décrit comment les analyses systématiques et aléatoires, de durée de vie et de risque financier, peuvent s'effectuer à l'aide de micro-ordinateurs en utilisant la simulation de Monte Carlo. L'utilisation du programme "Venturer", qui a été développé par l'auteur, est illustrée par un exemple de passerelle.

ZUSAMMENFASSUNG

Der Beitrag beschreibt wie eine systematische stochastische Lebenserwartungsberechnung unter Einbezug der Kosten mit Hilfe einer Monte Carlo Simulation auf Mikrocomputern durchgeführt werden kann. Die Anwendung des vom Autor entwickelten Programmes "Venturer" wird am Beispiel einer Fussgängerbrücke erläutert.

1. INTRODUCTION

A designer usually has a target life, referred to as a design life, for which s/he will attempt to design the structure. The actual service life, however, will be shorter or longer than the design life. It is, therefore, necessary to examine the uncertainties that influence the service life if rational decisions for economical design are to be made. The economic analysis should not only acknowledge the engineering uncertainties (i.e. physical, statistical and model) but must also take into account the financial risks caused by the uncertainties in the economic environment. The life-cycle costing exercise must include estimates of construction cost, salvage value (if any), design life as well as maintenance/repair costs over the life of the structure. Deterministic life-cycle (i.e. single point) estimates produced to compare and evaluate alternative designs can lead to bad decisions. These estimates, at best, aspire to produce likely values of life and cost. Figure 1 shows that if only likely values of the objective function (e.g. life) of the alternative designs A and B (on scale OS) are available to a designer then B will be chosen. On the other hand availability of the full probability distribution not only provides insight into the nature of the designs but also throws a very different light on their relative merits, and may lead to the alternative A being chosen as a safer or a low risk design. There is a rapidly growing awareness in the professions of the need for taking a view that reflects the stochastic nature of the problems that the designers are called upon to model and analyse.

2. DESIRABLE FEATURES OF LIFE EXPECTANCY STUDIES

All quantitative studies have two important features. Firstly, there must be a mathematical model which expresses the relationship between the objective (e.g. life, cost) and the various variables that are recognised to be the controlling factors. For a model to be successful it should be reasonable, complete and adaptable. A reasonable model does not violate the basic logic of the process being modelled and provides plausible results. A 'complete' model contains all the important influencing factors (termed uncertainties) and their interrelationship. Generally a real-life system will have innumerable factors influencing the objective. Inclusion of all these will cause loss of manipulative flexibility. Unimportant factors have to be identified and neglected. An adaptable model can be easily enriched in light of new knowledge and insight that ensues from its use as well as in light of continuing research in materials and structures. The studies should reflect this and allow the designer to assess the effect of choice of a model on the objective. Secondly, a reliable probability distribution of the objective can result only from a set of carefully estimated probability distributions of the variables (termed uncertainty profiles). When these are based on empirical studies or historical data it is important to be aware of the changing circumstances and environment. Due weight should be given to expert opinion and motivational and cognitive biases should be avoided by using Delphi technique. An important point that needs to be emphasised is that the uncertainty profiles of the various variables do not all conform to some convenient shape but can range, for example, from a near normal for one variable to near exponential shape for another within a model.

3. ANALYTICAL TYPE OF STUDY

This type gives stylised probability distribution of the objective function. It is obtained analytically from the uncertainty profiles of the controlling factors which have to be expressed in stylised forms. The mean and standard deviation of the objective are obtained from those of the variables and from the coefficients of correlation between them. This involves the use of simplifying assumptions about the variables and their correlation coefficients [1,2]. With many design problems of even modest complexity this approach may not even be possible [3] unless the model is simplified so much that it is no longer reliable; in that case the analyst gets a (so called) precise answer to a wrong question. Surely even an approximate answer to the right question is vastly superior. The author's "experiments" with senior engineers, postgraduates and undergraduates, have shown that they are ill-at-ease with this method and experience difficulties even with very simple problems.

4. MONTE CARLO SIMULATION TYPE OF STUDY

Unlike a field or a laboratory experiment, simulation can be conducted entirely on a computer by expressing the interactions and the dependencies among the various controlling factors in the form of a mathematical relationship. It has been used in a host of situations [4,5,6]. In Monte Carlo Simulation, we "construct" a large number of structures with our model to reflect the characteristics of our design. From these large number of results, the probability distribution (termed venture profile) of the objective is obtained. The input information does not have to be forced into some idealised mathematical shape, and full probability distribution of the objective is obtained. There is virtually no constraint as far as the complexity of the model is concerned. Thought processes associated with analysis and synthesis in this approach are positively more attractive to engineers. Sensitivities of the objective to the various factors as well as to the various mathematical models can be easily studied. This helps to identify the factors which contribute most to the phenomenon under study and to assess the effects of errors in the estimation of the data and in development of the models. Efforts to improve the estimates and the design procedures can then be made in proportion to their relative importance. Additionally the results of these analyses are very valuable for drawing attention and allocating effort towards improving the construction, maintenance and use of structures.

5. MICROCOMPUTER SIMULATION

It is most relevant to note that in the coming decade, microcomputer power is expected to quadruple, at least. Memory size alone is likely to quadruple every three years for a constant price. Bell [7] predicts that "the power of todays Cray X-MP (four processors delivering a peak power of one billion floating point operations per second... and a main memory of one million 64-bit words) will be available in a workstation". This is roughly 60,000 times the power of a personal computer. Even today, one of the criticisms levelled against simulation is that it is computationally expensive. This view is out of date by a number of years. These costs will be very trivial in the near future. Other reservations that are often expressed are the time and the cost of programming. This is valid if every problem needs to be programmed separately. To overcome this and to encourage greater use of probabilistic modelling techniques a package has been developed [1] which is designed to (a) provide an aid to learning modelling and simulation techniques and (b) provide an applications program for general use in a host of situations. It has been used for synthesising and analysing a number of problems, such as cost risk analysis of a hydroelectric project, stability of dams, project appraisal and atomic bomb detonation effects [4]. The application program allows direct entries of the mathematical model and data interactively without any need to access the codes. These entries can be "enriched" and revised as and when necessary. Other features included are resimulation, filing and hard copy facilities. Facility for sensitivity analysis is of particular value. Perhaps the most important argument for using Monte Carlo Simulation is the fact that the exponents of the analytical approach resort to using this method to check the validity and the reliability of their analytical methods.

6. ILLUSTRATION

Figure 2 shows a section of the foot-bridge used as the example. For this illustration we shall consider the service life in terms of the deck (slab) only. In order to determine the service life of a structure it is necessary to identify the influencing factors, their actual effects and the respective failure mechanisms. This is called "Failure Mode and Effect Analysis" (FMEA). A check list of the factors, mechanisms and their effects for structures can be obtained from various studies [8]. Corrosion of reinforcement as a result of carbonation of concrete is considered to be the most significant failure mode. Here we shall consider failure mechanisms mainly due to this factor. The effects of the other factors are implicitly included in variables that define the climatic conditions.

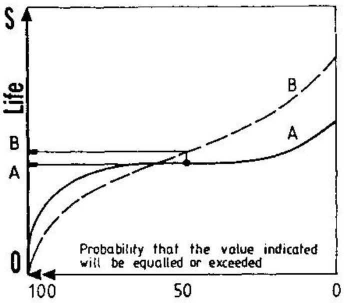

Fig.1 Comparison of Venture Profiles

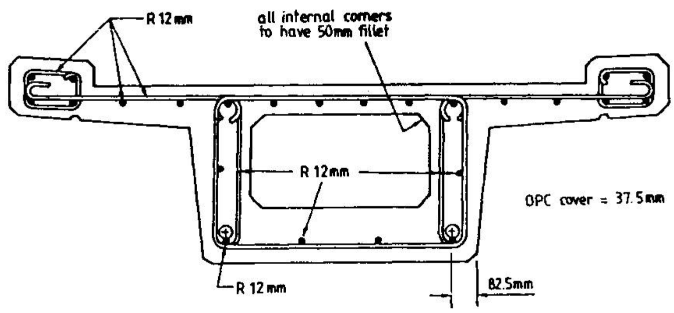

Fig.2 Foot-bridge: X-section

6.1 Model for Life

The question as to which model should be used is an area of wide controversy. The author does not intend to enter into this argument, and for his illustration he will employ a model provided by Siemes et al [8]. In their model the service life (L) is the sum of three elements; the time for carbonation, the time gap until the visibility of corrosion and the prolongation of the service life due to coating, i.e.

$$L = t_{cb} + t_{cr} + t_{ct}$$

Enrichment of this equation yields the following model:

$$L = \left[\frac{(c-d)}{R\,k} \times \frac{2.7}{46w-17.6} \right]^2 + \frac{0.08c}{d_i\,V_c} + \left[\frac{(c-d)S}{180 f_o} \times \frac{(T/T_c)\ln f_o}{1-e\hat{\ }-[(T/T_c)\ln f_o} \right]$$

where t_{cb} = carbonation time; t_{cr} = time to visible corrosion
t_{ct} = time extension due to coating; w = water/cement ratio
R = cement type factor; k = climate type factor
c = concrete cover; d = carbonated depth (mm)
d_i = bar diameter (mm); V_c = rusting rate (mm/year)
T = maintenance period; T_c = coating durability parameter
f_o = damage of coating (mm/year) S = coating thickness (mm)

For economy of space the last term is discarded with the assumption that coating will not be employed. Our model now has seven variables. The diameter of bars d_i is considered to be deterministic (12 mm). The uncertainty profiles of the other six are shown in Figure 3. Venture profile of the service life is shown in Figure 4. The sensitivity profile shows the relative importance of the factors. The lengths of the darker bars are scaled to give the variations in the average value of the objective as influenced by changes in the respective uncertainty profiles. The lighter bars give the ratio of the total change in the average value of the objectives to the total change made in the average value of the variable.

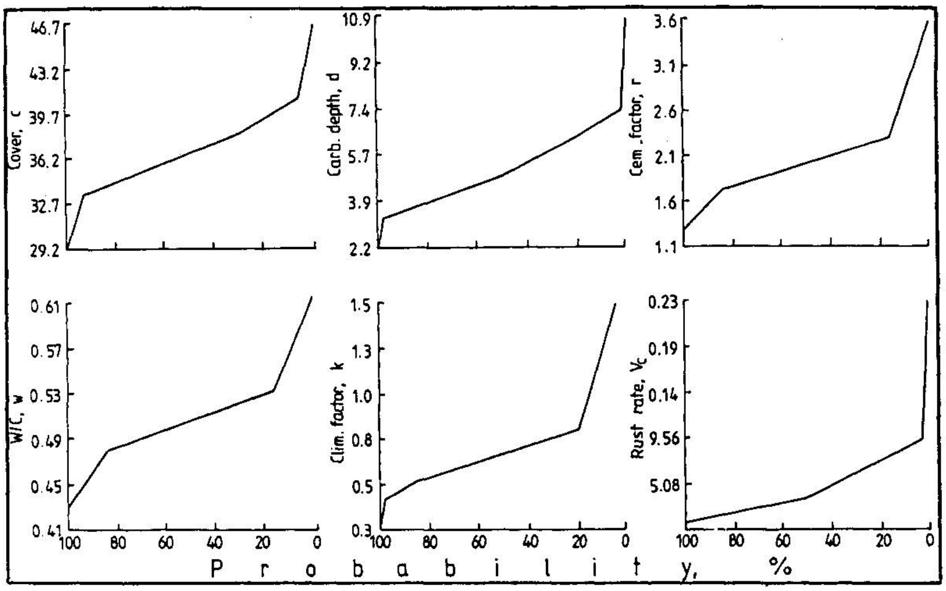

Fig.3 Uncertainty Profiles of variables in Life model

6.2 Model for Cost

There are, again, various ways in which the total cost of a structure can be modelled. For illustration let us use the following for estimating the Net Present Value (NPV) of the bridge.

NPV = Present worth factor x salvage value + present worth factor x (annual income - annual maintenance cost) - initial construction cost.

Enrichment of the model leads to the following; written in the form which can be entered directly through the keyboard (with the help of inbuilt logical checks and editing facilities provided in the Venturer) with meaningful variable names.

```
NPV = (1/(1 + (discourate/100))^life) * salvage)
    + ([(1 + (discourate/100)) ^life-1]/[(discourate/100)
    * ((1 + (discourate/100)) ^life)] * (anincome - anmaint))
      - constrcost
```

Venture profile for life (L) obtained earlier can now be entered as an uncertainty profile. Figure 5 shows the results with the assumption that the ranges of the other five variables are, in order in which they appear above, 7 to 12%; 14 to 147 years; £70,000 to 130,000; £100,000 to 220,000; £50,000 to 90,000 and £1,000,000 to 1,400,000. The results are obviously very valuable in design and economic studies such as evaluation of alternatives or establishing optimum maintenance policies. The sensitivity profile puts the relative importance of the engineering and economic factors in proper perspective.

7. CONCLUSIONS

Monte Carlo technique as implemented in the form of VENTURER, an interactive and tolerant educational and applications package, helps to overcome effectively the serious limitations of the analytical approaches to life-cycle studies. It encourages healthy scepticism towards assumptions in modelling and towards quantification of the influencing factors. It encourages and facilitates sensitivity analyses at the modelling and parameteric stages.

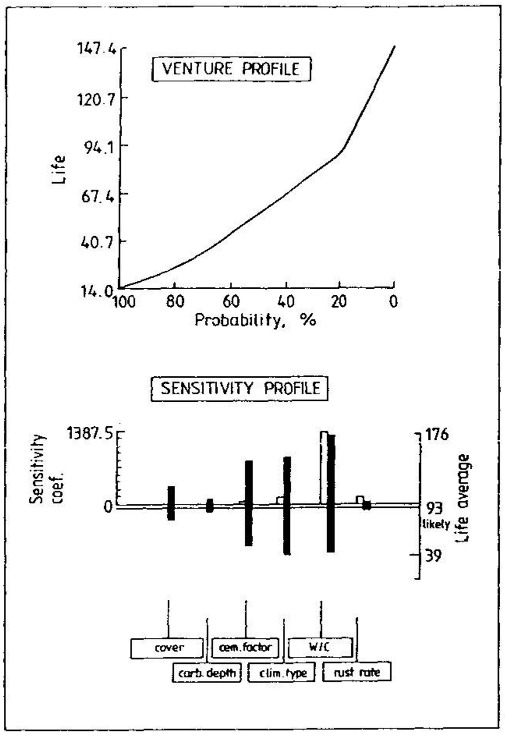

Fig.4 Life results

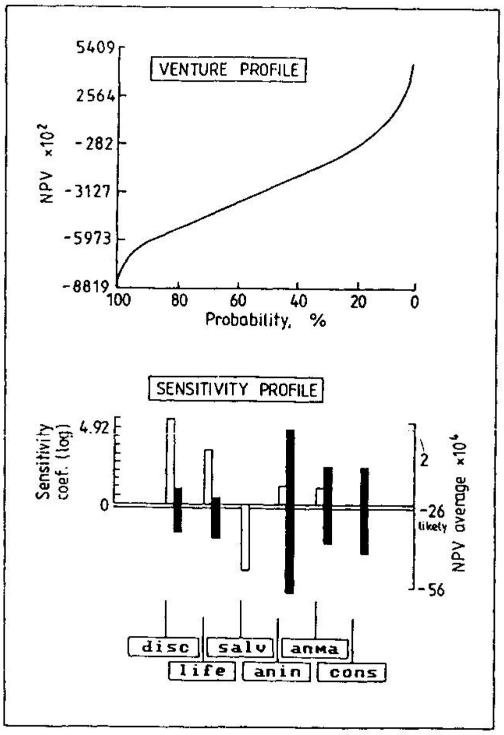

Fig.5 Cost results

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Remaining Service Life of Corroding Structures

Durée de vie restante de structures corrodées

Restlebensdauer von korrodierten Stahlbetonbauten

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SUMMARY

The prediction of the remaining service life of a corroding structure has been calculated up to now using empirical approaches and without taking into consideration the possible progressive structural damage that corrosion provokes. In a first attempt, values of several corrosion rates were implemented in Tuuti's model on service life and some kind of quantitative calculation could be presented. In the present paper, engineering considerations are introduced in the approach and the loss in load-carrying capacity is calculated for different corrosion rates assuming several simplifications.

RÉSUMÉ

La prédiction de la durée de vie restante des structures en train de se corroder, est faite habituellement par calculs approximatifs et sans considérer les différents niveaux de détérioration que la corrosion provoque. Dans la première tentative, des valeurs de la vitesse de corrosion ont été introduites dans le modèle de Tuuti sur la durée en service et un certain niveau de quantification a pu être atteint. Dans la présente communication, quelques considérations structurelles sont introduites et des exemples de pertes de résistance mécanique sont calculés en fonction de la vitesse de corrosion.

ZUSAMMENFASSUNG

Die bisherigen Verfahren zur Bestimmung der Restlebensdauer von korrodierten Stahlbetonbauten waren empirische Näherungen ohne Berücksichtigung der zunehmenden Schädigungen. In einem ersten Versuch wurden im Modell aus Tuuti die Daten verschiedener Korrosionsgeschwindigkeiten verwendet, um quantitative Abschätzungen zu erhalten. In der vorliegenden Arbeit werden zusätzliche ingenieurwissenschaftliche Betrachtungen zur Einschätzung der Tragwiderstandsverminderung vorgenommen. Die Restlebensdauer wird, unter der Annahme gewisser Vereinfachungen, für verschiedene charakteristische Korrosionsgeschwindigkeiten abgeschätzt.

INTRODUCTION

Service life prediction is complex matter in which both technical and economical consequences are involved. The need to study parameter has arisen from the unexpected premature deteriorations shown by reinforced concrete structures exposed to aggressive environments. The corrosion of reinforcements has resulted to be one of the most frequent causes of these premature failures.

Different proposals intended to calculate, either the life time of a new structure or the remaining life of a deteriorating one, may be found in the literature (1). Three of the authors of this contribution have also suggested in previous papers (2)(3) a methodology to calculate the remaining service of structures damaged by rebar corrosion. This methodology could be proposed due the large amount of corrosion intensity values, i_{corr}, which were collected by the authors along 20 years of experiments. These i_{corr} values were determined from Polarization Resistance results, Rp, measured in specimens prepared in the laboratory and on-site in real structures. In this paper a new advance in this line of research is offered, which considers the conversion of corrosion rate values in loss of load-carrying capacity terms. Some simple examples for columns and beams are calculated.

CALCULATION OF THE REMAINING SERVICE LIFE OF A CORRODING STRUCTURE

Three main points need to be considered to attempt to calculate the remaining service life: 1) The type of deterioration process involved; 2) The main parameter which controls the deterioration rate; 3) The unacceptable level of damage which makes the structure unsafe.

In the evoked previous paper (3)(4), for the particular case of corroded structures the following answers were given to these points:

1. Tuuti's model (4) was adopted as a deterioration model for a corroding structure. This simple model considers an initiation and a propagation period.
2. The loss of bar cross-section of the rebar was taken as rate-determining parameter. This loss in cross-section was determined from real i_{corr} values, whether they remain constant along the propagation period of (2) whether they change with the moisture content of the concrete (3).
3. The levels of deterioration suggested by the CEB in its Bulletin no. 162 were those taken into account.

Figure 1 is the result of jointly considering all these aspects. This figure allows the approximate calculation of the residual service life in terms of corrosion rates and of the loss in bar cross-section, assuming that this loss in diameter decreases linearly with the corrosion rate. The following relationship may be stablished from Figure 1.

$$\emptyset\,(t) = \emptyset_i - 0.023.\; i_{corr}.t$$

$\emptyset\,(t)$ = the rebar diameter at time t (mm)
$\emptyset_i$ = the initial diameter of the rebar (mm)
i_{corr} = the corrosion rate ($\mu A/cm^2$)
t = the time after the beginning of the propagation period (years)
0.023 = the conversion factor of $\mu A/cm^2$ into mm/year

However, the translation of these concepts into engineering terms is necessary, if the remaining load-carrying capacity and the safety of the structure are to be de - termined.

ENGINEERING CONSEQUENCES OF THE REBAR CORROSION

The main undesirable effects of the corrosion in the structure may be summarized as: a) a loss in the steel integrity: loss of cross-section and likely in the mecha nical properties; strenght and ductility; b) the splitting and spalling ot the cover with a loss in the concrete cross-section in the case of spalling; c) a loss in bond between concrete and steel in the case of cracks running paralled to the reinforcements and provided that the loss in steel section is high.

Very little attention has been paid in the literature to these effects (5)(6) and thus, the experimental data are scarce and usually obtained through and artificial acceleration of the corrosion process.

HYPOTHESES CONSIDERED IN THE PRESENT STUDY

The study of the load-carrying capacity loss will be approached at three different levels. A first one which consider the deterioration of a section in such a way that the strength loss against different action effects (bending moment, shear force, axial force, etc.) could be established. A second level which will introduce the deterioration model of an element, so that the loss in load-carrying capacity if isolated elements (for instance; simply supported beams) could be proposed. Finally a third level which will consider the whole structure and take into account the possible redistribution of the action effects, provided that it is allowed by the remaining materials ductility. For the purpose of the present study only the first level is going to be considered.

The assumption considered here may represent the case of corrosion in carbonated concrete where cracks are not produced in the cover because the concrete remains wet and therefore the oxydes may diffuse through the pores. They are:

- an homogeneous loss around the whole steel surface is supposed wheter i_{corr} remains constant or varies with ambient humidity. No pitting or localized corrosion is studied at this moment,
- no loss in bond is produced, which means that no parallel cracks were generated during the corrosion process and therefore the cover remains free from damages,
- no loss in steel mechanical properties is taken into account.

EXAMPLES AT THE CROSS-SECTION LEVEL

Ultimate bending moment (M_u)

The decrease in the ultimate bending moment has been studied at a cross-section 0.40 m deep and 0.25 m wide, with a tension reinforcement (4 Ø 14 mm or 2 Ø 20 mm).

Four corrosion rates have been considered (0.1, 1.0 and 100 $\mu A/cm^2$), kept constant along time, and it has been assumed that corrosion only affects the decrease in the diameter of the rebars, according to the equation given in the preceeding section.

In this way and bearing in mind the conventional hypotheses for reinforced concrete, the curves in figure 2 have been plotted, which show the decrease of the ultimate bending moment in terms of the time elapsed since the beginning of the reinforcement corrosion (propagation time). Thus, while corrosion rates of 0.1 and 1 $\mu A/cm^2$ result in a slight decrease of the safety factor along the 50 years of the service life assumed, rates of 10 and 100 $\mu A/cm^2$ result in the desappearance of the safety factor in lifetimes of 16 and 2 years respectively, when the sections have been reinforced with 14 mm diameter rebars.

In figure 2, it can also be observed that cross-sections reinforced with higher diameter bars are less sensitive to the damages produced. Thus, with two reinforcements with the same steel ratio but with different bar diameters (4 Ø 14 mm and 2 Ø 20 mm) and for the same corrosion rate (10 $\mu A/cm^2$), the loss of the safety factor is reached at different time lapses of 16 and 22 years, respectively, the corrosion damage being reached at an earlier time in the cross-section reinforced with the 14 mm diameter bar. The interest to use big-size rebars is thus shown in order to delay the damages if an adequate concrete cover is provided.

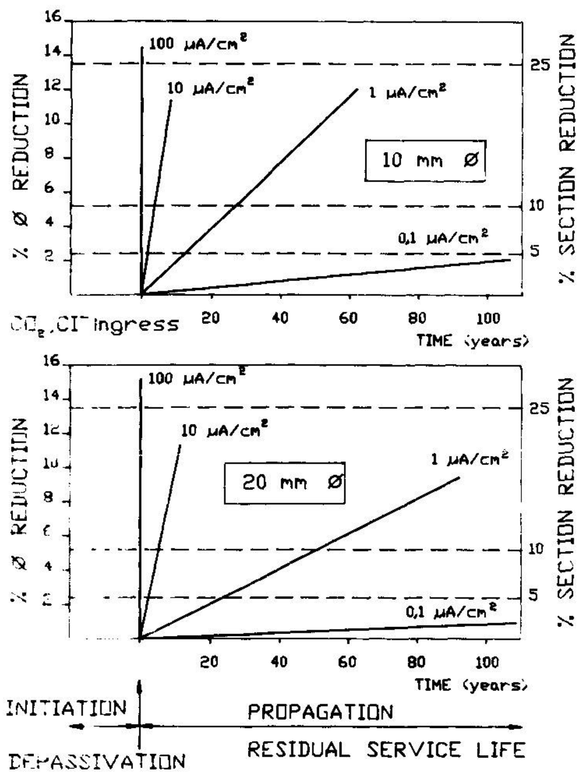

Fig. 1 - Rebar life time in function of its diameter and corrosion rate

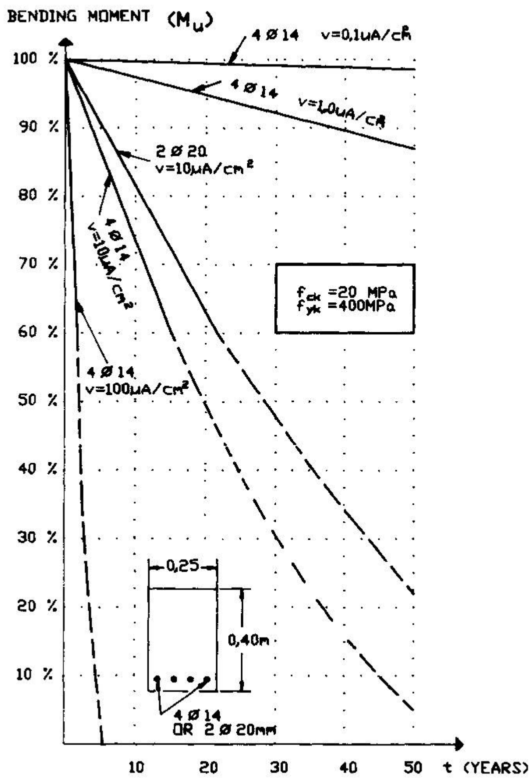

Fig. 2 - Loss in bending moment in function of the corrosion rate

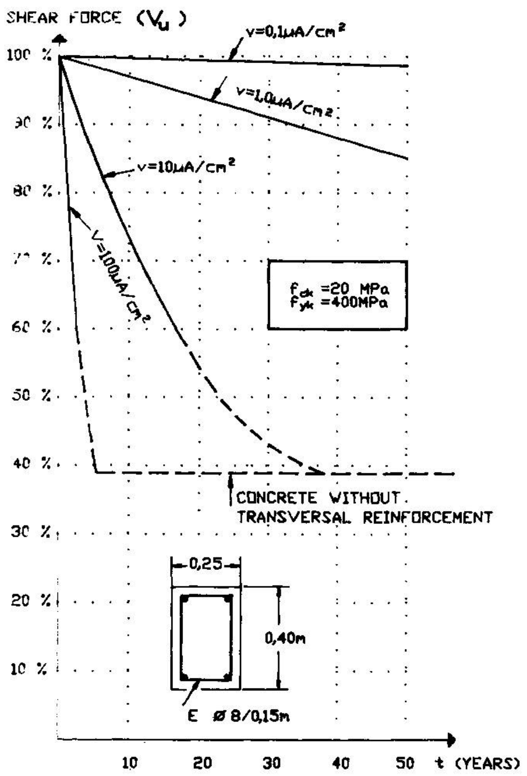

Fig. 3 - Loss in shear force in function of the corrosion rate

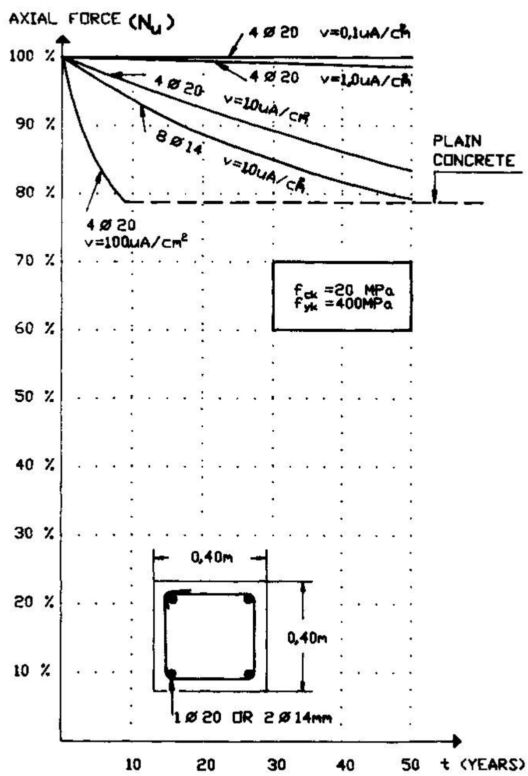

Fig. 4 - Loss in axial force in function of the corrosion rate

Ultimate shear force (V_u)

The decrease in the ultimate shear force of the same concrete cross-section reinforced with 8.0 mm diameter stirrups, located at 0.15 m distances, has also been investigated for different corrosion rates.

It is assumed that the deterioration of the longitudinal reinforcement is negligeable, what may be true, upon the reinforcement having a thicker cover, and that the shear force can be estimated by adding the shear carried by the concrete to the shear analysed by the truss analogy, which consists of the concrete and the longitudinal and transversal reinforcements.

The curves in figure 3 have thus been plotted which show again that corrosion rates of 0.1 and 1 $\mu A/cm^2$ scarcely impair that value of the shear force along the 50 years lifetime contemplated for the structure, while with corrosion rates of 10 and 100 $\mu A/cm^2$ the safety factor becomes zero along periods of 15 and 2 years respectively. At the latter two corrosion rates, the web reinforcement collapses along 4 and 35 years periods, and, from that time, the shear force is exclusively carried by the concrete itself and by the longitudinal reinforcement. This situation is somewhat theoretical, since, after these time periods and at these corrosion rates, the derioration of the longitudinal reinforcement must also be quite considerable, and therefore the shear force carried by this reinforcement must also be largely smaller than the shear shown in the horizontal lengths of the curves shown in figure 3.

Ultimate axial force (N_u)

Finally, the reduction in the bearing capacity of a 0.40 x 0.40 m cross-section axially loaded, reinforced with rebars at its four corners has also been studied for the same corrosion rates as indicated in the foregoing cases.

The curves plotted in figure 4 shown the evolution of the axial force along the time elapsed from the beginning of the reinforcement corrosion and have been estimated by adding up the axial force carried by the concrete to that carried by the reinforcement. There has not been taken into consideration the possible buckling of the longitudinal reinforcement, when neither the concrete cover nor the transversal reinforcement, also deteriorated can duly brace it.

At the cross-section reinforced with a 20 mm diameter rod at each corner, the damage caused is significative for corrosion rates of 10 $\mu A/cm^2$, so that, at 50 years life, most ot the axial force is virtually carried by concrete alone. For corrosion rates of about 100 $\mu A/cm^2$, the reinforcement disappears in its entire by before 10 years, so that the axial force is then carried by concrete only.

It is well known that a centrally loaded section is a theoretical situation, for these always appears certain eccentricities induced by the loads or by imperfections in the concrete cross-sections. In this connection, the possible irregular damaging of the reinforcement, may also result in a certain additional eccentricity which, combined with the previous eccentricities, results in a cross-section with a highly damaged reinforcement bearing with greater difficulting the theorical simple compressive stresses. Therefore, the horizontal lengths of curves in figure 4 give "optimistic" values, which will be reduced due to the higher sensitivity of plain concrete to the action of such "accidental eccentricities".

In figure 4 there can also be compared the different evolution of the axial force with the time elapsed, in cross-sections reinforced with the same steel ratio, but different rebar diameters (1∅ 20 mm or 2∅ 14 mm at each corner). It is shown again the interest of using greater diameters, taking in advance the precautionary measures mentioned above.

DISCUSSION

The estimation of the remaining service life of corroing structures has been, up to now, mainly based on empirical or qualitative models and on the subjetive experiencie of expertises. A quantitative method could not be found in the literature.

The methodology presented here relates the loss in load-carrying capacity with the loss in steel cross-section as a function of its corrosion rate. It is still a simplified method of approaching the prediction of the remaining service life and some simplifications had to be assumed due the lack of experimental information, but it offers a semiquantitative methodology to make predictions concerning the damage in the critical sections of the structure. Provided that, the corrosion intensity of a corroding structure and its age are known, an estimation of the time required to reach a critical loss, can be made.

For a more accurate estimation, several questions should be experiments. Aspects such as bond, steel mechanical properties, the presence of cracks, will have to be elucidated considering different corrosion or damage levels. Also, not only sections, but simple elements (beams and columns) and, finally, the whole structure will have to be investigated.

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Remaining Fatigue Life of Old Steel Bridges

Capacité de résistance à la fatigue des vieux ponts en acier

Beurteilung der Restlebensdauer alter Stahlbrücken

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Klaus Brandes, received his graduate as well as his doctorate degrees at the Technical University of Berlin. After working for some years in the field of streel structures, he joined the Federal Institute of Materials Research and Testing (BAM) in 1968 where he is concerned with various fields of structural engineering.

SUMMARY

Because of the large number of existing old steel bridges, the evaluation of the remaining fatigue life of these bridges is a very important economic problem and also an interesting one from the engineering point of view. For investigating old steel bridges, crack growth tests have been added to the well established types of investigation. To verify the established procedure of investigation and rating traffic load will be simulated on dismantled bridges in the laboratory.

RÉSUMÉ

Etant donné le grand nombre de vieux ponts en acier existants, l'évaluation de leur capacité de résistance à la fatigue est un problème économique de grande importance et un problème de grand intérêt pour l'ingénieur. Aux méthodes classiques d'inspection des vieux ponts en acier, il est possible d'ajouter l'étude du développement de la fissuration. Afin de pouvoir vérifier et étalonner la procédure d'inspection établie, les cas de charge de trafic ont été simulés en laboratoire sur des ponts démantelés.

ZUSAMMENFASSUNG

Die Ermittlung der verbleibenden Nutzungsdauer alter Stahlbrücken stellt wegen der grossen Anzahl dieser Brücken ein beachtliches wirtschaftliches Problem dar und ist zugleich eine interessante ingenieurwissenschaftliche Aufgabe. Bei Untersuchungen alter Stahlbrücken wurden die üblichen Untersuchungen um Rissfortschrittsversuche an standardisierten Proben erweitert. Zur Bestätigung der Beurteilungsverfahren werden alte ausgebaute Brücken simulierter Verkehrsbelastung bis zum Auftreten von Rissen unterworfen.

1. INTRODUCTION

In Germany as well as in other countries, there are many old riveted steel bridges which are approaching their design life after about hundred years of service. The replacement of all these bridges far exceed the available financial resources. However, even if the funds existed, replacement would be the least acceptable option in several cases because many of the bridges are historic structures [1]-[4].
Being concerned with the rating of old steel bridges, we added a further component to the procedure of investigation. We performed crack growth tests on samples of the material of some high stressed structural elements of the bridge shown in fig. 1, thus determining data to define the fatigue behaviour of the material [5].

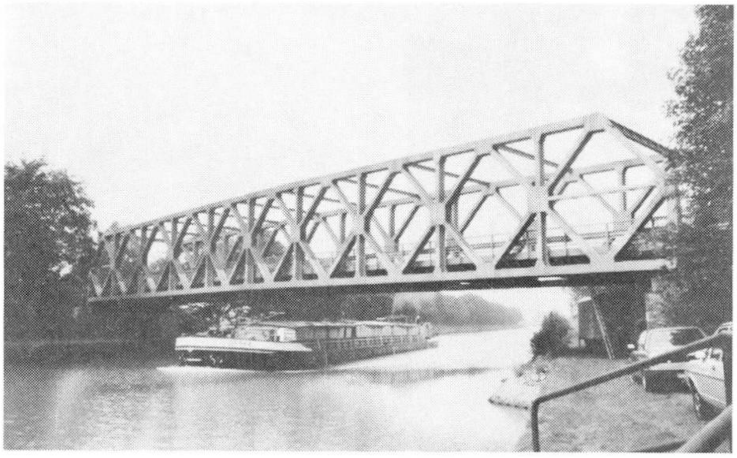


Figure 1.
Riveted highway bridge of 59 m span in north-west Germany

However, faced with the problem of rating old bridges, the need of further research is recognized in order to calibrate the results of the rating procedure with experience. This calibration can only be done by simulating traffic load on old removed bridges the stresses in which are well-known from measurements before removing. Research is under way in BAM on bridges one of them is shown in Fig. 2.

2. BRIDGES UNDER INVESTIGATION

Two bridges are under investigation (fig. 1, fig. 2). In the following, a brief description about the investigation and their results are presented.

2.1 Truss Bridge Crossing a Canal in North-West Germany

The load bearing elements of the bridge (fig. 1) that crosses the Dortmund-Ems-Kanal near Osnabrück, are outlined in fig. 3. It was built in 1953, the rolled girders were produced in the thirties. During an inspection, cracks have been detected at some of the cross girders near the connection to the principal load bearing elements, trusses of 59 m span (fig. 4). The investigation of the bridge included strain measurements at longitudinal and cross girders under static load and under traffic flow and tests on the

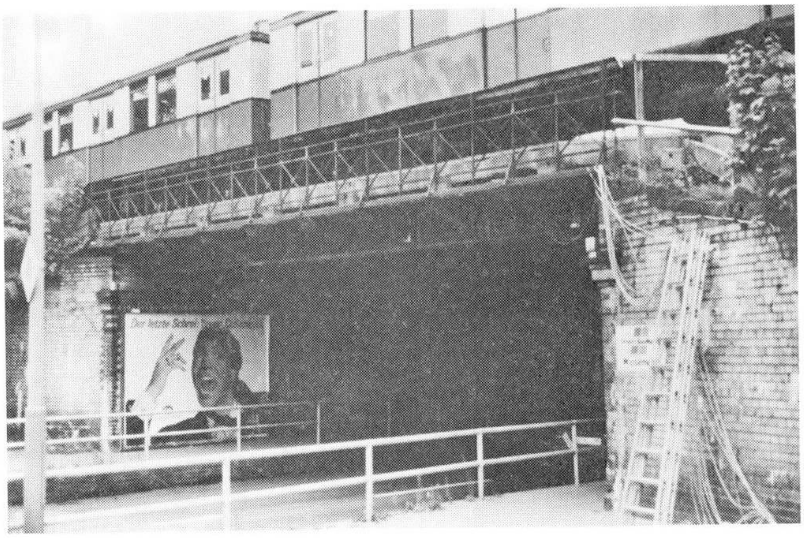


Figure 2.
Girder Bridge of
Berlin Metropol-
itan, replaced in
1988

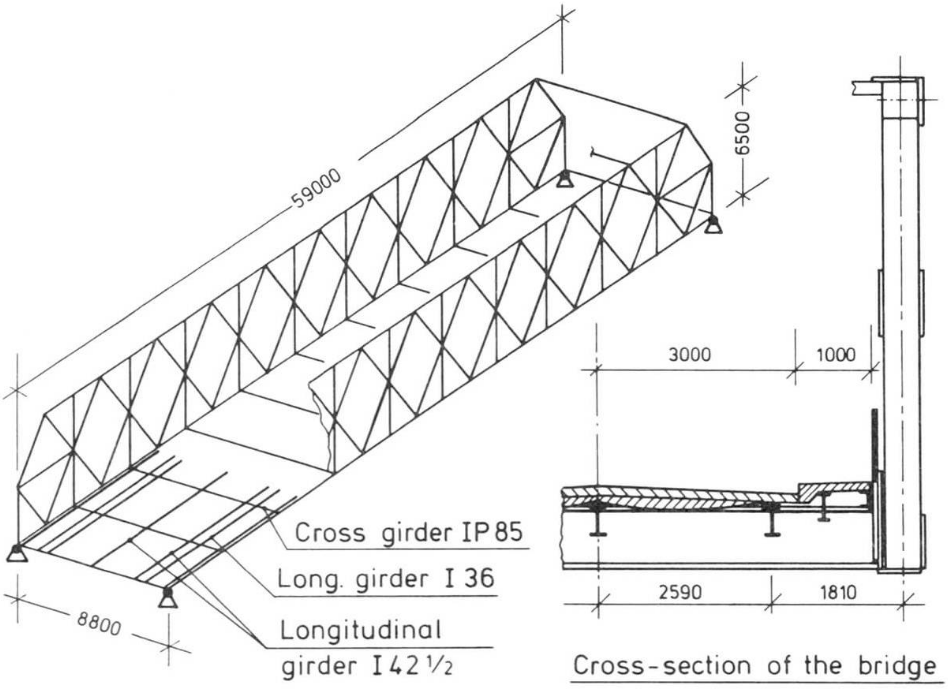


Figure 3: Load bearing elements of the bridge of fig. 1

material of the structural elements in question. The measurements under defined static load by a two-axle truck of about 20 t weight showed that the calculated stresses at the points of maximum stressing are considerably higher than the measured ones. From the measurements under traffic we obtained that only in very few cases of passing of heavy trucks, stresses exceeded the cut-off limits of the standardized fatigue curves [6] (fig. 5). In addition to the standardized material investigation, we performed crack growth tests on CT-specimens as recommended in ASTM E-647-83. The results in terms of the representing Paris equation (fig. 6)

$$\frac{da}{dN} = C\,\Delta K^n \qquad (1)$$

a crack length
N number of loading cycles
ΔK cyclic stress intensity factor
C;n material parameters of crack growth

confirmed that the material could be regarded as mild steel concerning the fatigue behaviour.

Figure 4.
Structural detail at the connection of the cross girders to the main truss girder and the location of cracks

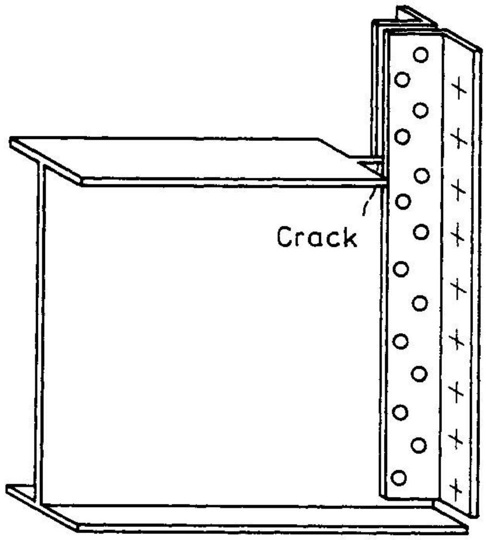
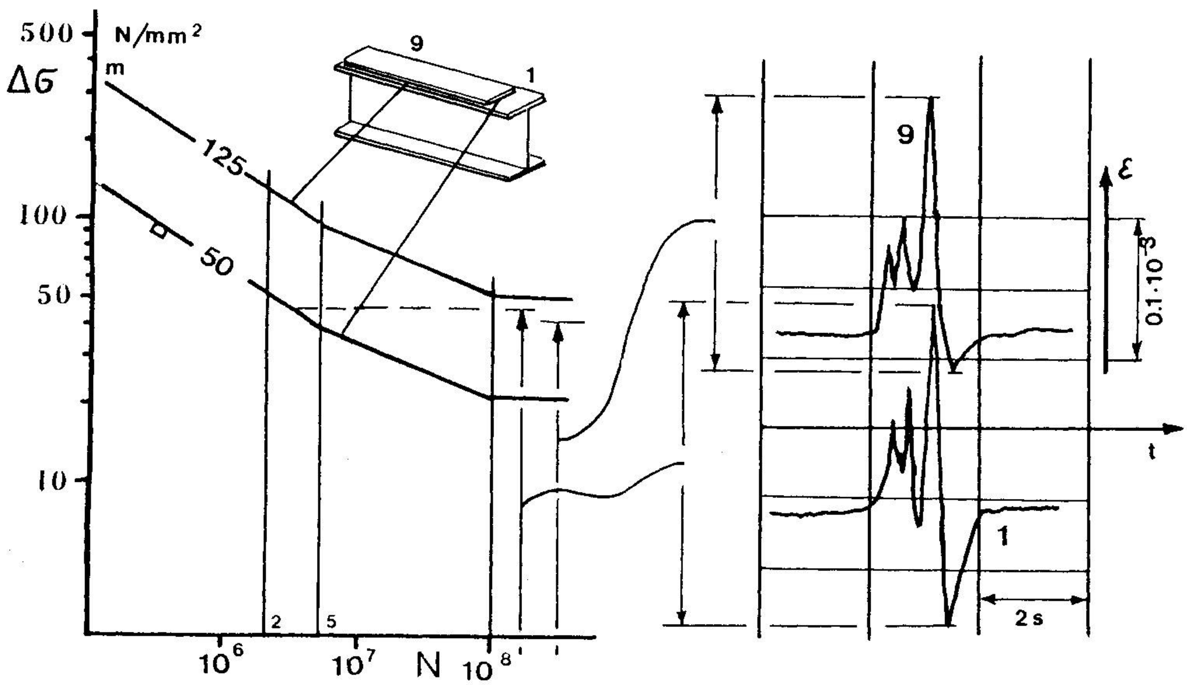


Figure 5 (below).
Fatigue strength curves (categories 125 and 50) and maximum measured stress cycle at two measuring points at the central longitudinal girder during passing of a heavy vehicle.



The crack growth tests offered the possibility to assess the amount of growing of a crack of a certain length which was not detected during an inspection within the time interval up to the next inspection (3 years). From the measurements, it turned out that the cracks (fig. 4) were caused by residual forces.

2.2 Riveted Girder Bridge of Berlin Metropolitan

The single-track bridge (fig. 2) with a span of about 12 m was built in 1896. It had to be replaced because the road it crosses had to be widened.
Before the bridge was removed, we performed strain measurements at several points of the structure, at elements of the main girders and of cross girders. The stresses during the hours of measurement did not reach the cut-off limits of the standardized European Fatigue Strength Curves[6]. However, we do not know accurately which type of traffic passed the bridge during the nearly 100 years of the bridge's life in an eventful history.

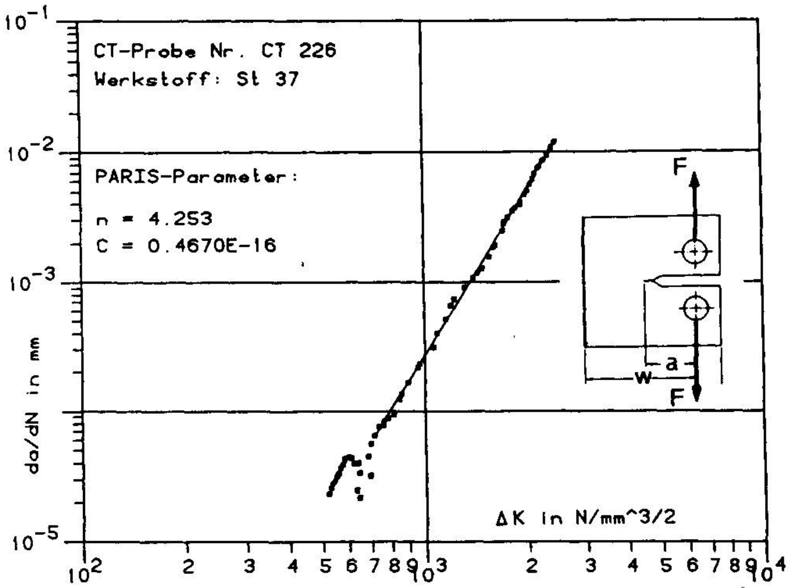


Figure 6: Diagram of crack growth rate da/dN versus cyclic stress intensity ΔK as elaborated for a compact tension (CCT) specimen (ASTM E 647-83)

It is intended to simulate traffic load on this bridge and another one similar to it in the laboratory to confirm the assessment of remaining fatigue life by the evaluation procedure that is common practice.

3. PROBLEMS OF APPLICATION OF CRACK GROWTH INVESTIGATION

The application of fracture mechanics in the evaluation of remaining fatigue life implies the modelling of cracks in structural elements which can probably occur, fig. 7. The crack growth is only governed by the stress intensity at the crack's tip as expressed by the Paris equation (1). It is an advantage of the concept of fracture mechanics that it leads to a deeper insight into the problem of survival of structural members under cyclic loading than only the application of fatigue strength curves after calculation of cummulative damage by collectives of stresses. Fracture mechanics procedures are recommended when cracks have been observed, because the shape and the dimensions of the cracks are known as well as the surrounding stress fields.

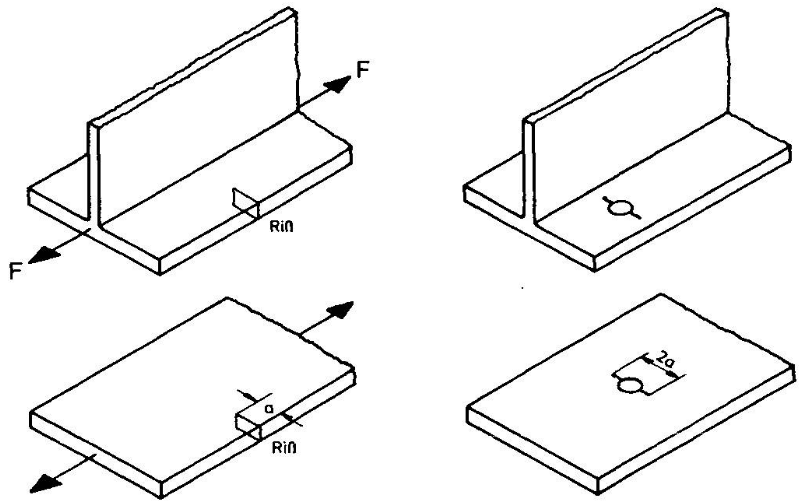


Figure 7:
Crack pattern for evaluating stress intensity factors and crack propagation

4. CONCLUSION

The evaluation of the remaining fatigue life of old riveted steel bridges is a very important task of structural engineering and comprises several different investigations. Faced with the problem to give an estimation of the remaining fatigue life of a certain bridge, which had cracks in a few structural elements, we extended the commonly used evaluation procedure by crack growth tests. The results of these tests made it possible to identify the material in terms of fatigue strength and gave the fundament to determine the propagation of postulated or observed cracks within the inspection period of time.

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Stress Corrosion of Cement Mortars in Ammoniumsulfate Solution

Corrosion sous contrainte des mortiers de ciment dans des solutions de sulfate d'ammonium

Spannungskorrosion von Zementmörteln in Ammoniumsulfatlösungen

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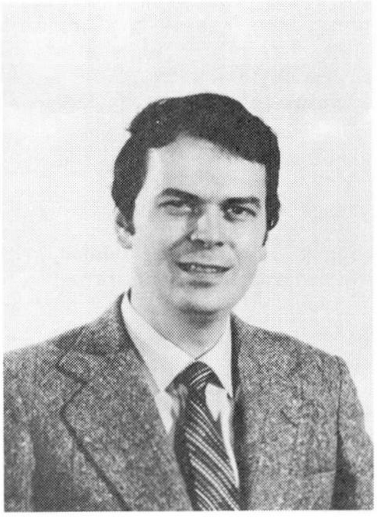
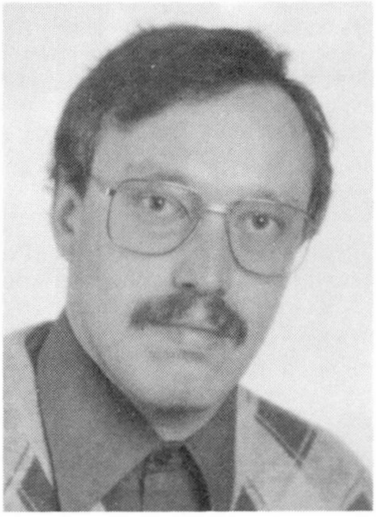
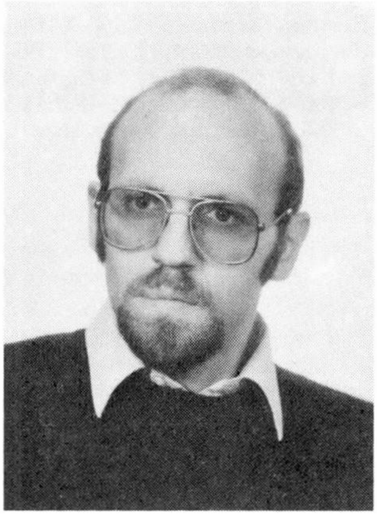


Prof. Dr.-Ing. habil. Ulrich Schneider, born 1942, received his master's degree in mechanical engineering at the TU Braunschweig in 1969. He received his doctorate in civil engineering 1973 and works as a university lecturer since 1979. In 1981 he was awarded the chair in building material science in the dep. of civil eng. at the Univ. of Kassel. His research work comprises mainly the durability and corrosion of non-metallic inorganic material and thermal properties of building structures and materials.



Dr.-Ing. Erich Nägele, born 1953, received his master degree in chemistry at the University of Karlsruhe, FRG in 1978. He received his doctorate in civil engineering 1981. Dr. Nägele works now in the civil engineering department at the University of Kassel in the field of stress corrosion of non-metallic inorganic materials and on the electric properties of materials.



Norbert Dujardin, born 1950, received his civil engineering master degree at the University of Aachen. For 4 years he was involved in the design and calculation of pipe supports for nuclear reactors. For the last 3 years he has been working in the civil engineering department at the University of Kassel. His research is devoted to the stress corrosion of non-metallic inorganic materials and fracture mechanic of concrete.

SUMMARY

The effects of type of cement, of water/cement-ratio and of surface coatings on the stress corrosion of cement mortars in 5%- ammonium sulfate solution are reported. The effects observed for different cements and water/cement-ratios in stress corrosion correspond to the effects observed in classical corrosion, i.e. mechanical stresses are not present. Coated specimens, however, show a distinct decline in strength when a mechanical stress acts simultaneously to a chemical stress and when the coatings are destroyed by cracks. Thus, the use of coatings has to be reconsidered, taking into account stress corrosion.

RÉSUMÉ

Ce travail examine les effets du type de ciment utilisé, du rapport eau/ciment et de la protection des surfaces vis-à-vis de la corrosion sous contrainte des mortiers de ciment dans les solutions de sulfate d'ammonium. Les effets du type de ciment et du rapport eau/ciment sont les mêmes que pour une corrosion classique, s'il n'y a pas d'actions mécaniques. Des échantillons montrent une diminution de résistance marquée lorsque des actions mécaniques agissent simultanément à une action chimique et lorsque les couches superficielles sont détruites par fissuration.

ZUSAMMENFASSUNG

In dieser Arbeit wird über den Einfluss der Zementart, des W/Z-Wertes und von Oberflächenbeschichtungen auf die Spannungskorrosion von Zementmörteln in 5%iger Ammoniumsulfatlösung berichtet. Die für die verschiedenen Zemente und W/Z-Werte beobachteten Effekte entsprechen denen bei normaler Korrosion, d.h. wenn zusätzliche mechanische Spannungen nicht vorhanden sind. Beschichtete Proben zeigen jedoch unter Spannungskorrosion einen starken Festigkeitsabfall, wenn die Beschichtung Fehler aufweist. Daher muss die Verwendung von Beschichtungen neu überdacht werden, wobei die Spannungskorrosion zu berücksichtigen ist.

1 INTRODUCTION

Chemical attack and mechanical properties of cementitious materials are usually being studied separately. However, in practice, chemical attacks are common to load bearing structures, therefore the simultanous action of chemical and mechanical stresses, known as "stress corrosion", has to be considered in any evaluation of the durability of cementitious materials. It has been shown that cementitious materials like many other building materials are also subjected to stress corrosion [1-5].

2 EXPERIMENTAL

2.1 Materials

The experiments were performed with mortar prisms 4x4x16 cm made according to the German standard DIN 1164. Three cements were used throughout the tests, a German Portland cement,PZ 35F (PC), a German blast furnace slag cement, HOZ 35L with a slag content of 50% by weight (BFC) and a German fly ash cement, FAZ 35L with a fly ash content of approximately 30% by weight (FAC) . The analysis of the cements is given elsewhere [2- 5]. The aggregate was quartzitic German standard sand.

2.2 Specimens

The specimens were made with the general composition aggregate : cement : water = 3:1:w/c. Hence the contents of cement and aggregate were kept constant in all experiments.

The specimens were made with notches 10mm depth, 45^{o} opening angle by pressing a suitable wedge into the fresh mix. A detailed overview of the tests is given in table 1. The specimens were cured for 28 days under water before immersed into the aggressive media.

2.3 Test Procedure and Test Program

The mortar prisms were immersed into 5%-ammonium sulfate solutions and loaded with load levels up to 50% of their initial flexural strength, which was determined prior to immersion. The loading device is shown in fig. 1. It is decribed in more detail in [2-5].

Unloaded specimens were stored in the same containers, so loaded and unloaded specimens were stored in the same solutions and under the same conditions. The load levels applied to the respective series may be taken from table 1, where the test program is summarized.

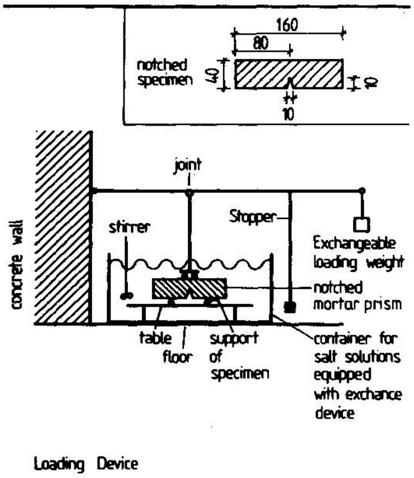


Fig.1: Loading Device for Stress Corrosion Tests

Table 1: Test Program

Series No.	Effect investigated	Type of Specimen	Load levels (%)
1	Type of Cement	W/C= 0,7; PZ, HOZ, FAZ	0 30
2	W/C-Ratio	PZ , HOZ W/C=0,55; 0,65;0,75	0 30
3	Surface Treatment and Coating	PZ , W/C=0,7 Sand blasted PUR-coated EP-coated	0

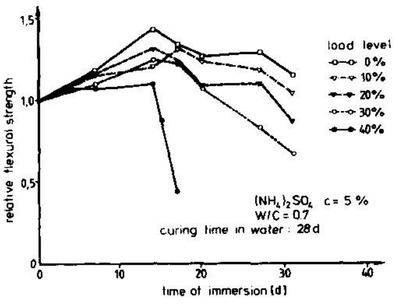
3. RESULTS AND DISCUSSION

Due to the limited space available not all results can be presented and discussed here. Thus only a brief survey on the most importnant features of the stress corrosion of cementitious materials is given and those topics most important for building practice are discussed briefly. Further details may be found in [2-5].

3.1 Stress Corrosion Phenomena in Aqueous Solutions

Fig.2, where the relative flexural strength is plotted versus the immersion time, shows some results of stress corrosion experiments from series 1 in 5%- ammonium sulfate solutions. Strength first increases due to chemical reactions like the formation of calciumsulfoaluminmates etc. and then decreases due to the formation of gypsum which causes sulfate expansion. If a mechanical stress acts simultaneously to the chemical one the initial strength development is

Fig.2: Stress Corrosion of Portland Cement Mortars



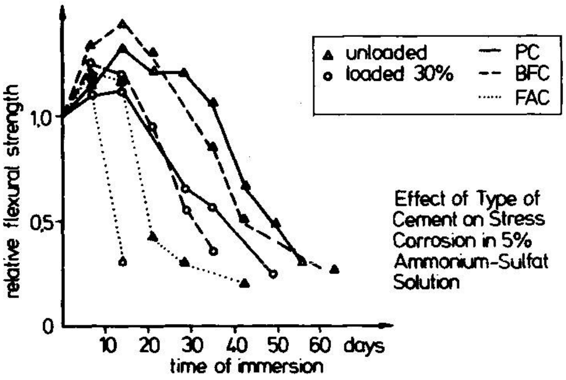
Effect of Load on Strength Development
of Mortar Prisms

diminished and the deterioration of the specimens occurs at earlier times. At very high load levels no initial strength increase is observed. Thus, stress corrosion leads to a significant decline in flexural strength for loaded specimens compared to unloaded ones. The higher the load the earlier failure occurs. Similar results are obtained for other media [2,3,5]. Thus it may be concluded, that cementitious materials are subjected to stress corrosion in many common aggressive aqueous solutions.

3.2 Effect of Type of Cement

Fig.3 shows the effect of type of cement. Stress corrosion occurs for all types of cement investigated.

Fig. 3: Effect of Type of Cement on Stress Corrosion of Mortars



The type of cement affects the life time i.e. in ammonium sulfate solution the portland cement and the blast furnace slag cement specimens show a higher life time compared to the fly ash cement specimens respectively, because in the corrosion times involved the higher $Ca(OH)_2$ content of the portland cement and the blast furnace slag cement pastes acts as a buffer. The strength decline is most pronounced for the fly ash cement. For the loaded specimens the same behaviour is observed. Portland cement and blast furnace slag cement show nearly the same strength decline with immersion time, whereas the fly ash cement showns very rapid failure. Thus, in stress corrosion the effect of the type of cement is the same as in ordinary corrosion of cementitious materials.

3.3 Effect of Water/Cement-Ratio

Due to the fact that cementitious materials in ammonium sulfate solution are subjected to two entirely different forms of corrosion failure, namely the dissolving attack and the sulfate expansion attack the effect of the water cement ratio depends also on the type of cement. Fig.4 shows the effect of the w/c-ratio for portland cement and fig.5. for blast furnace slag cement prisms both unloaded and loaded with a load level of 30% of initial strength. The decline of strength becomes

Fig.4: Effect of W/C-Ratio on Stress Corrosion of Portland Cement Mortars

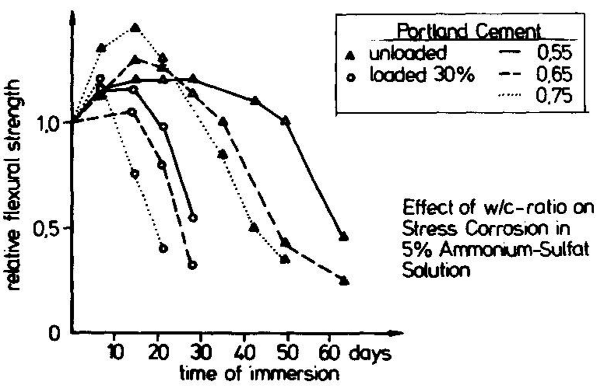
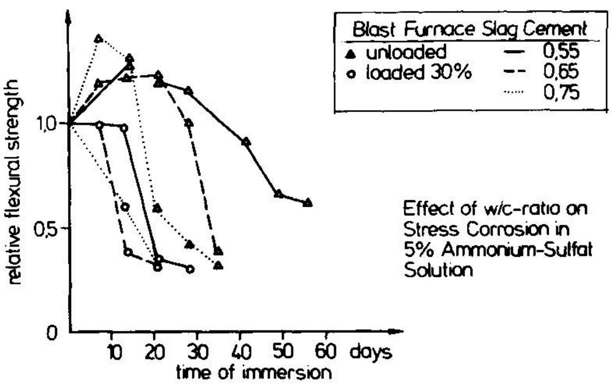


Fig.5: Effect of W/C-Ratio on Stress Corrosion of Blast Furnace Slag Cement Mortars

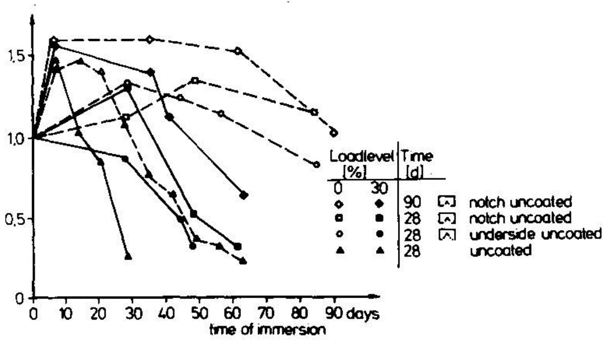


more distinct with increasing w/c-ratio for loaded and unloaded specimens. Thus, in general the effect of w/c-ratio on stress corrosion is the same as during ordinary corrosion without mechanical stresses. However, when a mechanical stress is superimposed onto the chemical stress the differences in strength development seem to become smaller than those observed with unloaded specimens. Furthermore, it can be seen from fig's. 4 and 5 that the initial increase in strength increases with increasing w/c-ratio, i.e. with increasing capillary porosity. This is an indication that the initial strength increase of mortar specimens stored in ammonium sulfate solution is due to the formation of new phases inside the pore volume.

3.4 Effect of Surface Coating

Fig.6 shows the decline of flexural strength of differently coated notched cement mortar prisms /6/. The coating prevents an attack on the coated areas of the specimen. Thus, the corrosion resistance of coated specimens is signifikantly higher than that of uncoated ones. The more of the specimens surface is covered by the coating the better is the performance under corrosion. However, the picture is quite different under stress corrosion conditions. The coated specimens, both those with the tensile side uncoated and those with only the notch region uncoated show extremely distinct load effects. This means, that if a coating has some failure points i.e. cracks, through which the aggressive medium can penetrate, these areas are the starting points of a severe stress corrosion, which under load, can reduce the life time of the structure to values of the unprotected material. The effect is observed for both types of coatings studied thus far, namely notch-free and one side free-coatings. It seems therefore to be necessary to reconsider the use of coatings at least for highly load bearing structural members subjected to a chemical attack.

Fig. 6: Effect of Coated Mortar Prisms under Stress Corrosion 5% Ammonium-Sulfat Solution



4.SUMMARY AND CONCLUSIONS

Stress corrosion leads to significant reductions in strength of cementitious materials subjected to simultaneously acting mechanical and chemical stresses. Several media, amongst which are ammonium and sodium sulfate have been shown to cause stress corrosion. Severe reductions in strength and life time are the consequence of this process. In general, most of the important parameters of concrete technology affecting corrosion resistance also affect stress corrosion behaviour in about the same manner. An additional deleterious effect of stress corrosion has been discovered with coated specimens. Cracks and other failures in the coating result in an enhanced stress corrosion reducing the life-time of a coated structure to that of an uncoated one. Stress corrosion is further assumed to be responsible for the large differences known to exist between conventional laboratory studies, neglecting the effects of mechanical stresses on chemically attacked cementitious materials. It is therfore urgently necesseary to study the observed effects in more detail and to incorporate stress corrosion in the standards and evaluation procedures of durability of cementitious materials.

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Prestressed Bridge Girders after 20 Years of Service

Poutres en béton précontraint après 20 ans de service

Vorgespannte Brückenträger nach 20 Jahren Beanspruchung

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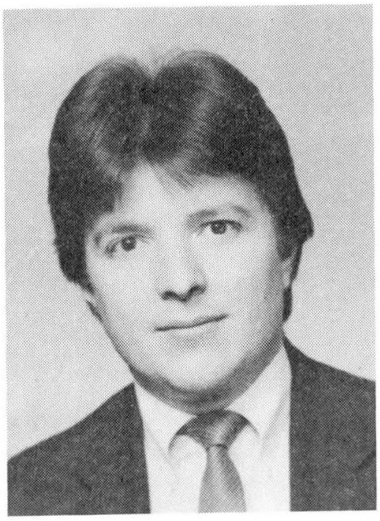

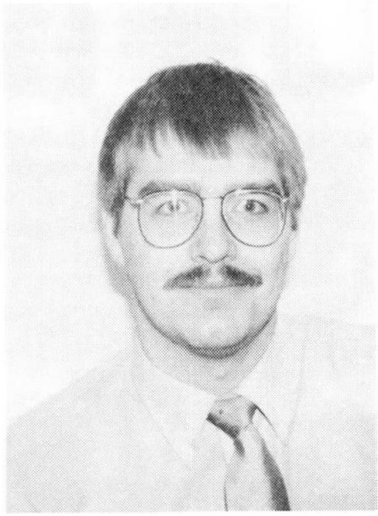

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SUMMARY

The prestress losses, material properties, fatigue, and structural performance characteristics of four prestressed bridge girders are described. The girders had been in service under actual traffic conditions for twenty years, and showed little corrosion or durability problems. Prestress losses with time were approximately equal to those predicted by current American codes.

RÉSUMÉ

Les pertes de précontrainte, les propriétés des matériaux, la fatigue, et les caractéristiques structurales de quatre poutres de pont en béton précontraint sont décrites. Les poutres ont été en service sous des charges de trafic durant vingt ans, et n'ont pas eu de problèmes de corrosion ou de durabilité. Les pertes de précontrainte avec le temps ont été pratiquement égales à celles prescrites par les codes américains actuels.

ZUSAMMENFASSUNG

Die Vorspannungsverluste, Materialeigenschaften, Ermüdung und das konstruktive Verhalten vier vorgespannter Brückenträger sind beschrieben. Die Träger waren für ca. 20 Jahre unter Verkehrslasten in Gebrauch und zeigen nur geringe Korrosions- oder Gebrauchstauglichkeitsprobleme. Die Vorspannungsverluste im Verlauf der Zeit entsprechen etwa den Werten der zur Zeit gültigen amerikanischen Normen.

1. INTRODUCTION

The determination of the actual material and structural properties is a key step in the rating of structures. In the case of most steel bridges the assessment of the material properties is quite straight forward. This is not the case, however, for concrete and prestressed concrete bridges where significant time effects are involved. In prestressed concrete bridges, in particular, loss of prestress due to relaxation, creep, shrinkage, and cyclic loads is very difficult to predict. In this paper the results of a study on four prestressed girders subjected to real traffic for twenty years is reported. The main objectives of the study were:

(1) To determine the actual prestress losses of the girders and to compare them to those given by current code-prescribed equations.
(2) To assess the remaining fatigue life of such girders.
(3) To determine the material properties of the girders and components and to compare them with non-destructive test results.
(4) To determine the amount of impact damage that such a girder can sustain before replacement or repair is required.
(5) To assess the performance of two types of strand repair techniques subjected to fatigue loading.

Due to space limitations only the results of the first two girder tests will be addressed with respect to items (1)-(4).

2. DESCRIPTION OF THE SPECIMENS

In 1986 a four-span county road bridge over an interstate highway in Minneapolis, Minnesota, was removed due to road realignment work. Four of the girders removed from the center spans of this bridge were brought to the University of Minnesota Civil and Mineral Engineering Structures Laboratory for testing. The girders were originally fabricated in July of 1967. At the time of removal they had been in service for approximately twenty years.

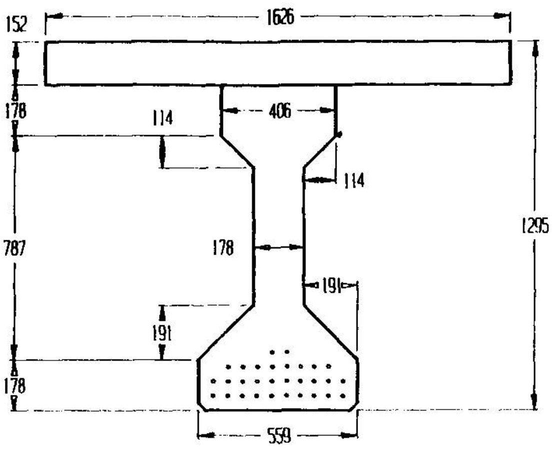

Figure 1 - Girder dimensions (in mm)

The girders are standard AASHTO-PCI Type III girders, and details of the girders are shown in Figure 1. The girders were 1143 mm. deep, 19.71 m. long, prestressed with thirty 13 mm. diameter 1724 MPa stress-relieved strands. Twenty-two of the strands had a straight profile in the bottom flange of the girders. The remaining eight strands were draped; the two hold-down points were located 1525 mm. either side of centerline. The strands were initially stressed to a design prestressing level of 1206 MPa. The construction records indicate that the girders were pulled to approximately 2 percent over this design prestress.

Each girder was tested individually, with a new slab cast to simulate the actual bridge. The new slab was 152 mm. thick, 1626 mm. wide, and reinforced in the same manner as the original bridge slab. The loads were applied by two actuators located at the hold-down points, resulting in a constant moment region over the middle 3025 mm of the girder.

3. PRESTRESS LOSSES

The girders arrived at the laboratory in an apparently uncracked condition. The only visible damage was located at the ends of the girders: a small amount of epoxy paint covering the ends of the strands had spalled and rust was evident on the ends of the strands. The first loading applied to the girders was a cracking cycle. This test was repeated several times to accurately determine the crack opening load. The measured load at first cracking was 378 kN, and the loading was continued to 623 kN, about 45% of its calculated ultimate capacity. Small cracks propagating up to 620 mm from the bottom fiber in the constant moment region were the only visible damage due to these loadings. Four techniques were used to estimate the prestress losses:

(a) Formation of the first crack: From the load at first cracking and tensile capacity of the concrete from cores, the prestressing can be estimated.
(b) Reopening of cracks: By carefully monitoring the opening of cracks with high-resolution LVDT's during reloading, the decompression load can be calculated.
(c) Discontinuities of the load-deflection curve: The change of stiffness when the cracks reopen can be obtained directly from load-deflection curves.
(d) Exposing, instrumenting, and cutting strands: By using strain gages, a direct measurement of the prestress level can be obtained.

While it is difficult to state the initial prestress because the construction documents are the only source available, the in-situ prestress levels as given by methods (a) to (c) were very close to one another. They indicated a remaining prestress of 895 MPa in the strands, or about 74% of the original prestressing. Method (d) gave somewhat lower values (64%), but this can be explained by slight misalignment of the gages, transfer length and differences in prestress from strand to strand. The total prestress losses calculated were about 310 MPa, very close to the lump sum prestress losses predicted by the current AASHTO specification [1].

4. FATIGUE LOADING

The complete load histories imposed on the two girders are shown in Table 1. The load history was selected to model past and current American bridge specifications [1] for prestressed girders. They are based on the bottom fiber stress. The oldest specifications allowed no tension there, while more recent editions allow between 0.8 f'_c and 1.6 f'_c, where f'_c is the compressive strength in kgf/cm^2.

Girder 1 - Following the static tests (G1PL) to investigate prestress losses, the girder was subjected to almost three million cycles of load at increasing levels of nominal tensile stress in the bottom fiber. The corresponding stress ranges in the strands for fatigue tests G1F1 through G1F4 were 55, 69, 90, and 207 Mpa, respectively. Only a small crack growth was noticed during the initial cycling for G1F1 and G1F2; in general cracks stabilized with the first 10×10^4 cycles at each level. Most new cracks developed during the static tests conducted intermittently to monitor the amount of damage accrued during from fatigue. Figure 2 shows the load-deflection curves after each of the loading runs. Very little, if any, fatigue damage was evident after this very severe load history. There was noticeable permanent set in the specimen only after loading G1F4.

Girder 2 - After an initial series of static tests (G2PL), the beam was cycled for two million cycles (G2F1 and G2F2) in its undamaged state. The concrete in the bottom flange at the centerline was then chipped away to expose four strands to simulate damage due to impact from a truck travelling underneath the bridge (G2D1). The beam was then subjected to fatigue loading (G2F3). Two bottom strands were then cut to simulate additional damage (G2D2), and the girder was fatigued again (G2F4). After this two more strands were cut (G2D3) and the fatigue load repeated (G2F5 and G2F6). One strand broke during G2F4, and at least another during G2F5. Figure 3 shows the load-deflection curves for static tests after each of these runs.

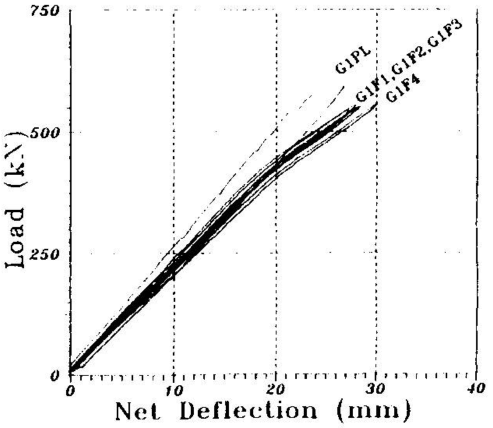

Figure 2 - Load-deflection for G1

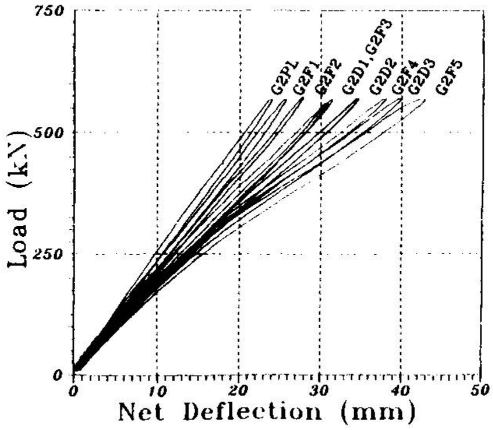

Figure 3 - Load-deflection for G2

The results of the tests on the first two specimens indicated that fatigue loading which produces strand stress ranges of less than 104 MPa has little or no effect on the ultimate strength and ductility of the member. The size of these girders and their close spacing in the field means that very little damage could have been done by fatigue since the sections were uncracked during their service life. The stress range in service was probably less than 20 Mpa.

TABLE 1 - Load Histories

Girder Number	Test Label	Load Type	Number of Cycles (N)	Bottom Fiber Stress	Purpose of Test
1	G1PL	Static	10	$3.2\sqrt{f'_c}$	Determine cracking load and losses
1	G1F1	Cyclic	5×10^5	0	Fatigue
1	G1F2	Cyclic	10×10^5	$0.8\sqrt{f'_c}$	Fatigue
1	G1F3	Cyclic	12×10^5	$1.6\sqrt{f'_c}$	Fatigue
1	G1F4	Cyclic	6×10^4	$3.2\sqrt{f'_c}$	Fatigue
1	G1U	Static	1	---	Test to ultimate
2	G2PL	Static	10	$3.2\sqrt{f'_c}$	Determine cracking load and losses
2	G2F1	Cyclic	5×10^5	$0.8\sqrt{f'_c}$	Fatigue
2	G2F2	Cyclic	15×10^5	$1.6\sqrt{f'_c}$	Fatigue
2	G2D1	Static	5	$1.6\sqrt{f'_c}$	Concrete around strands removed
2	G2F3	Cyclic	5×10^5	$1.6\sqrt{f'_c}$	Fatigue
2	G2D2	Static	5	$1.6\sqrt{f'_c}$	Cut two strands
2	G2F4	Cyclic	5×10^5	$1.6\sqrt{f'_c}$	Fatigue
2	G2D3	Static	5	$1.6\sqrt{f'_c}$	Cut two strands
2	G2F5	Cyclic	18×10^3	$1.6\sqrt{f'_c}$	Fatigue
2	G2U	Static	1	---	Test to ultimate

5. ULTIMATE STRENGTH TEST

After approximately three million cycles, both Girder 1 and 2 were tested monotonically to failure (Figure 4). In both cases the failure was initiated by the upward buckling of the longitudinal slab reinforcement at very large centerline deformations (530 mm or greater). The final failure occurred as crushing of the slab followed by an explosive outward failure of the poorly confined beam web. For Girder 1, the failure occurred at a load of 1303 kN and at a centerline deflection of 530 mm. The load at ultimate constituted approximately 95 percent of the ultimate capacity of the beam based on nominal material properties and the assumption that all steel yielded. By the time failure was reached the entire 10 ft at the center of the beam had formed a long plastic hinge, and inclined shear cracking had moved out from the constant moment region to the quarter points in the beam. The inclined shear cracks in this area had reached the top flange of the beam, while flexural cracking had progressed to the bottom of the slab. The fatigue loading did not appear to have affected the ultimate strength of the section.

The failure for Girder 2 was similar, except that it occurred at a much lower load (890 kN) and somewhat higher deflection (635 mm.). This was due to: (1) the cutting of the strands and loss of concrete section and (2) fracture of at least two additional strands during G2F4 and G2F5. The cracking in Girder 2 during the last three loading runs was very severe, with large portions of the bottom flange completely separated from the girder.

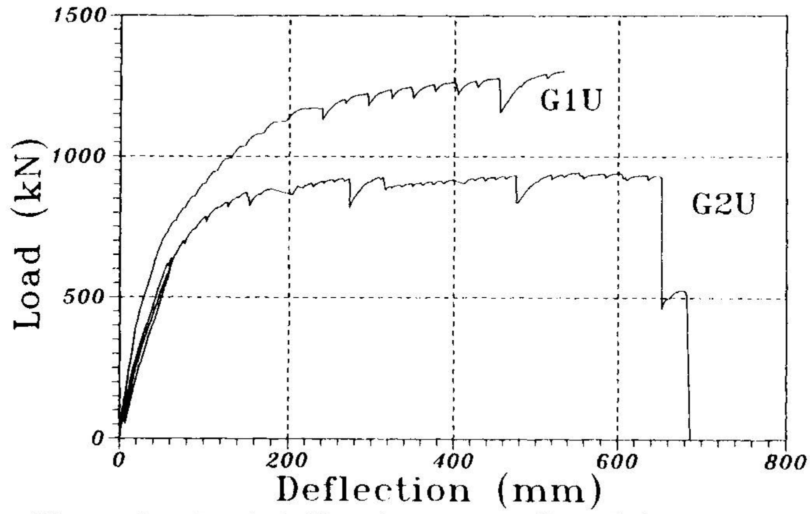

Figure 4 - Load-deflection curves for ultimate tests.

6. MATERIAL PROPERTIES

A series of nondestructive tests was conducted on Girder 1. The tests included a variety of uniformity/strength tests as well as chloride ion penetration investigations. Windsor probe (penetration test), Schmidt hammer (surface hardness), pulse velocity (compressive wave velocity) and breakoff tests (lateral pressure required to detach countersunk cylinder) were conducted and were correlated with 50 mm. and 100 mm. diameter cores drilled from the girder. In these tests, the concrete was found to be quite uniform throughout the girder with a compressive strength of about 58 Mpa. THe results showed a low coefficient of variation for all of the test methods with the exception of the break-off test and the 100 mm. cores. Problems with obtaining straight cores and good capping of the specimens help explain this variability.

Because the girders had been exposed to substantial amounts of deicing salts as spray from the interstate highway underneath, corrosion of the strands was considered a major potential hazard. The girders themselves received little or no deicing salts from above, as the original deck acted as a barrier. The top steel in the original deck was quite corroded, but the chlorides had not penetrated down to the girders. The strands which were exposed to check the effective prestress of Girder 1 were located in the flange near the end of the girder in an uncracked region. Evidence of some pitting corrosion appeared on one of the strand within 100 mm. of the end of the girder. Otherwise, the strands, which had a cover of 50 mm., appeared to be in excellent condition. Chloride ion penetration tests gave readings which were within the threshold limits of 250-350 ppm by weight of concrete usually assumed as the corrosion threshold. The highest reading obtained was 270 ppm, but the next highest readings were on the order of 120 ppm. It is interesting to note that the readings obtained from the bottom flange were consistently higher than those obtained from the web, and the readings obtained on one side of the girder were consistently higher than those obtained from the other side. It is expected that the girders which were exposed to the incoming traffic would have higher readings because the salt-concentrated mist tends to be carried under the bridge with the forward motion of the cars. Consequently, the side of the girder with the higher concentration of salts most likely faced the incoming traffic.

7. SUMMARY

The tests carried out so far indicate that the prestressed girders were in excellent condition after twenty years of service in an aggressive environment. There did not appear to be problems associated with corrosion of the girders, most likely because of the excellent concrete quality and the depth of cover. The prestress losses, estimated to be on the order of 310 Mpa, correlated well those predicted by the current AASHTO lump-sum method. The fatigue loading imposed indicated that for loadings up to HS20-44 or Type 3S2 vehicles the stress range was probably in the infinite life region of the fatigue curves. Thus the girders could be reused in other bridges if they are removed with care and NDT techniques indicate adequate material properties and no evidence of strand corrosion.

ACKNOWLEDGEMENTS

The work described herein was funded by a generous grant from the Minnesota Department of Transportation (MNDOT). The advice and support of Don Fleming, MNDOT State Bridge Engineer, and Gabe Bodoczy, MNDOT Research Director, are acknowledged. The efforts of graduate students Paul Bergson and Florinda Coggins are also gratefully acknowledged.

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Problems in Concrete Culvert Durability Studies

Problèmes liés à l'étude de la durabilité des ponceaux en béton

Probleme mit Untersuchungen zu Dauerhaftigkeit von Betondurchlässen

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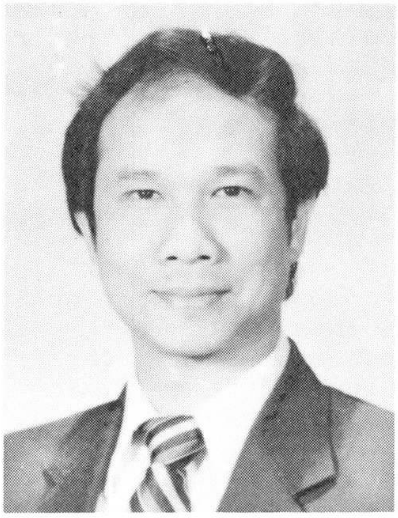

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SUMMARY

Several studies made concerning the durability of concrete culverts have led to inconclusive results. Inadequate consideration of enabling and triggering events contributed to the general problems in assessing the life expectancy of culverts. The more recent techniques using regression analyses for determining the culvert's service life encountered problems related to the inadequacy of sample data representative of the culverts and the inferior concepts employed to treat the data.

RÉSUMÉ

Quelques études, menées sur la durabilité des ponceaux en béton, ont débouché sur des résultats peu concluants. Des considérations insuffisantes, en ce qui concerne les multiples paramètres à considérer, contribuèrent à rendre difficile le problème général de la prévision de la durabilité des ponceaux. Des techniques d'analyse récentes, basées sur les régressions, furent appliquées à la détermination de l'aptitude au service des ponceaux. Toutefois, ces analyses se sont heurtées à l'inadéquation aussi bien des données simplifiées représentatives des ponceaux que de l'infériorité des concepts employés pour traiter les informations.

ZUSAMMENFASSUNG

Verschiedene Untersuchungen zur Dauerhaftigkeit von Betondurchlässen führten nicht zu schlüssigen Resultaten. Ungenügende Berücksichtigung der massgebenden Einflüsse führten zu Problemen bei Lebensdauerprognosen. Neuere Versuche unter Verwendung von Regressionstechniken scheitern an mangelhafter, nicht representativer Datenbasis und an ungenügenden Konzepten bei der Verarbeitung der Daten.

1. INTRODUCTION

Emphases of earlier studies of culvert durability are primarily placed on field surveys and experience of the analysts. Simple manipulations and statistics were performed manually to estimate culvert service life. Certain studies show the effects of enabling events resulting from pipe manufacturing and installation procedures. Examples of these events are the inadequacy of material, design, or construction processes. However, during the culvert durability studies, information concerning these events was not considered. The most frequently investigated events that caused the deterioration of culverts are the environmental conditions. These conditions are called the triggering events or external events that triggered the deterioration of the culverts, such as, the acid attack and flow velocity in the culverts. Current studies emphasize more on these events.

With the utilization of computers, more sophisticated statistical models can be performed using a large number of data. Unfortunately, this does not necessarily guarantee the reliability of the techniques developed. In some cases, casual observations were performed, and inadequate number of data was used to force the production of prediction models. In the following section, significant studies on durability of concrete culverts from several sources, including the author's own investigations, are presented. Emphasis of the studies are placed on the problems that may occur in relation to the reliability of the results.

2. STUDIES OF CONCRETE CULVERT DURABILITY

Annual deterioration, qualitative, and regression studies had been performed for the durability of concrete culverts. The following are examples of such studies performed in various states and summarized in Tables 1 and 2. Experimental studies are not discussed here.

2.1 Annual Deterioration and Qualitative Studies [1]

The earliest culvert durability study known to the author was performed in the State of Georgia in 1928, where 252 monolithic concrete culverts and 326 concrete concrete culverts were examined. The culverts were installed between 1915 and 1924. The average life expectancy is 31.2 and 27.6 years for monolithic culverts and concrete culverts, respectively. Another survey of 1,837 reinforced concrete pipe culverts was conducted in West Virginia between 1932 and 1933. The age of the culverts inspected ranges from 3 to 11 years. Variables contributing to concrete durability were discussed. The study results in 50 years average expected service life with an average annual deterioration of 2%. In 1947, Pennsylvania Department of Highways performed culvert study based on a survey of 10,439 concrete culverts installed from year 1918 to 1927. The study concludes a life expectancy of 40 years assuming that yearly deterioration remained constant.

A total of 442 concrete pipes were investigated in the State of Mississippi in 1964. The age of pipes ranges between 5 to 41 years, averaging 22.9 years. The study that was conducted qualitatively based only on field performance prescribed no life expectancy. However, it concluded an excellent condition of the pipes.

2.2 Regression Studies [1]

The research performed in 1974 for the State of Utah may be the first study of concrete culvert durability using regression analysis. Fifty eight pipe culverts were studied; however, only 14 of them were of concrete. A rating scale from 0 (failure) to 10 (excellent) was used in the study. Several dependent variables were used such as pH, soluble salt content, and electric resistivity. Pipe rating was used as the dependent variable. The study recommended service lives of 40 years for the design of interstate highways and 30 years for other installations.

Two surveys of concrete culverts were conducted by the Ohio Department of Transportation: the first was in 1972 for 545 culverts and the second was the reinspection of 64 concrete pipes in 1984. In the first study the rating scale used ranges from 1 (excellent) to 5 (poor). Pipe rating was used as the dependent variable, while age, slope, pH, sediment depth, and rise were used as independent variables. Two sample groups, pH below 7 and above 7, were analyzed for separate prediction models. Emphasis was placed on the latter group which resulted in prediction models. For pH value over 7, the models yield culvert service life of thousands of years. The second study is a refinement of the earlier one performed for sample group of pH below 7. Pipe ratings from 0 (as manufactured) to 100 (reinforcing gone) were used here. Rating was used as the dependent variable, while flow, pH, pipe size, slope, and sediment depth were used as the independent variables.

The author of this paper performed two regression studies. The first is based on information of 521 sections of concrete culverts made available by the Ohio Department of Transportation (ODOT) [1]. Rating scale with ranges similar to that of ODOT (1=excellent to 5=poor) was used in the study. Pipe rating was used as the dependent variable, while age, rise, pH, slope, and sediment depth become the independent variables. No grouping of pH values was performed here. Six multiplicative and six additive models were generated as the results of the study. However, these models can not be used for predicting the service life of a particular culvert. The expected "service life" of the culverts yield 86 years. The second study was performed by the author as a discussion of of ODOT's culvert study [3].

3. PROBLEMS ENCOUNTERED IN STUDIES

3.1 General Problems

Studies for determining the life expectancy of culverts have not been completely satisfactory. The variability in the design, material, manufacturing, installation, and maintenance of concrete culverts were seldom included in the analyses since usually such information is not available. Table 2 shows that only the earlier study in Georgia investigated the enabling events. Other studies had implicitly included the existence of these events through the rating scales. Furthermore, observations of culverts are usually performed at a certain "point in time" during the survey, where the variables related to the culvert geometry and triggering events are measured. During the observations, these triggering events are assumed constant. However, events such as, flow velocity, flow depth, sediment depth, and acidity may not be the same throughout the life span of culverts. These factors may contribute to the inaccuracy and unreliable results of analyses based on the average annual deterioration or regression analyses.

3.2 Inadequacy of Culvert Data

Many of the sample data, including those used by the author, are not representative of culverts in a particular state. For example, Figure 1 shows a bi-nodal sample distribution of pH values of observations in the State of Ohio. A representative sample data is expected to have a uninodal distribution. A plot of the variable Age and pH of these data shows a "boxing" of culvert ages above pH=7 as shown in Figure 2. These data indicate the lack of observations in other pH regions.

3.3 Conceptual Problems

In the regresssion analyses, the rating of culverts is usually treated as the response variable dependent upon other variables, such as, age, rise, flow depth, flow velocity, sediment depth, slope, and pH values. Only independent variables that contribute any information to the prediction of the culvert rating are included in the regresssion equation. An example of such an equation is as follows [2]:

$$RATE = -05469+0.0316AGE+0.0099RISE+11.2484/PH+0.2377SLOPE^{1/2}$$

In several studies, the variable AGE is algebraically exchanged with variable RATE, such that AGE becomes the response variable. Then, as RATE is set to a scale that represents the expiration of the culvert's service life, AGE becomes the "service life" of the culvert and the prediction equation is used for predicting a particular culvert for given variables. This technique is erroneous since throughout the regresssion, AGE is assumed, as it should be, an independent variable, but at the same time becomes a dependent variable when algebraically altered with RATE. Also, an attempt to treat AGE as a response variable for use in the regression analysis is inapropriate, since AGE is the time from installation to inspection of the culverts and is logically independent from other variables [3].

Incomplete or inadequate data adds to the problem of predicting the culvert's service life. For example, in a sample data set, the majority of the data were related to culverts still in service, and only a small fraction of the inspected samples represent expired culverts [3]. Despite the fact that none of the culverts in the data set are over 60 years old, yet through the algebraically altered prediction equation, one could predict that many of these culverts are already several hundred years old. Figure 3 shows an example result showing the relation between the

TABLE 1. Method of analysis and life expectancy of culverts

STATE	METHOD	SAMPLES	AGE (YRS)	LIFE EXP. (YRS)
Georgia	Avg. Annual Loss	252 monol. 326 pipes	2-13	31.2. 27.6.
W. Virginia	Avg. Annual Deterioration	1,837 RC pipe	3-11	50.0
Pennsylvania	Avg. Annual Deterioration	10,439 culv.	20-29	40
Mississippi	Qualitative Field Perf.	442 pipes	5-41	NA
Utah	Regression	14 pipes	NA	30-40
Ohio (ODOT)	Regression	545 pipes	1-45	Varies
Ohio (ODOT)	Regression	64 pipes	14-56	Varies
Ohio (ODOT)	Regression	198 pipes	5-57	Varies
Ohio (OSU)	Regression	521 pipes	1-45	86 (avg)
Ohio (OSU)	Regression	198 pipes	5-57	NA

TABLE 2. Enabling and triggering events considered for analysis of culvert

STATE	TIME OF SURVEY	VARIABLES RELATED TO ENABLING EVENT	VARIABLES RELATED TO TRIGGERING EVENT
Georgia	1928	*Fill height *Poor quality control of concrete mat'l *Joint failure of rigid sectional culverts *Poor alignment *Poor maintenance	*Wet/dry cond. *Scour cond.
W. Virginia	1932	*Type of fill *Trenching method	*Acidity *Flow of water
Pennsylvania	1947	NA	*Flow of water *Wooded area
Mississippi	1964	*Alignment offset *Faulty joints	*Clay flow upward and outward *Acidity
Utah	1974	NA	*Acidity *Soluble salt *Electric Resistivity
Ohio (ODOT)	1972, 1984, 1988	NA	*Acidity *Sediment depth *Presence of sediment *Flow depth *Flow velocity
Ohio (OSU)	1986, 1988	NA	*Flow depth *Flow velocity *Acidity *Sediment depth *Presence of sediment

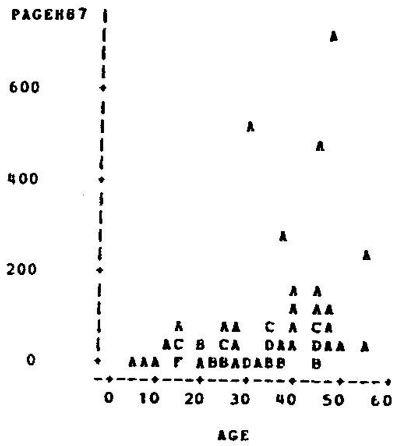

Fig. 3. Actual Age vs. Predicted age

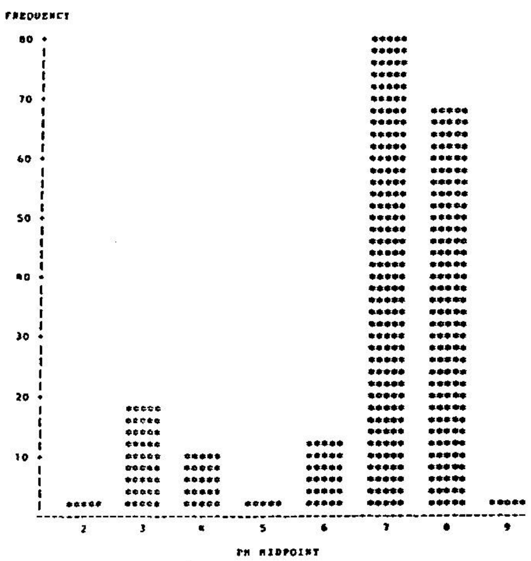

Figure 1. Binodal sample distr.

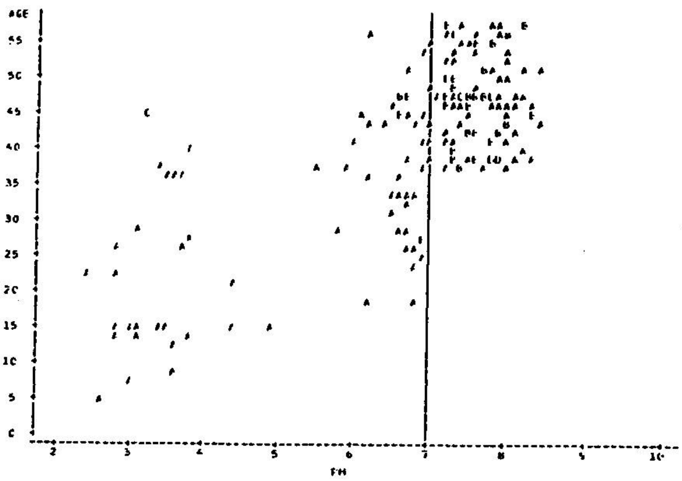

Fig. 2 . Boxing effect of sample data

predicted age (obtained using the prediction equation) and the actual age of the culverts [3]. Note that about 30% of the predicted ages are more than the oldest culverts in the sample. In the figure, the correct term of "predicted age" is used instead of the "predicted service life" of the culverts. Such an equation could erroneously predict up to nearly 3,500 years of concrete culvert service life.

4. CONCLUSIONS

The inadequacies of data and information concerning the culverts, in addition to the often conceptual problems involving the methods of treating the data, have resulted in the large variability of the predicted "service life" of the culverts. With the emergence of computers, culvert experts began to capitalize the use of regression techniques. However, the author feels that the results of recently applied regression techniques are not conclusive, if not unreliable. Problems had occurred in relation to the culvert data and the concept employed for analyzing the data. Several culvert data sample sets used for regresssion analyses are inadequate and not representative of culverts of a particular environment. Furthermore, these sample sets often represent culverts still in service while regression analyses were employed to generate prediction equations for determining the expiration of culverts' life.

The author of this paper feels that much has to be done if one expects reliable yet accurate results using regression techniques. Specifically, adequate amount of expired culvert samples from the same environmental conditions are required. Also, samples manufactured and installed in a relatively similar period of time (small range of age) are preferred. In addition, experimental accelerated testing of culverts installed in certain environmental conditions should also be considered.

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Design Approach with Respect to Durability

Approche de projet en vue d'une bonne durabilité

Bemessungsvorgehen zur Gewährleistung einer hohen Dauerhaftigkeit

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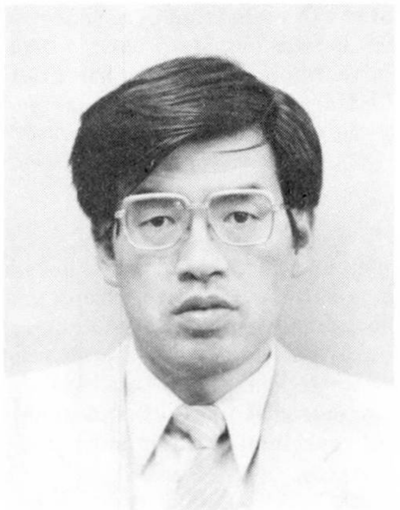

Yoshiro Kohama, born in 1945, received his Doctor's degree in 1981 from Nagoya University, spent ten years in research and education on structural engineering.

SUMMARY

The present report deals with a case study of realization of design specification with respect to durability on a medical and welfare complex for the aged. Various kinds of durability damage for the life span should be analyzed and synthesized to develop design specifications architecturally, structurally and environmentally which can be successfully accomplished at construction sites.

RÉSUMÉ

Ce rapport concerne le projet relatif à un complexe de soins hospitaliers, pour personnes agées, la conception étant soumise aux conditions imposés non seulement par la construction mais aussi par l'environnement, du point du vue de la durabilité et de la maintenance. Les dommages éventuels, tels que déformation des planchers, altération des revêtements de sol, neutralisation du béton, infiltration des parois, dilatation ou resserrement des joints, ont fait prévoir un ensemble de dispositions visant, au niveau même de la construction, à satisfaire aux tests de contrôle de la qualité.

ZUSAMMENFASSUNG

Dieser Beitrag behandelt eine Fallstudie für die Ausschreibungsbedingungen eines Seniorenheimes im Hinblick auf die Dauerhaftigkeit. Verschiedene Arten von Alterungsschädigungen sind abzuklären, um den Entwurf bezüglich architektonischen und ingenieurmässigen Anforderungen derart zu verbessern, dass die Dauerhaftigkeit durch die Ausführungskontrollen gewährleistet werden kann.

1. INTRODUCTION

Durability of buildings is related to various kinds of complicated characteristics including traditional construction technology and material, the climate of temperature, humidity, wind and sunshine quantity at the construction site, various type of loading, particularly due to natural causes such as earthquake or typhoon, and client attitude toward maintenance after completion. Thus, reinforced concrete buildings(RC) should be designed and constructed in terms of these factors architecturally, structurally and environmentally. Damages of concrete building due to durability over its life span in Japan should be divided into three categories of loading of earthquakes, of non-structural members such as finishment or facement appeared naturally or artificially and of serviceability including environmental equipments, change of occupancy and architectural deterioration. Particularly Japanese seems sensitive to this architectural deterioration to a large extent, which should be predicted at an early stage of project. Generally the climate of Japan has distinct four seasons with arid, cold winter and humid, hot summer. Traditionally people has enjoyed their life under beautiful cherry blossoms in spring, a hazy moon in autumn, even cicada sound of hot summer in rapport with the nature in which people would like to die after aging. Historically Japanese architecture as a shelter, built on soft soil layers or alluvium, has been reflected conformity to the nature, including, positively, these beautiful seasons and, negatively, strong earthquake or typhoon. Furthermore, because of dense urban area due to recent economic growth landownership demands more complicated restrictions on almost all projects. If traditional shelters, which is made of wood, could resist severe cold winter, life of remaining seasons should be comfortable with aid of breeze. After the war a large number of concrete shelters appear and not necessarily conform to the traditional Japanese mind architecturally and environmentally. Consequently, it is necessary to develop more sophisticated design and technology in conformity to the nature and human behavior with respect of durability of concrete shelter over its life span.

The present report deals with a case study of realization of design method of durability on a medical and welfare complex for the aged which locates outside Nagoya where it is hot and stick in summer, and cold and windy in winter and furthermore sometimes has heavy rain. Concrete buildings in the complex should resist disastrous loadings of strong earthquake and typhoon structurally and the natural climate environmentally and architecturally[1],[2].

2. PRESENT COMPLEX

This private medical and welfare complex for the aged holds 200 beds and consists of three RC wings, namely, a medical clinic, two nursing homes and a day-care center as shown in Fig. 1. Various kinds of restriction surrounding this project demand more sophisticated design review not only structurally but also environmentally or even architecturally particularly of durability closely correlated to maintenance for the life span. Prominent properties are as follows.

Location : a coastal city outside Nagoya with 300 thousand population,
surroundings : faced a national route of considerable traffic noise, vibration and air pollution,
on alluvium or soft soil layers,
climate : temperature of $-2°C \sim 32°C$,
humidity of 50% $\sim$ 95%, occasional coastal wind and heavy rain,
facilities : a medical clinic (two story RC),
nursing homes (three story RC) and a day care center (one story RC),
including various equipments (solar panel, heating, cooling, sprinkler systems).

Hence, the design and construction should be accomplished against durability of these properties in harmony with the financial requirements.

3. DURABILITY DEFECT

This type of RC buildings, which is popular in Japan, are frequently built in dense urban area on alluvium certainly subjected to strong earthquake for their life span thus indispensably resulting the choice of appropriate piling foundation and anti- seismic walls in frame, which is controlled by regional structural regulation and code which can narrow diversity of durability with respect to safety against loading to satisfactory extent. Durability damages concerning to architectural details including cladding and finishment has diversity similarly to equipment system, which are closely related to maintenance by the clients for the life span of building. From a recent investigation[3] ill-conditions

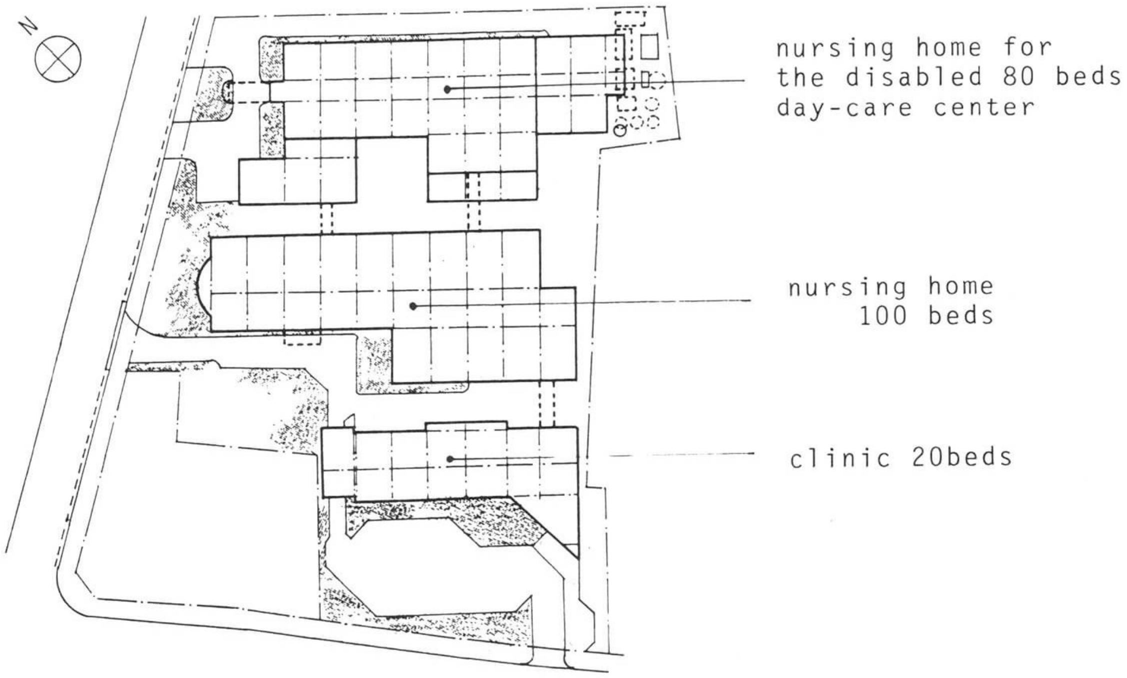

Fig. 1a Site plan of the complex

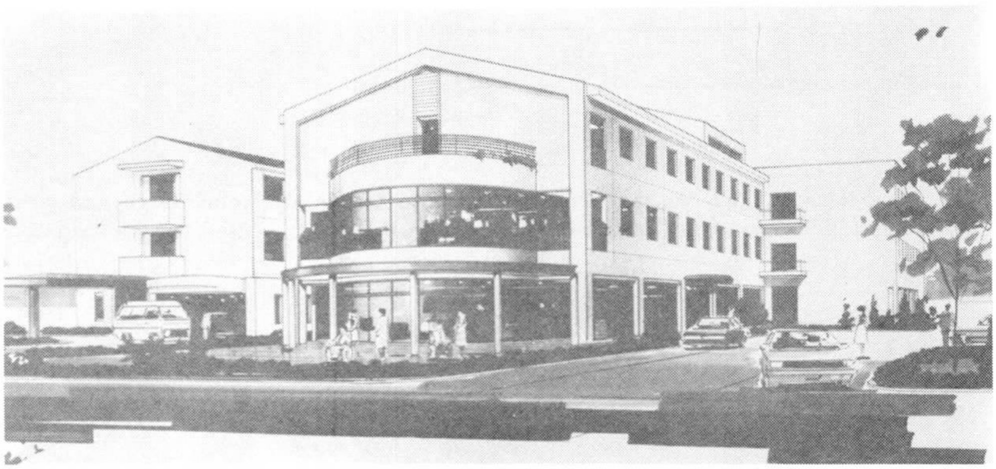

Fig. 1b Perspective of wings

due to a lack of durability can be classified in order of number of causes as follows.

ill-condition due to durability
- materials
- waters(rain, leak of equipment and pipe)
- strength of materials
- deterioration by aging
- form and size
- application
- noise and vibration
- temperature
- electricity and RI
- coloring
- miscellaneous

These ill-conditions can be extended into actual defects in detail which should be more practically evaluated at design specification and construction.

- defect
  - leaking
    - roof
      - water proofing materials
      - slope
      - flashing
      - number of roof drain
      - over-flow
      - pitched roof
    - exterior wall
      - intentional cracks
      - sealing materials
      - embeded pipe diameter $\leq \frac{1}{5} \times D$
      - coverage of pipe$\geq 30mm$
    - piping
      - leak of oil or gas
      - anti-errosion
  - dewing
    - roof
      - heat insulation, thickness
      - domain
    - exterior wall
      - heat insulation, thickness
      - domain
    - window
      - waste of dewing
      - heat insulation of frame
    - humid area
      - ventilation
      - air duct
      - heat insulation of pipe
  - wall-off
    - tiling
    - cornering
  - fire proof
    - spreading(grease trap)
    - ducting
  - settlement
    - porch slab
    - adjoining pipe
    - embeding

Furthermore, thus classified defects can be deployed more concretely from design and construction point of view. The following is an example of cracks which will be prevented as strictly as possible at each job stage.

- concrete cracking
  - design specification
    - exterior wall
      - intentional crack
      - wall thickness
      - reinforcement, a_t
      - diagonal reinforcement
      - opening crack
      - piping
    - roof, eaves
      - reinforcing
      - diagonal reinforcing
      - intentional crack
  - ready mixed concrete
    - mixture
      - slump
      - water cement ratio
    - material
      - normal
      - AE
  - concreting
    - amount per day
    - humid curing

As a result these categorical characteristics on durability are strongly correlated to each other which is, however, difficult to evaluate its priority precisely. When these characteristics are asserted as a database of AI technology, which consists of facts and rules, it becomes rather easy to search an optimal path to any durability goal by means of declarative languages such as Prolog which has versatility of backtracking manipulation. Herein, on an personal computer effective rules are asserted by Prolog.

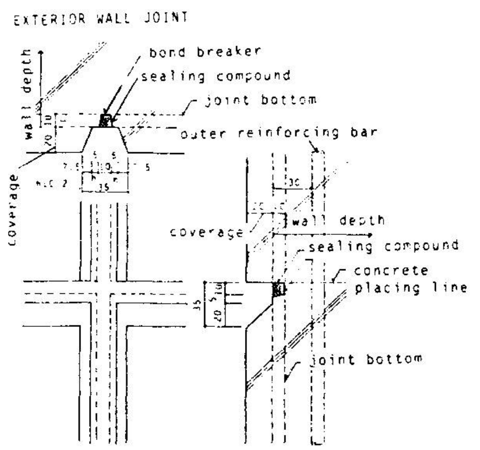

Fig. 2a Exterior wall joint

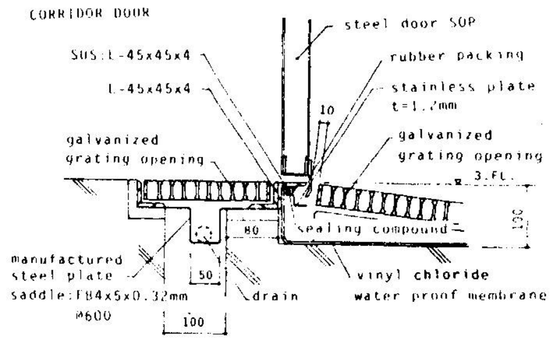

Fig. 2b Corridor door

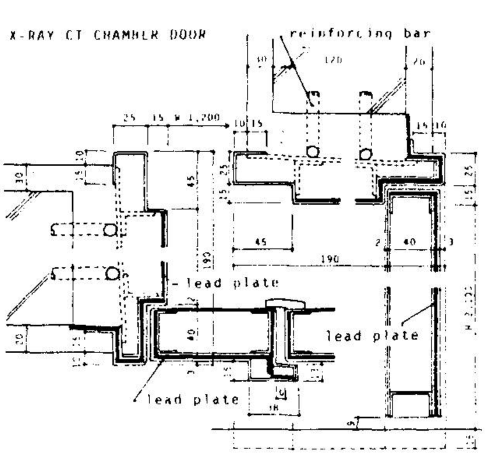

Fig. 2c X-ray CT chamber door

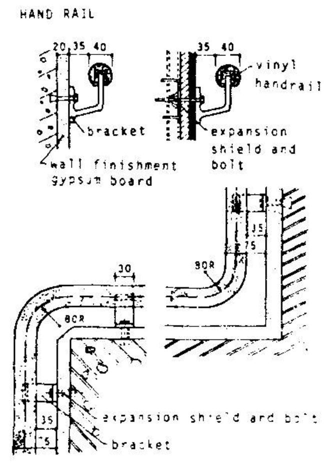

Fig. 2d Handrail

4. DESIGN SPECIFICATION

Although the occurrence of durability defects should be avoided as strongly as possible particularly after completion, an architecture should also original and beautiful itself. Thus creation of idea must be flexible with consideration of the restriction of durability. It is necessary to develop more rational details as the design specification to the construction site with reasonable investment. The design specification should be an understandable description of necessary information.

Fig. 2 shows several examples of details developed relating to durability defects. Furthermore, these details should be translated into field information for workers who will appreciate and realize by means of TQC.

5. REALIZATION BY TQC

From the investigation on durability of various RC buildings, damages concerned with construction process are due to the following reasons, namely, uncertain information of design specification to the construction field and a lack of appropriate construction technique. Particularly, on the latter there is a large technological diversity among workers and technicians at the site whose traditional skills become to decline as economical growth. It is necessary to develop a more progressed approach or an alternative to synthesize these unlevel skills to attain the prescribed durability tolerance.

To accomplish a certain level of quality control the traditional management or SQC (Statistical Quality Control) demands an inspection system, which tends to decrease productivity. Although a strict and large amount of inspection is accomplished there may appear oversights which causes durability defects particularly after completion. Hence, TQC (Total Quality Control) movement can be applied,

which means continuous quality control movement by QC circles at the site by all members particularly participated by workers who propose and devise various improvements on problems related to durability. TQC was originally developed in the industrial production process, and then prevailed extensively in the construction field successfully in Japan.
Concrete goals relevant to almost all kinds of field problems are discussed in QC groups including field workers and technicians at the construction site continuously. These group discussions are reflected to the construction process in progress. The construction industry of Japan has accepted this movement from the early seventies prevailing prominently nowadays. Presently the TQC is extended to even GWQC (Group Wide Quality Control). At the present construction site field workers including engineers accomplish the TQC movement to realize the durability tolerance specified by the prescribed design specifications. Many QC circles are established, each of which consists of several persons who belong to the same occupation. Each QC circle find an activity subject on durability from its surroundings, which is not necessarily sophisticated from the engineering point of view. Rather trivial problems to be improved at the site should be preferred, which QC circle participants analyze and discuss. The causes and results of subject are derived by means of interaction charts after discussion ordinarily for half an hour or less of several times on duty hours. This implies that a direct reflection of proposals by the field workers to concrete device of improvement can be accomplished. Naturally, these proposals are found through daily construction process and the results of their realization is compared to goal. This process is repeated until satisfactory level is attained. Thus, the realization of TQC is practically achieved for cost less than the required.

6. CONCLUDING REMARKS

Medium and low rise RC buildings, which are in the majority in Japan, reflect the social, economical situation with even sometimes chaotic contradictory results of durability demands including safety and deteriorations of architectural, environmental serviceability after completion. These durability demands should be realized and balanced not only engineeringly, architecturally but economically at each stage of design specification and then construction level. It is practical to make analysis and synthesis of durability by AI technology on a personal computer to arrange facts and rules with respect to damages. Thus obtained design specification is deployed at the construction site. However its reliable realization is not always easy because of unlevel skill of subcontract workers. TQC movement by these workers can ensure the realization at the site successfully and furthermore voluntarily. This process could be successfully applied to highly facilitied RC buildings of a medical and welfare complex for the aged to guarantee of balanced durability for life span.
Finally, the authors would like to thank Shimizu Corp. on this study.

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