PRESENTATIONS

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Durability Aspects in the Design of Steel Highway Bridges

Aspects de durabilité en vue du projet de ponts-routes métalliques

Aspekte der Dauerhaftigkeit beim Entwerfen der Stahlstrassenbrücken

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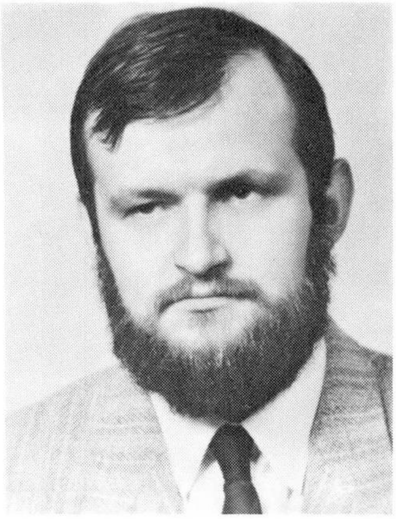

Adam Wysokowski, born 1954, received his civil engineering degree at Technical University of Wroclaw, Poland. Earned his D. Sc. after five years at Institute of Civil Engineering of that University. Since 1986 has been head of Bridge Dep. of Roads and Bridges Research Institute. Adam Wysokowski investigates problems of durability of bridges, particularly steel highway bridges fatigue.

SUMMARY

The paper contains a description of a method proposed by the author involving analytical and graphic method of operational durability estimation and design which takes into account the fatigue of the structural elements of bridges. By using this method, not only the operational life but also the permissible standard stress can be determined for the analyzed structural elements at the design stage, whereby the fatigue hazard is eliminated. This has been illustrated by calculation examples.

RÉSUMÉ

L'auteur propose une méthode analytique et graphique servant à déterminer la durée d'exploitation ainsi qu'à établir un projet en tenant compte de la fatigue des éléments structuraux des ponts. La méthode considérée permet de déterminer la durée de vie ainsi que la contrainte admissible pour les éléments structuraux analysés lors du projet. Le sujet est illustré par des exemples de calcul.

ZUSAMMENFASSUNG

Der Autor schlägt eine analytische und graphische Methode zur Bestimmung der Dauerhaftigkeit sowie des die Ermüdung der Brückenkonstruktionselemente berücksichtigenden Entwerfens vor. Die dargestellte Methode erlaubt die Bestimmung der Lebensdauer sowie die Festlegung der zulässigen Normspannung für die Konstruktionselemente. Das Vorgehen wird an einem Beispiel erläutert.

1. INTRODUCTION

Elements of steel bridges work in very unfavorable atmospheric conditions and they are subjected to variable loads of different duration and intensity. This applies particularly to highway bridges. As a result, a relatively great number of steel highway bridges have to be renovated and reconstructed. Also the neglect of fatigue effects at the design stage has resulted recently in numerous fatigue failures of various components of steel highway bridges.

In this context, the problem of complex evaluation of the operational durability margins of the existing bridges appears. And in the case of bridges that are to be designed, the problem how their structural elements should be designed to avoid the fatigue hazard and thereby to prolong their operational life becomes very important.

2. OPERATIONAL DURABILITY OF STEEL BRIDGES

Stress spectra from real loads serve as the basis for the evaluation of the operational durability of bridge elements. Continuous field measurements conducted on bridge objects are one of the ways in which the spectra can be obtained. However, due to the considerable problems associated with their realization, their labor-consumption and the necessity of using complicated equipment, the number of such measurements [1], [3] carried out on highway bridges in the world is rather small. For this reason we decided to carry out our own investigations that covered five steel highway bridges [4] situated on international heavy traffic routes. The investigated bridges differed in their static scheme, their structure, the number and the span of the main girders and the situation and the type of the deck. On all the bridges, measurements of unit strains and simultaneous recording of traffic volume and its composition were conducted. On the basis of the obtained results, the effect of real loads on the operational strength of different elements of the highway bridges was determined.

Due to the high labor-consumption and the high costs of operational field investigations it would have been very hard to carry out a sufficiently great number of them in order to obtain representative results for all the different bridge elements. Therefore it seemed advisable to apply simulation methods [2], [5] and to use the experience gained from the investigations carried out on the real objects. For this purpose a computer program called TE-MD (operational durability of highway bridges) was developed. Because of the complexity of the considered problem, additional subprograms LW and MOMN that together with TE-MD form one system were developed. A deterministic model of the operational loads of bridge objects situated on the main roads in Poland determined on the basis of the author's measurements of the traffic volume and its composition on bridges (inside and outside cities) and the technical specifications of the different types of trucks that move on Polish roads formed a base for the calculations that were carried out with the help of the above program. Moreover, a random model of traffic on a bridge [4] which takes into account, among other things, the effect of simultaneous passing of vehicles over the bridge on the obtained static values and numbers of cycles was used in the calculations.

The TE-MD program can be used for the determination of stress spectra and operational durability parameters of different types of bridges of any static schemes, spans, cross-sections, any number of traffic lanes and at any number of considered vehicles, any traffic volumes and speeds. The basic strength and operational durability parameters obtained by computer simulation and the graphic relationships between them are shown in fig. 1.

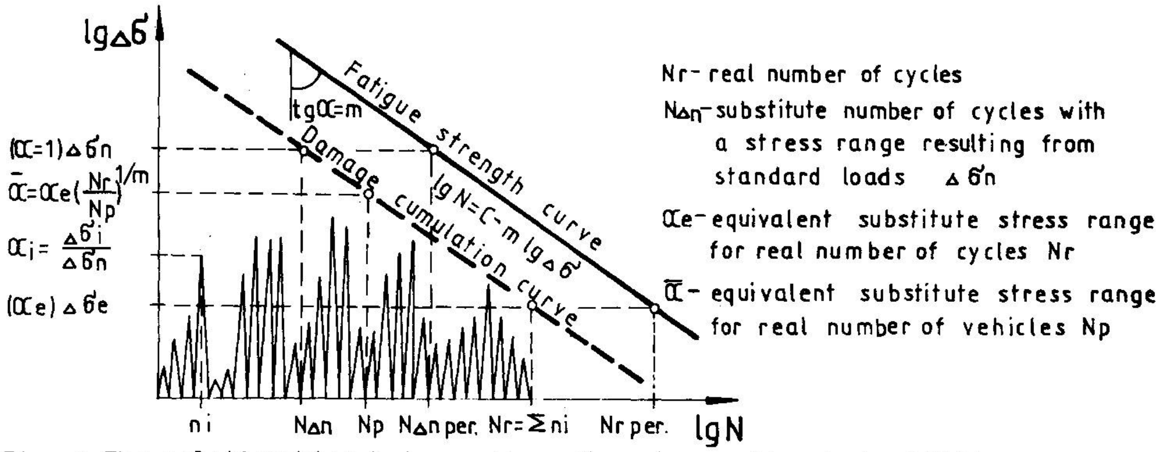

Fig. 1 The relationships between strength and operational durability parameters

Using expression

$$T = \frac{10^{a}}{(\sigma_n)^m \cdot 365 \cdot N_n^d} \qquad (1)$$

after the substitution of values N_n^d (N_n for 24 hours) determined by simulation as well as fatigue strength curve parameters a and m, operational durability T (in years) of a considered bridge element can be determined for standard stress σ_n and for the type of the notch that occurs in it.

Fig.2 shows exemplary diagrams of operational durability T (in years) determined for the longitudinal ribs of the deck at cross-bars flexibility γ = 0,05 depending on: a) traffic volume per 24 hours N_p^d , b) type of notch, c) design standard stress σ_n.

After transformation of formula (1) the relationship for permissible stress from moving standard loads at which the fatigue hazard will be avoided assumes the form

$$\sigma_{n,per.} = \left[\frac{10^{a}}{T \cdot 365 \cdot N_n^d}\right]^{1/m} \qquad (2)$$

3. AN EXAMPLE OF BRIDGE DESIGNING INVOLVING OPERATIONAL DURABILITY

In order to illustrate the described method of designing bridge elements that takes into account operational durability, an example of the determination of permissible stresses due to standard loads is presented.

3.1. Data

A longitudinal rib the orthotropic plate of a two-lane higway bridge where the longitudinal cross-bar spacing is t=3,50 m and cross-bar flexibility γ =0,05 is designed.

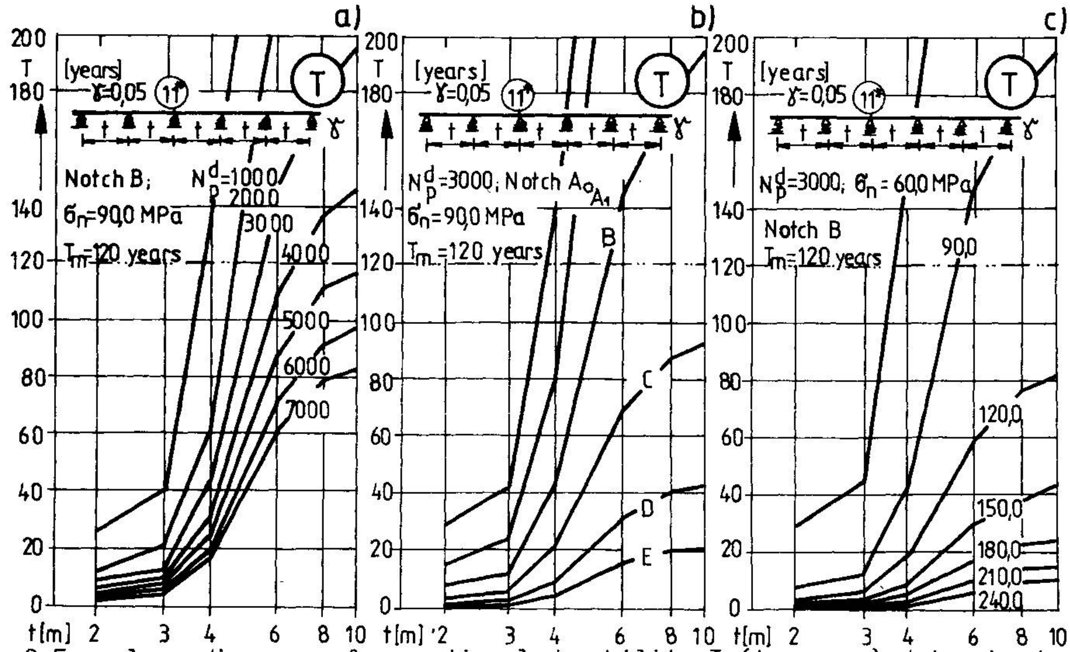

Fig. 2 Exemplary diagrams of operational durability T (in years) determined for the longitudinal ribs of the deck at cross-bars flexibility γ=0.05 depending on: a) traffic volume per 24 hours N_p^d, b) type of notch, c) design standard stress σ_n

Determine the permissible values of stress resulting from standard moving loads $\sigma_{n,per.}$ which can be adopted for the lower fibers of the span section of the rib without fatigue hazard in the case of notch B (without transverse welds) and C (with transverse welds) under assumed durability T=120 years and 24 hours volume of trucks traffic N_p^d=3500.

3.2 Determination of Permissible Standard Stress

Using the diagrams of equivalent substitute cycle numbers N_n^d [4] for the considered longitudinal rib at N_p^d = 3500 and t = 3,50 m, value N_n^d = 78,4 is determined. Then, from formula (2) the permissible standard stress for notch B is calculated

$$\sigma_{n,per.} = \left(\frac{10^a}{T\cdot 365\cdot N_n^d}\right)^{1/m} = \left(\frac{10^{12.48}}{120\cdot 365\cdot 78.4}\right)^{1/3.0} = 95.81\,\text{MPa} \quad (3)$$

and for notch C

$$\sigma_{n,per.} = \left(\frac{10^{12.16}}{120\cdot 365\cdot 78.4}\right)^{1/3.0} = 74.94\ \text{MPa} \quad (4)$$

Similar values of permissible standard stress can be determined graphically using a special nomogram [4] and appropriate curves N_n^d. For the above data, the values read from fig.3 are $\sigma_{n,per.}$ = 98,0 MPa for notch B and 77,0 MPa for notch C.

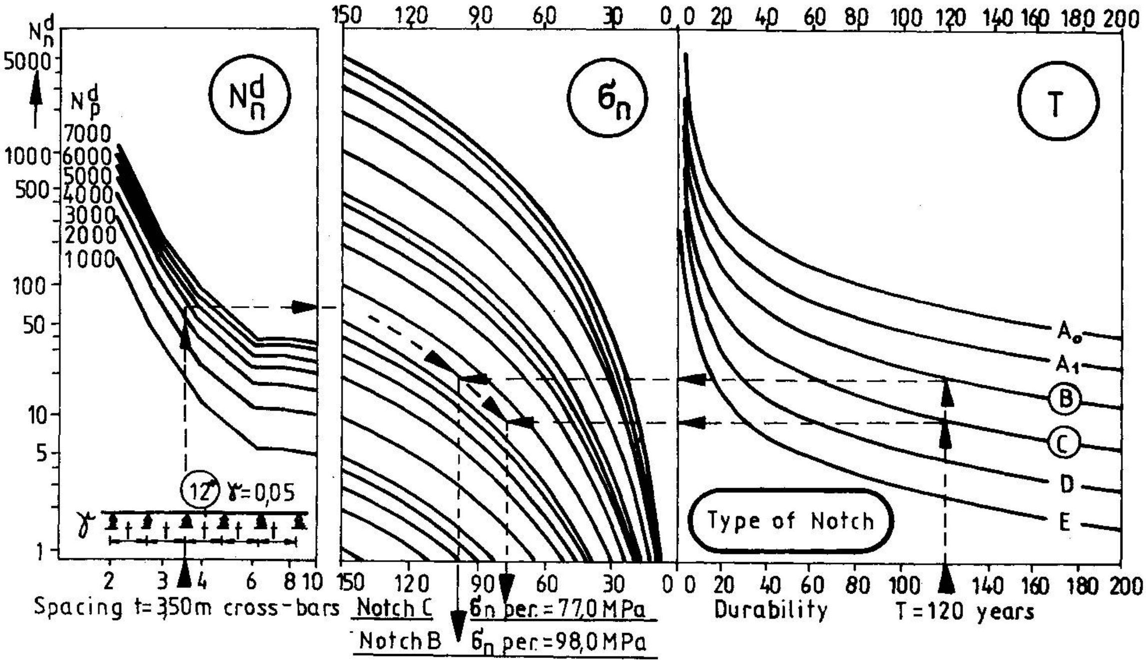

Fig. 3

4. FINAL REMARKS

In spite of the necessarily concise treatment of the problem, it is evident that the application of computer simulation verified by field studies resulted in the development of a relatively simple design method of steel highway bridges that takes into account operational durability.

On the basis of the experience gained from the practical use of this method, the following recommendations aimed at the prolongation of the operational life of to-be-designed steel bridge spans can be formulated:

- unfavorable structural notches should be avoided, particularly in deck elements and in span sections of the main beams characterized by a continuous static scheme,
- for the above elements at unfavorable notch coefficients, the design standard stress should be reduced appropriately (increase the cross-section) which will prolong their assumed operational life considerably (fig. 2c),
- for fatigue reasons, one should avoid main girders with a three-span static scheme having all the spans of equal length and aim at a situation where the length of the middle span in this scheme will be greater than that of the terminal spans,
- grades of high strength steel should, due to their susceptibility to fatigue, be used only in elements with small notch coefficients.

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Fatigue Strength and Behavior of Prestressed Concrete Bridge Girders

Résistance et comportement à la fatigue des poutres de pont en béton précontraint

Ermüdungsfestigkeit und Verhalten von Spannbetonbrückenträgern

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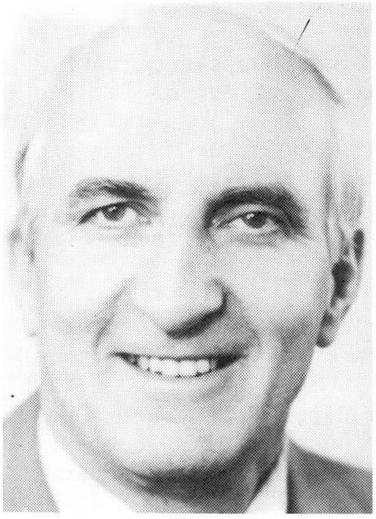

John Breen, born in 1932, earned the PhD. at the University of Texas, and is the past Chairman of the ACI Building Code Committee.

Basile Rabbat, born in 1944, earned the PhD. at the University of Toronto, and is present Secretary of the ACI Committee on the Building Code.

SUMMARY

This paper constitutes a summary of selected research investigations related to the strength and behavior of prestressed concrete bridge girders. The investigations selected were inter-related and continuous over a twenty-year period. The investigations address aspects of strength and behavior related to flexure and fatigue, with fatigue emphasized in the paper.

RÉSUMÉ

Cet article présente un résumé des travaux de recherche ayant rapport à la résistance et au comportement des poutres de pont en béton. Les études choisies sont liées les unes aux autres et représentent un effort continu sur une période de vingt ans. Elles traitent certains aspects de la résistance et du comportement des gaines de précontrainte, du comportement en flexion et de la résistance à la fatigue.

ZUSAMMENFASSUNG

Dieser Artikel ist eine Zusammenfassung ausgesuchter Forschungsberichte, die sich mit der Baustoffestigkeit und dem Verhalten von Spannbetonbrückenträgern befassen. Die inhaltlich zusammenhängenden Beiträge behandeln einen Zeitraum von zwanzig Jahren. Sie behandeln Aspekte der Baustoffestigkeit und des Verhaltens bezüglich Biegewiderstand und vor allem Ermüdungsfestigkeit.

1. INTRODUCTION

This paper is a summary of selected research investigations related to the strength and behavior of pretensioned concrete bridge girders. The investigations selected were all inter-related and continuous over a twenty year time span. The investigations address strength and behavior related to strand blanketing, shear behavior under static load, flexural behavior and fatigue strength and behavior. The main focus of the paper is on fatigue strength and behavior.

Emphasis is placed on recent tests at the University of Texas. The overall objectives of the Texas project were:

1) To re-examine the fatigue resistance of typical pretensioned concrete girders with emphasis on the effect of fatigue on the strength of the girders.

2) To determine the effect of level of tension stress in the precompressed tensile zone on the capability of pretensioned girders to withstand traffic loading without strand fatigue during their design life.

3) To re-examine the approximate stress range to be used in design of prestressed concrete members to avoid fatigue failure in strands.

The main objective of this paper is to re-evaluate the recent research relative to its contribution and continuity of research previously conducted under sponsorship of the Louisiana Department of Transportation and Development.

The research and tests which are reviewed in this paper are:

1) Portland Cement Association tests reported in 1965 related to strand blanketing on half scale girders.

2) Tulane University tests reported in 1975 related to strand blanketing of half scale and full size girders.

3) Portland Cement Association tests reported in 1980 related to strand blanketing and fatigue tests of full size girders.

4) University of Texas tests reported in 1984 related to fatigue strength and behavior of full size girders.

The paper indicates that the Texas research provided continuity and served to expand previous research: particularly in those areas including the influence of stress losses, strand stress ranges, nominal tensile stresses, overloads and passive reinforcement.

2. EVALUATION OF RESEARCH EFFORT

2.1 Introductory Remarks

The main objective of this paper is to integrate the results of research conducted by the University of Texas for the Texas Department of Highways and Public Transportation and the Federal Highway Administration, with related research previously conducted under Louisiana Department of Transportation and

Development sponsorship. The evaluation is concerned mainly with the strength and behavior of full-size pretensioned prestressed concrete girders tested under conditions of static and cyclic flexural loading.

2.2 Tests Prior to 1980

The tests sponsored by the Louisiana Department of Transportation and Development and conducted at Tulane University were reported in 1975. They were directed at the elimination of draped strands in prestressed concrete girders through the use of strand blanketing techniques, and were based in part on previous tests at the Portland Cement Association. The results of the Tulane tests indicated that the unbonding of prestressed strands through the use of blanketing techniques was an effective method that could be used as a means of eliminating draped strands in prestressed concrete girders. This conclusion was limited to the condition of static loading in flexure. Because the tests related to the blanketing of strands had not included fatigue considerations, a new investigation was initiated at the Portland Cement Association for the Louisiana DOTD and the Federal Highway Administration.

The investigation at the Portland Cement Association reported in 1979, was to determine the effects of the elimination of draped strands in full-size prestressed concrete girders. Specifically, draped strands were eliminated by using straight strands with unbonded lengths at their ends, thus creating "blanketed strands." The effect of blanketing on the strength and behavior of prestressed concrete girders under repeated loading was the thrust of the investigation. Based on comparative fatigue tests of full-size Type II AASHTO-PCI girders, it was shown that, to control stresses in the end regions of pretensioned bridge members, straight strands having unbonded blanketed lengths at the ends of girders can be used effectively and economically as an alternative to draped strands. The PCA tests and conclusions had the effect of expanding the Tulane tests and conclusions into the range of fatigue loading, insofar as blanketed strands were the issue. However, the PCA tests also indicated that fatigue of strands may be an important consideration in prestressed girders designed according to Codes where a nominal maximum concrete tensile stress of 0.50 $\sqrt{f'c}$ MPa as calculated assuming an uncracked section is permitted under service loads. The PCA test program called for 5 million cycles of loading between dead load and dead load plus live load. These girders tested at a nominal maximum concrete tensile stress of 0.50 $\sqrt{f'c}$ MPa failed before the 5 million cycles were reached. All girders had artificial crack formers so that they were not uncracked under repeated loads. Premature fatigue fractures were responsible in part for the PCA recommendation that "further research is needed to determine the fatigue properties of prestressing strands, as well as the level of tension in the concrete at which pretensioned girders would be able to withstand traffic loading without strand fatigue during their design service life." Thus the PCA tests confirmed and extended the conclusions of the Tulane tests relative to blanketed strands. However, the PCA tests indicated that further research was needed relative to fatigue strength and behavior of prestressing strands and pretensioned girders.

The Bridge Committee of the Prestressed Concrete Institute reviewed the PCA report and commented as follows:

> "1. Results of this one series of tests should not be used as a basis for making radical changes in current design criteria that are based on numerous other tests. Consideration should be given to the large number of cycles of full load that were required to cause a strand fatigue

failure, the details of the tests which were not designed to evaluate strand fatigue properties and to artificially formed cracks at which the failures occurred.

2. Possible significance of the test results has been studied and an investigation of fatigue in prestressed concrete members designed to current criteria is being considered by the AASHTO Subcommittee on Bridges and Structures."

The Texas investigation was initiated as a result of potentially unfavorable results found in the PCA tests.

2.3 Tests After 1980

The tests conducted at the University of Texas and reported in 1984 were for the purpose of determining the fatigue strength of full-scale pretensioned concrete bridge girders. Blanketing of strands was not a consideration in the Texas investigation and was not included in the test program; the focus of the program was fatigue strength and behavior. In addition, the Texas tests provided an opportunity for response to comments offered by the Bridge Committee of the Prestressed Concrete Institute. The Texas study included a comprehensive investigation of fatigue of prestressing strands. The results of the strand study have been reported in detail, and are not addressed in this report. The Texas investigation further provided the opportunity for participation by the Louisiana DOTD through the inclusion of production AASHTO-PCI specimens fabricated in Louisiana, in addition to the Texas Type C girders; thus continuing the direct involvement of the Louisiana DOTD.

In evaluating the Texas research effort relative to its contribution and continuity to research previously conducted under the Louisiana DOTD sponsorship, differences between the AASHTO-PCI girders and the Texas Type C girders tested should be noted. The AASHTO-PCI girders were production line girders fabricated in a prestressing plant. They were steam cured, and were not retensioned after the first full tensioning operation. Plant fabrication procedures were essentially the same for all AASHTO-PCI girders tested in the Tulane, PCA and Texas investigation. The Texas Type C girders tested were not steam cured. Before placing the concrete, however, the strands were retensioned to the desired stress in order to reduce relaxation losses which occurred during the period in which the steel was tied and the forms were set. These two effects, the retensioning and the lack of steam curing, could reduce the relaxation component of the stress losses.

In any event, the effective prestress force was not known precisely, but was closely estimated from measurements of decompression moments as well as from an analytical model. Variations in prestress losses can substantially affect strand stress ranges and hence fatigue life. Because of differences in fabrication procedures and because of other variables including geometry, the tests of the AASHTO-PCI girders and the tests of the Texas Type C girders are, to some extent, evaluated separately with respect to their contribution and continuity to previous tests.

In general, results of the Texas study indicated that present AASHTO indirect design criteria of flexural fatigue strength of pretensioned concrete girders, through limitation of the nominal tensile stress in the precompressed tensile zone, will not ensure adequate fatigue life. The Texas study further indicated that pretensioned concrete bridge girders without well distributed confined passive reinforcement, which are actually subjected to loads producing nominal tensile stresses of 0.50 $\sqrt{f'c}$ MPa can fail as a result of fatigue.

AASHTO specifications allow 0.50 $\sqrt{f'c}$ MPa tension in the extreme fibers of the precompressed tension zone of prestressed concrete flexural members. It has been implicity assumed that fatigue failure would not occur at this design level for a significant number of cycles. The three AASHTO-PCI girders loaded to a nominal concrete tensile stress of 0.50 $\sqrt{f'c}$ MPa in the PCA tests failed at 3.63, 3.78, and 3.20 million cycles, respectively. The three AASHTO-PCI girders included in the Texas tests were loaded to maximum nominal concrete tensile stresses of 0.52, 0.52, and 0.29 $\sqrt{f'c}$ MPa, respectively. The first girders failed at 2.84 million cycles, the second deteriorated rapidly after 4.50 million cycles, and the third girder (at 0.29 $\sqrt{f'c}$ MPa) did not fail.

The main variable between the two AASHTO-PCI specimens tested at 0.52 $\sqrt{f'c}$ MPa was the presence of a few modest static overloads. It was indicated that very occasional overloads (in this case 20 percent above the fatigue load) can drastically reduce fatigue life. The specimen that failed at 2.84 million cycles was subject to overload; the specimen that deteriorated rapidly at 4.5 million cycles was not subject to overload. In addition, one girder was precracked while the other was left to crack naturally under cyclic loads. It was found that an uncracked girder will crack after only about 1000 cycles of loading to a nominal tensile stress of 0.52 $\sqrt{f'c}$ MPa.

Three of the Texas Type C girders were loaded to a tensile stress range comparable to the 0.50 $\sqrt{f'c}$ MPa. One girder was loaded to a tensile stress of 0.50 $\sqrt{f'c}$ MPa and subsequently failed in fatigue at 1.91 million cycles. The second girder was loaded to a tensile stress of 0.46 $\sqrt{f'c}$ MPa and subsequently failed in fatigue at 2.29 million cycles. The third girder had 1290 sq mm of confined passive reinforcing steel in the lower flange to control cracking, was loaded to a tensile stress of 0.46 $\sqrt{f'c}$ MPa, and subsequently failed in fatigue at 9.43 million cycles. The small amount of well distributed conventional reinforcement was credited with greatly extending the fatigue life, and was very instrumental in reducing creep related prestress losses.

A tabulation of the three AASHTO-PCI girders included in the Texas tests, and the three Texas Type C girders loaded to an approximate tensile stress of 0.50 $\sqrt{f'c}$ MPa is shown below. Included in the tabulation are the three AASHTO-PCI girders tested at PCA with a nominal tensile stress of 0.50 $\sqrt{f'c}$ MPa.

The extended life of specimen No. 9 has been attributed to a small amount of well distributed conventional reinforcement. Specimens No. 7 and No. 8 tend to confirm the results of specimen No. 1, No. 2, and No. 3, AASHTO-PCI girders tested at PCA. Specimen No. 6, an AASHTO-PCI girder, was loaded to a tensile stress level of 0.29 $\sqrt{f'c}$ MPa and did not fail.

Specimen	Maximum Nominal Concrete Tensile Stress During Fatigue Loading	Fatigue Life
	$\sqrt{f'c}$ MPa	millions
1. PCA G-10	0.50	3.63
2. PCA G-11	0.50	3.78
3. PCA G-13	0.50	3.20
4. A22 - 2.84	0.52	2.84
5. A22 - 5.00	0.52	4.50
6. A22 - 5.95	0.29	5.95(NF)
7. C16 - 1.91	0.50	1.91
8. C14 - 2.29	0.46	2.29
9. C16 - 9.43	0.46	9.43

The results for specimen No. 4 and specimen No. 5 (identical AASHTO-PCI specimens) differed, with the reduced life of specimen No. 4 attributed to the intentional occasional overloads.

The fatigue life of specimen No. 5 was greater than similar AASHTO-PCI girders tested at PCA under compatible levels of nominal concrete tensile stress. However, the nominal tensile stress levels used in the PCA tests were based on assumed stress losses of 20 percent, whereas the Texas study indicates that the measured stress losses for the AASHTO-PCI girders is approximately 13 percent. Stress losses less than those assumed would result in smaller nominal tensile stresses with an expected increase in fatigue life.

Specimen No. 7 and specimen No. 8 were loaded respectively to nominal tensile stress levels of 0.50 and 0.46 $\sqrt{f'c}$ MPa, and experienced corresponding lives of 1.91 and 2.29 million cycles.

Based on the above discussion, it is considered that general compatibility exists in comparing the fatigue life of the three AASHTO-PCI specimens tested at PCA, and those specimens in the Texas research loaded to the same nominal tensile stress.

3. SUMMARY

It is felt that the Texas research effort has contributed significantly and has provided continuity to the research previously conducted by the Louisiana DOTD. The Texas research effort broadened previous investigations through more detailed study of stress losses and the influence of stress losses on fatigue life; through inclusion of nominal concrete tensile stresses, during fatigue loading, that ranged from 0.29 to 0.88 $\sqrt{f'c}$ MPa; through consideration of the effect of occasional overloads; and through study of the effects of the addition of passive reinforcement as a means of extending fatigue life.

The Texas research effort directly addressed issues tabulated in the PCA report and directly related to fatigue life. The Texas report indicates that fatigue failures will occur at and below the AASHTO design limit of 0.50 $\sqrt{f'c}$ MPa and can occur at 0.50 $\sqrt{f'c}$ MPa at less than 2 million cycles.

The report also indicated that at a maximum nominal tensile stress of approximately 0.46 $\sqrt{f'c}$ MPa one specimen failed after 2.29 million cycles and another failed after 9.43 million cycles. This is approximately a fourfold difference and indicates that this design parameter is inappropriate.

In any event, the Texas report indicates that if a nominal tensile stress limit is used to implicity guard against fatigue failure, the limit should be 0.25 $\sqrt{f'c}$ MPa in the absence of adequate, well distributed and well confined passive reinforcement. The Texas report recommends that when fatigue is considered important, design for fatigue of pretensioned girders be based on the stress range determined from a cracked section analysis and the lower bound S-N curve appropriate for the strand.

Ermüdungsbemessung im Spannbetonbau

Fatigue Design of Prestressed Concrete Members

Dimensionnement à la fatigue de constructions en béton précontraint

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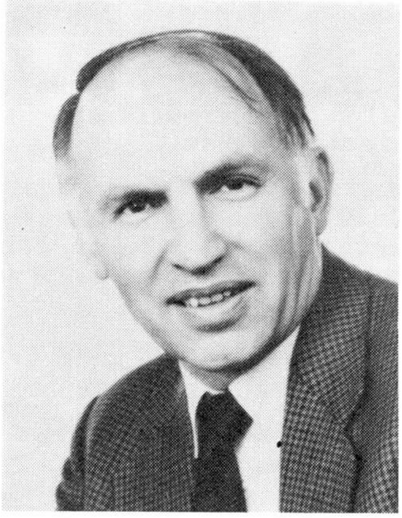

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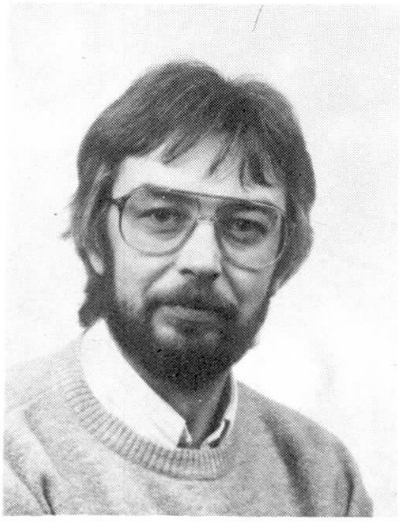


Geboren 1954, Studium des Bauingenieurwesens an der TH Darmstadt, 1981 Diplom. Bis 1985 Mitarbeiter in der Techn. Abteilung einer Baufirma, anschliessend wissenschaftl. Assistent bei Prof. König.

ZUSAMMENFASSUNG

Der Beitrag erläutert einige Aspekte, die bei teilweise vorgespannten Konstruktionen hinsichtlich der Ermüdung berücksichtigt werden müssen. Weiterhin wird das für den MC 90 vorgeschlagene Konzept für die Ermüdungsbemessung vorgestellt.

SUMMARY

Some aspects concerning the fatigue behaviour of partially prestressed members are briefly described. The concept of fatigue design proposed for MC 90 is introduced.

RÉSUMÉ

L'article présente quelques phénomènes de fatigue dans le béton partiellement précontraint. Le concept de dimensionnement à la fatigue pour le code MC 90 est présenté.

1. EINLEITUNG

In der Vergangenheit wurde bei der Bemessung von vorgespannten Konstruktionen der Ermüdung nur wenig Bedeutung beigemessen, da der Vorspanngrad in der Regel so gewählt wurde, daß die Konstruktion im Zustand I verbleibt. Eine Ermüdungsbemessung wurde lediglich für die Verankerungen und Kopplungen der Spannglieder für notwendig erachtet, da an diesen Stellen die Ermüdungsfestigkeit deutlich abfällt gegenüber derjenigen des Spannglieds auf freier Strecke.

Untersuchungen in den letzten Jahren zeigten jedoch, daß nicht nur Verankerungen und Kopplungen ermüdungsgefährdet sein können, sondern auch die Spannglieder selbst.

Dieser Beitrag erläutert in kurzer Form zunächst einige Aspekte, die insbesondere bei teilweise vorgespannten Konstruktionen zu berücksichtigen sind. Anschließend erfolgt eine kurze Beschreibung des für den MC 90 vorgeschlagenen Nachweiskonzeptes.

2. EINFLUSS DES VORSPANNGRADES AUF DIE ERMÜDUNGSBEANSPRUCHUNG

Der prinzipielle Zusammenhang zwischen Vorspanngrad und Ermüdungsbeanspruchung ist in Bild 1 dargestellt. Es verdeutlicht, daß bei vorgespannten Konstruktionen, die unter Gebrauchslasten den Zustand II erreichen, die Schwingbreite der Spannungen deutlich größer wird als bei solchen Konstruktionen die im Zustand I verbleiben.

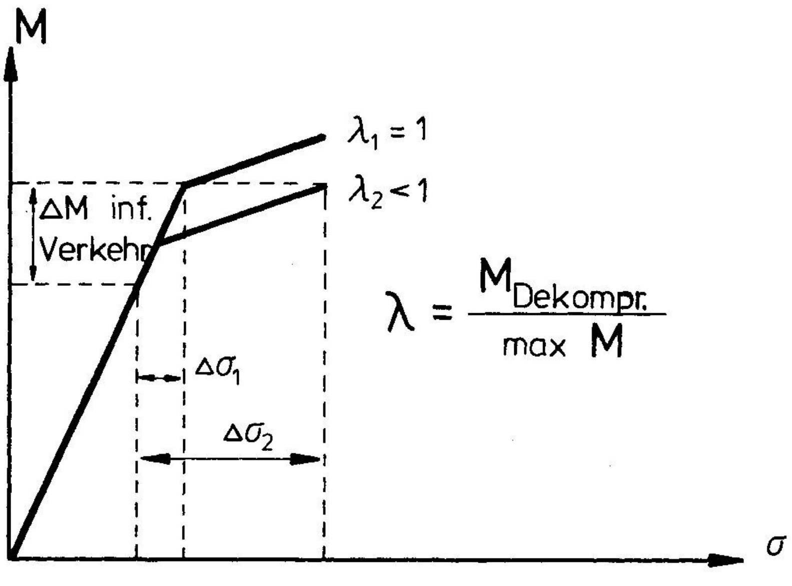


Bild 1: Zusammenhang zwischen Vorspanngrad λ und Ermüdungsbeanspruchung

Dies ist jedoch nicht nur bei planmäßiger teilweiser Vorspannung der Fall. Viele Konstruktionen die nach den Regeln der beschränkten oder vollen Vorspannung bemessen wurden, weisen infolge falscher Einschätzung der Temperatureinflüsse, der Eigenspannungen oder der Setzungen Risse auf.

3. REIBKORROSION

Das Ermüdungsverhalten von Spanngliedern in teilweise vorgespannten Konstruktionen wurde in den letzten Jahren im Rahmen mehrerer Forschungsvorhaben untersucht (1),(2),(3). Die Ergebnisse zeigen eindeutig, daß die Ermüdungsfestigkeit durch Reibkorrosion deutlich reduziert wird. Reibkorrosion kann an solchen Stellen auftreten, wo zwei metallische Partner hohem Querdruck ausgesetzt sind und sich relativ zueinander bewegen.

In teilweise vorgespannten Konstruktionen bilden Spannstahl und metallisches Hüllrohr die beiden Partner, der hohe Querdruck entsteht in den Bereichen mit großen Spanngliedkrümmungen. Die Relativbewegung erfolgt durch das Öffnen und Schließen der Risse in diesen Bereichen.

Die Versuchsergebnisse (Tabelle 1) zeigen, daß Reibkorrosion die Ermüdungsfestigkeit bis zu 50% reduzieren kann.

Spannstahlsorte	Dauerschwingfestigkeit für $\sigma_0 = 0,55 \cdot \beta_z$ nach Zulassung	nach Versuchen mit freien Proben	nach Versuchen mit Reibdauerbeanspruchung	bezogene Dauerschwingfestigkeit	zulässige Schwingbreite nach DIN 4227 Teil 2
	$2 \cdot \sigma_{aZ}$	$2 \cdot \sigma_{aF}$	$2 \cdot \sigma_{aR}$	σ_{aR}/σ_{aF}	$0,4 \cdot 2 \cdot \sigma_{aZ} \leq 140$
1	2	3	4	5	6
	$[N/mm^2]$	$[N/mm^2]$	$[N/mm^2]$	[-]	$[N/mm^2]$
St 1080/1230; ø26,5 mm rund, gerippt	240	285	Mü: 200 Aa: -	0,70	96
St 1420/1570; ø12,2 mm vergütet, rund, glatt	340	390	Mü: 175 Aa: 170	0,44	136
St 1470/1670; ø7,0 mm kaltgezogen, rund, glatt	585	350	Mü: - Aa: 160	- 0,46	140
St 1570/1770; ø15,3 mm Spannstahllitze	260	250	Mü: 150 Aa: 170	0,64	104

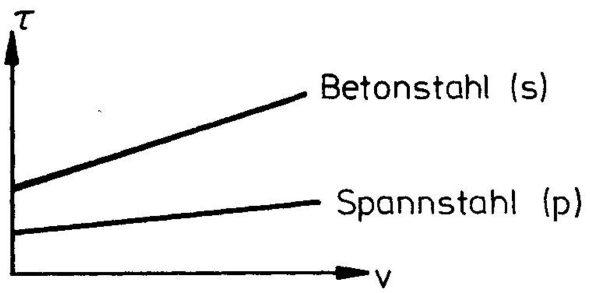
Mü: Ergebnisse von Versuchen an der TU München
Aa: Ergebnisse von Versuchen an der RWTH Aachen

Tabelle 1: Versuchsergebnisse (1),(2)

In einigen Versuchen (3) wurden anstatt der üblichen metallischen Hüllrohre, Kunststoffhüllrohre verwendet, wodurch das Ermüdungsverhalten deutlich verbessert werden konnte.

4. SPANNUNGSUMLAGERUNG BEI GEMISCHTER BEWEHRUNG

Üblicherweise werden die Spannungen in einem gerissenen Querschnitt mit der Annahme ermittelt, daß die Dehnungen sich proportional zum Abstand von der Nullinie verhalten. Dies trifft bei gemischter Bewehrung in der Regel nicht zu. Der Spannstahl mit seinen meist schlechten Verbundeigenschaften erfährt im Riss geringere Dehnungen als ein auf gleicher Höhe liegender Betonstabstahl (Bild 2).



Beziehung zwischen Verbund-spannung (τ) und Schlupf (v)

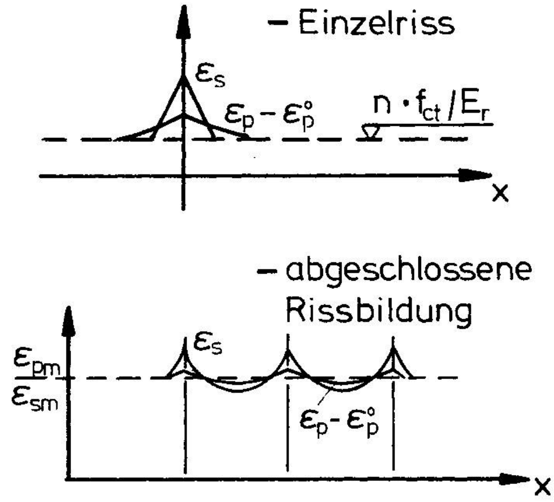
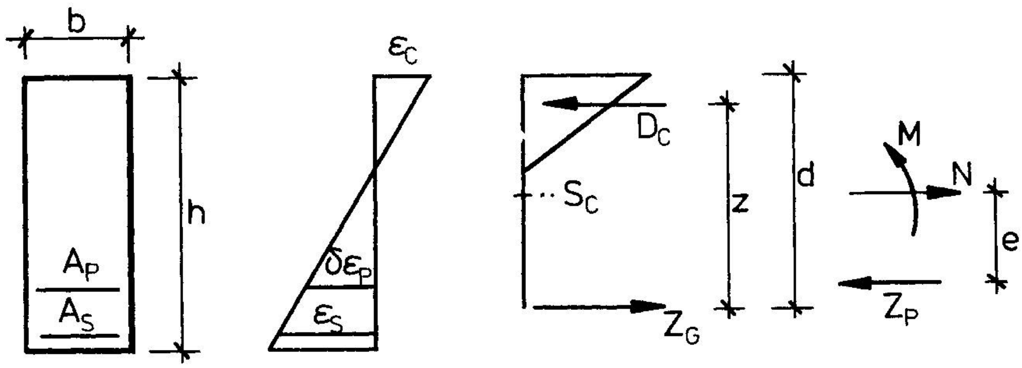


Bild 2: Zusammenwirkung von Betonstabstahl und Spannstahl bei Rissbildung (4)

Bei teilweiser Vorspannung können die tatsächlich auftretenden Spannungen wie folgt ermittelt werden:

Zunächst muß die gesamte Zuggurtkraft Z_G im Schwerpunkt der Stahleinlagen bestimmt werden, wobei die Betonzugfestigkeit zu vernachlässigen ist (Bild 3).



$$Z_G = \sigma_{SR} \cdot A_S + \delta\sigma_{PR} \cdot A_P$$

Bild 3 : Ermittlung der Zuggurtkraft Z_G am gerissenen Querschnitt

Aus dieser Zuggurtkraft ergeben sich dann die Spannungen im Betonstabstahl σ_{SR} und der Spannungszuwachs im Spannstahl $\delta\sigma_{PR}$ zu:

$$\sigma_{SR} = \frac{Z_G}{A_S + A_P} + \frac{0.3 \cdot f_{ct} \cdot A_P}{A_S + \xi_1 \cdot A_P} \cdot \frac{1 - \xi_1}{A_S + A_P} \cdot A_{b,eff} \qquad (1)$$

$$\delta\sigma_{PR} = \frac{Z_G}{A_S + A_P} + \frac{0.3 \cdot f_{ct} \cdot A_S}{A_S + \xi_1 \cdot A_P} \cdot \frac{\xi_1 - 1}{A_S + A_P} \cdot A_{b,eff} \qquad (2)$$

mit f_{ct} = Betonzugfestigkeit
ξ_1 = $\xi \cdot (d_s/d_P)$
$A_{b,eff}$ = $3 \cdot b \cdot (h-d)$
d_s = Durchmesser des Betonstahls
d_P = Durchmesser des Spannstahls
(für Spannstahlbündel muß ein Ersatzdurchmesser gewählt werden: $1.6\sqrt{A_v}$ mit A_v = Fläche des Spannstahlbündels)

ξ = 0,2 für glatte Spannstähle
ξ = 0,4 für Litzen
ξ = 0,6 für profilierte Spannstähle
} für Vorspannung mit nachträglichem Verbund

ξ = 0,6 für Litzen
ξ = 0,8 für profilierte Spannstähle
} für Vorspannung mit sofortigem Verbund

Bei vollkommen gleichen Verbundeigenschaften von Betonstahl und Spannstahl (ξ_1=1 in Gl. 1 u. 2) erfährt der Spannstahl keine Entlastung. Dies kann in Spanngliedkrümmungen der Fall sein, wenn der Querdruck für hohe Reibungskräfte sorgt.

5. ERMÜDUNGSBEMESSUNG – KONZEPT DES MC 90

Der Vorschlag sieht drei verschiedene Möglichkeiten vor, die im folgenden für den Stahl beschrieben werden. Als Ausgangswert für die Bemessung wird in allen 3 Fällen die unter Gebrauchslasten ermittelte maximale Schwingbreite $\Delta\sigma$ benutzt. Sie ergibt sich aus 0.9 bzw. 1,1 facher Vorspannung (je nach günstiger oder ungünstiger Wirkung der Vorspannung), aus den ständigen Lasten (wirksamer Temperatureinfluß eingeschlossen) und dem ermüdungswirksamen Verkehrslastanteil.

5.1 Dauerfestigkeitsnachweis

Ein detaillierter Ermüdungsnachweis muß nicht geführt werden, wenn folgende Bedingung eingehalten ist:

$$\max \Delta\sigma \cdot \gamma_{Sd} \leq \Delta\sigma_R / \gamma_M$$

wobei: $\max \Delta\sigma$: maximale Schwingbreite der Stahlspannungen unter Gebrauchslasten
$\Delta\sigma_R$: vorgegebene Werte für die Dauerfestigkeit
γ_{Sd}, γ_M : Teilsicherheitsbeiwerte für Last bzw. Festigkeit

5.2 Zeitfestigkeitsnachweis

Dieser Nachweis berücksichtigt die angestrebte Lebensdauer und ist dadurch etwas präziser als der Dauerfestigkeitsnachweis. Für die einzelnen Materialien und Verbindungen werden charakteristische Wöhlerlinien vorgegeben, aus denen für eine gegebene Lastwechselzahl n die ertragbare Schwingbreite $\Delta\sigma_R(n)$ ermittelt werden kann. Es ist nachzuweisen daß:

$$\max \Delta\sigma \cdot \gamma_{Sd} \leq \Delta\sigma_R(n) \;/\; \gamma_M$$

5.3 Betriebsfestigkeitsnachweis

Der Betriebsfestigkeitsnachweis berücksichtigt neben der angestrebten Lebensdauer auch das tatsächlich einwirkende Lastspektrum. Ausgehend von der Palmgren–Miner Hypothese wird eine Schädigung D ermittelt und mit einem zulässigen Wert verglichen:

$$D = \sum_i \frac{n_{Sdi}}{n_{Rdi}} \leq D_{lim}$$

Bei diesem Nachweis wird die Belastung in verschiedene Klassen i eingeteilt mit den jeweiligen Lastwechselzahlen n_{Sdi}. Aus der Spannungsschwingbreite jeder Klasse ($\Delta\sigma_i$) wird dann über die Wöhlerlinie unter Berücksichtigung der Teilsicherheitsbeiwerte die zugehörige ertragbare Lastwechselzahl n_{Rdi} ermittelt.

Es ist jedoch möglich, durch entsprechende Aufbereitung der Daten, Bemessungshilfen zu erstellen, die eine einfache Durchführung des Nachweises erlauben. Für Kranbahnträger ist eine derartige Aufbereitung bereits erfolgt (5),(6), für Straßenbrücken wird sie derzeit im Rahmen eines Forschungsvorhabens an der TH Darmstadt durchgeführt.

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Synergetic Effects of Environment Actions and Fatigue

Effets synergétiques des influences extérieures et de la fatigue

Synergetische Auswirkungen von Umgebungseinfüssen und der Ermüdung

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SUMMARY

As far as reinforced concrete and prestressed concrete bridges are concerned, the importance of fatigue damage is progressively increasing. In the present paper stochastic action effects are considered together with experimental and theoretical methods for their evaluation. The methods for the assessment of safety for durability are discussed. The influence of corrosion on the laws for constitutive materials is properly taken into account.

RÉSUMÉ

Dans les ponts en béton armé et/ou précontraint, l'importance des dommages dûs à la fatigue est en progression. Les effets des actions stochastiques de même que les méthodes expérimentales et théoriques sont étudiées. L'article traite les méthodes de sécurité, tenant compte d'une manière appropriée de l'influence de la corrosion sur les lois des matériaux constitutifs.

ZUSAMMENFASSUNG

Es wird eine gleichzeitige Zunahme der Schadenfälle durch Ermüdung sowohl bei schlaff bewehrten als auch bei vorgespannten Brücken festgestellt. Sowohl die Betriebsbeanspruchung infolge Zufallslasten als auch die experiementellen und theoretischen Methoden ihrer Auswertung werden betrachtet. Weiter werden die Methoden zur Sicherheitsanalsyse diskutiert; der Einfluss der Korrosion der Materialien wird zweckmässig abgewogen.

1. INTRODUCTION

The present work is part of a systematic research, coordinated between the Universities of Pavia and Rome, devoted to the study of damage and reliability of bridges during their service lives.
First results of the research, limitedly to the case of materials damage due to repeated loads by means of nominal "stress spectra" were presented at the 25th Plenary Session of CEB in Treviso [4]. the research continued on one hand with the execution of field measurements of the response of an highway bridge, called "Pecora Vecchia", to directly get the "stress spectra", on the other taking into account other concurrent sources of damage.
The investigation is here extended to the study of environmental effect over the duration of service life, with special reference to the interaction between reinforcement's corrosion and fatigue strength. A calculation procedure is presented which fits in the philosophy of the "Generalized Design Space", proposed by Tassios [6].

2. STATEMENT OF THE PROBLEM

The durability of a given structure comes from the stability of thermodynamic state functions of its constitutive materials during the design service life.
Setting up calculation procedures for the design vs. durability of reinforced concrete and prestressed concrete bridges asks for evaluating the effect of instability-generating phenomena over the performance of resisting sections.
When progressive damage of materials is not present, the evaluation of ultimate safety of a structural member is expressed by the symbolic relation:

$$S_d \leq R_d$$

where S_d is the effect of the design mechanical action and R_d is the design strength, whose invariability during service life is tacitly supposed as a rule.
When damage of materials is present (corrosion,fire, abrasion,..) the term R_d is modified and subsequent reduction changes the state of safety, in the same manner as S_d would increase.
The quantitative evaluation of damage asks for a suitable mathematical model, able to take into account of the decrease of material performance.
When damage is due to the fatigue of reinforcement, the safety check is carried out by means of well known procedures, using SN law of material and the cumulative curve of stress - stress collective - applying a criterion of cumulative damage, generally the linear one due to Palmgren-Miner.
When steel reinforcement is subjected both to fatigue damage and to corrosion damage, corrosion's effects interact unfavorably with those due to repeated loads.
As it was experimentally shown that S-N curve in this case undergoes some changes, it is possible to evaluate the effect of corrosion on safety using modified S-N curves in the classical procedure for fatigue checking.

3. THE EFFECT OF REPEATED LOADS: THE STRESS COLLECTIVE

In order to evaluate the safety vs fatigue it is essential the knowledge of the cumulative curve of the amplitudes of the stress cycles, sometimes called more shortly "stress collective".
It can be obtained by field observation of the response of existing bridges, as in the case of the "Pecora Vecchia" bridge (on the highway Florence-Bologna near Barberino del Mugello), or it can arise from a code of practice, or it can be evaluated in an analytic way.
In the last case applied load is conveniently simulated, in its essential features, as a random process and the stress collective is evaluated by the standard analysis methods both in the time domain and in the frequency domain.

4. MATERIAL'S PERFORMANCE VS FATIGUE AND CORROSION : THE MODIFIED S-N LAW

In the case of phenomena of wet corrosion, the zone of metal's anodic dissolution is seat of "craters", which are very little discontinuities, which can lead to stress concentrations in the bars and decrease the length of fatigue life of material.
The onset of cracks is favoured by other forms of localized corrosion, and the subcritic crack propagation is facilitated by stress corrosion. When the stress corrosion is excluded, a decrease of the threshold value of the stress-intensity factor is noted (true corrosion fatigue).
As a consequence of corrosion the S-N law of materials is modified.[2]
In the field of mechanical and offshore engineering it is generally acknowledged that the corrosion leads to the progressive disappearance of the horizontal branch of S-N law and also to the variation of the slope of the oblique branch, which becomes more steep.[3]
Nevertheless as the opinions are not unanimous, it seems suitable to deepen the investigation and, in the meanwhile, to proceed cautiously.

5. THE CRITERION OF ACCUMULATION OF MECHANICAL DAMAGE

When the stress collective is not constant, the liner Palmgren-Miner criterion is usually applied to evaluate the damage due to variable amplitude stress cycles.
It is considered an acceptable compromise between the ease of use and the quality of its output informations, and it is consented by most codes [1].
In the presence of corrosion, which can lead to strong variations of materials' properties with an evolution during time different from the case of cyclic loads, the problem of modelling the interaction "corrosion-fatigue" arises. It is possible to deal with it by means of criteria of additivity or synergism.
In this paper, waiting for further studies, the Palmgren-Miner criterion is applied, meanwhile the corrosion-fatigue interaction was directly considered by means of the overall effects observed in experimental tests to get S-N law.

6. THE SAFETY ANALYSIS VS DAMAGE IN THE CASE OF CORROSION-FATIGUE

The safety analysis can be carried out working at Level 1 or at Level 2.
In the first case, Level 1, [1] there are two ways to perform the check. In the first one it is performed comparing the stress due to external loads with the allowable one for corrosion-damaged material. In the second one the cumulative damage is computed and it is compared with the allowable one, that is unity.
In the second case, Level 2, the safety index β is computed: the damage is included in the limit-state surface in the space of design variables by means of a proper S-N law.
The Level 2 is a powerful tool of investigation and is applied hereafter in the following numerical analyses.

7. NUMERICAL EXAMPLES

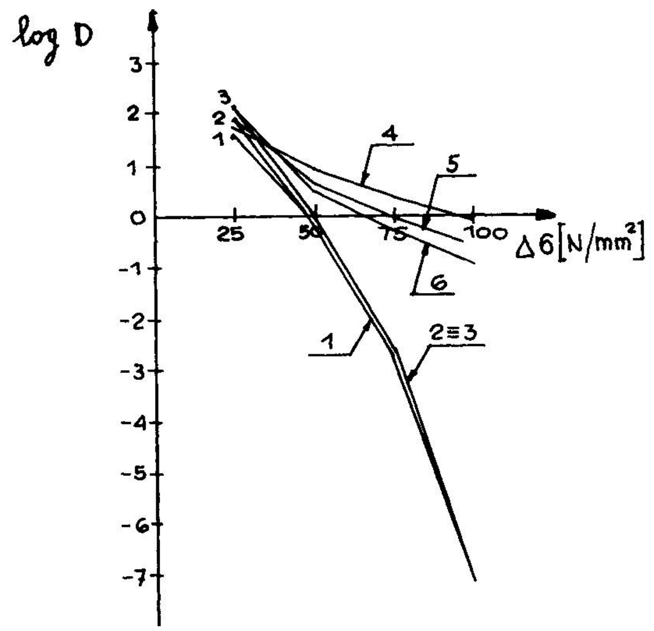
On the basis of previous concepts numerical applications were carried out.
A steel bar belonging to the deck of a prestressed concrete bridge was considered. Its behavior in the case of corrosion-fatigue was compared with the behavior in the case of conventional fatigue.
Corrosion was introduced in computations modifying the S-N law of material: first the horizontal branch was removed, then the slope of the curve was gradually increased, that corresponds to increase the exponent n in the crack growth law : $da/dN = C(\Delta K)^n$.
A parabolic stress collective (in semi-logarithmic scale) was used. This is the case of narrow-band structural response.
Its maximum level was supposed to be constant and equal to 100 N/mm^2.
The total number of stress cycles was modeled as a log-normal random variable, having mean value equal to 100×10^6 and c.o.v. equal to 0.05.
For the S-N law, a linear trend in logarithmic scale was used, fully described by:

a) ordinate at 2×10^6 cycles;
b) slope;
c) presence or absence of the horizontal branch after 2×10^6 cycles.

The ordinate at 2×10^6 cycles was considered as a log-normal random variable, having c.o.v. equal to 0.10.
Its mean value, the slope, and the presence/absence of the horizontal branch were submitted to a parametric analysis.
Four cases of the ordinate were considered (100,75,50,25 N/mm^2), three values of the exponent of the S-N law (3,4,5) and two cases of the trend of the S-N law for $N > 2 \times 10^6$ cycles (presence or absence of the horizontal branch), for a total of 4x3x2=24 cases.
For every case the deterministic damage D was first computed using the characteristic values of random variables (Fig.1), then Level 2 analysis was performed (Fig.2).

8. COMMENTS ON THE RESULTS OF NUMERICAL EXAMPLES

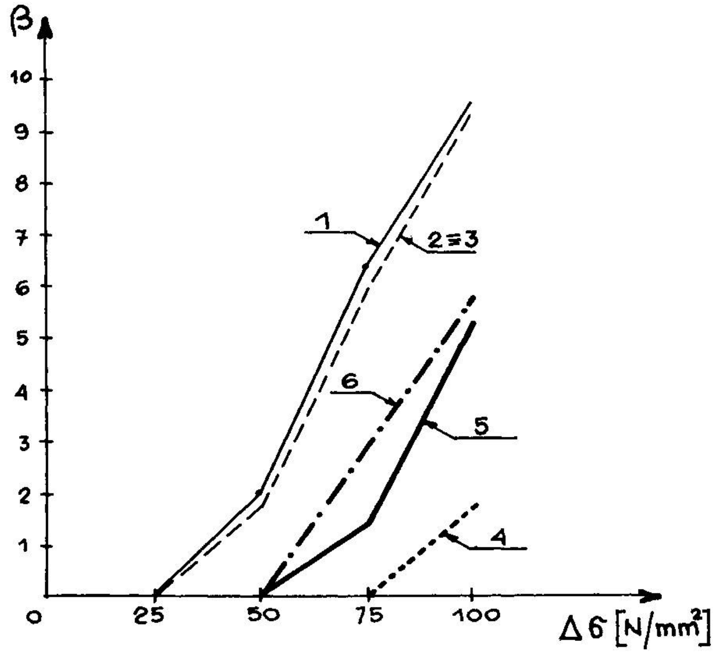
The effect of the presence of the horizontal branch in the S-N law seems to be prevailing over the variation in slope both in the



Parameters of S-N law: $NS^m=C$

Line	exponent	horiz. branch
1	3	yes
2	4	yes
3	5	yes
4	3	no
5	4	no
6	5	no

Fig.1 Deterministic damage D (Palmgren-Miner sums) for the 24 cases considered in numerical examples versus the ordinate of S-N law at $2x10^6$ cycles.



ameters of S-N law: $NS^m=C$

e exponent	horiz. branch
3	yes
4	yes
5	yes
3	no
4	no
5	no

Fig.2 Values of the safety index β for the 24 cases considered in numerical examples versus the ordinate of S-N law at $2x10^6$ cycles.

results of deterministic damage and in Level 2 computations.
In the case of S-N law with horizontal branch computed values of D and β are practically independent from the slope.
On the contrary in the case of S-N law without horizontal branch the values of are not greater than the half of the corresponding ones computed with the horizontal branch, even in the case of the most unfavorable slope.
Anyway the differences between the cases with higher exponents (4 and 5) are smaller than those between the cases with lower exponents (3 and 4).

9. CONCLUSIONS

A methodological approach for the evaluation of safety of bridges exposed to chemical-physical-mechanical damage was described.
In this frame a simplified method was presented which takes into account the corrosion even in the classical time-tension space. It is a first step toward the general approach in the hyper-space of all the design variables.
The numerical investigation performed showed a strong sensitivity of the procedure to the variations of the S-N law of reinforcing steel due to corrosion.
For this influence and on account of the limited number of reliable data on the actual behaviour of reinforcement vs. corrosion-fatigue, it is desirable to promote a systematic theoric-experimental research to get lacking data, as shown by proposed computation procedure.

AKNOWLEDGEMENTS

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Fatigue of Welded Structures at Low Temperatures

Fatigue des constructions soudées à basse température

Ermüdung geschweisster Konstruktionen bei tiefen Temperaturen

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SUMMARY

The results of integrated investigations of the low temperature (to – 70 °C) influence on the characteristics of static, dynamic and cyclic strength and crack resistance of welded joints in structural steels are presented, which are used as initial data for the design of cyclic-loaded metal structures by the existing design methods, based on deformation criteria of low-cycle fatigue as well as using the parameters and criteria of the fracture mechanics.

RÉSUMÉ

Les résultats des études globales de l'influence de basses températures (jusqu'à – 70 °C) sur les caractéristiques de la résistance statique, dynamique et cyclique, et la résistance à la fissuration des joints soudés en aciers de construction son présentés. Ces caractéristiques servent de base au dimensionnement des structures métalliques, soumises aux sollicitations cycliques, dans le cadre des méthodes actuelles, basées sur les critères de déformation, la fatigue avec un faible nombre de cycles, ainsi que les critères de la mécanique de la rupture.

ZUSAMMENFASSUNG

Es werden Ergebnisse komplexer Untersuchungen zum Einfluss tiefer Temperaturen (bis – 70 °C) auf die Kennwerte statischer, dynamischer und zyklischer Festigkeit und die Rissfestigkeit geschweisster Baustahlverbindungen dargestellt. Diese werden als Bezugswerte bei der nach üblichen Verfahren durchgeführten Berechnung zyklisch beanspruchter Metallkonstruktionen verwendet, wobei diesen Verfahren, welche Parameter und Kriterien der Bruchmechanik benutzen, die Verformungskriterien kleinzyklischer Ermüdung als Grundlage dienen.

1. ANALYSIS OF THE PROBLEM AND AIMS OF INVESTIGATION

A number of important metal structures work under low temperature conditions. Tanks and gas-holders, knock-down metal bridges and crane girders, transport galleries and overpasses, bins and silos, main pipelines, floating and stationary offshore drilling platforms are referred to such structures. These structures are subjected to both static cyclic loading (movement of a live load, periodic tank emptying and filling, product pressure variation in gas-holders and pipelines, wind pressure pulsation, wavy sea) and impulse (impact)loading.

Thus repeatedly loaded structures under severe climatic conditions are especially susceptible to brittle fracture, since for their manufacture mild and low-alloyed steels are primarily used, the critical brittleness temperature of which coincides with climatic temperatures interval. Besides, the non-isothermicity of the loading specific for these structures (sometimes with sharp temperature difference) also increases the danger of brittle fracture. It is the influence of cyclic (repeated static and impact) loading under low-temperature conditions that is often the reason of numerous failures. Under the influence of such loads cracks initiate in zones of increased stress concentration and non-uniformity of the material mechanical characteristics: either in the metal of the weld-adjacent zone of welded joints, close to weld intersection points or in butt welds with a lack of penetration or some other welding defects. At a dangerous influence of low temperatures indicates the fact that the failure intensity considerably increases (2 to 7 times) in winter periods.

Practice showed that the cracks may be formed already in the manufacturing phase or at rather early stages of operation and nevertheless the structures with such cracks may safely work. Depending on the purpose of a structure the structural limit state may be assumed as a development of an admissible crack (detected by non-destructive testing methods or specified in the manufacturing codes) or its critical length development characterized by a possibility of the structure brittle fracture (for a surface or through crack) or the structure depressurization (for a surface crack).

Thus, the limit state design of metal structures for cyclic strength should comprise two stages of structural behaviour - the stage of the fatigue crack initiation and the stage of its safe development. However, the existing codes for welded structures design for cyclic and brittle strength either don't take into consideration the effect of low (climatic) operation temperatures on the structure cyclic strength, or ignor the effect of the loading cyclicity at the evaluation of the structure load-carrying capacity by brittle fracture criteria. The effect of low temperatures is evaluated only through variations in the characteristic of fracture resistance under static loading (K_c). Besides, the difference between the static, cyclic and dynamic crack resistance of the main zones of the welded joint isn't taken into account, and only through cracks are considered, though surface and inner defects may be most probable.

On the basis of the found mechanism of welded joints in structural steels resistance to low-temperature static and low-cycle loading, initial data for cyclic loaded welded metal structures design by the criteria of low-cycle and brittle fracture were obtained.

2. EXPERIMENTAL INVESTIGATIONS RESULTS

2.1. Investigation procedure

For specimens cooling the contact method was used, that is, a refrigerant (liquid nitrogen vapour) contacted with a specimen surface. The advantage of the technique is in a free access to the zone being examined and in the possibility of installing the instruments to measure its stressed-strained state. The procedure developed allows to provide on large-scale models local cooling of their individual zones, simulating accidental leaking of a refrigerant directly on bearing structures due to damage of the structure insulating layers (liquid ammonia storage).

2.2 Physical and mechanical properties

The correct transition from measured strains to stresses is connected with the use of actual values of the materials elastic characteristics E and μ at design temperatures. The function of transverse deformation coefficient $\mu(\varepsilon_i)$ allows to take into account the peculiarities of deformation of relatively ductile steels, but being in a quasi-brittle state at decreased temperatures. The material loosening processes and, consequently, nonlinearity of the relationship between average stress σ_{cp} and average strain ε_{cp} can be described by means of the modulus of cubic strain by formula:

$$K_{\varepsilon_i} = \frac{2\sigma_i[1+\mu(\varepsilon_i)]}{9\varepsilon_i[1-\mu(\varepsilon_i)]}$$

Experimental data for $\mu^T(\varepsilon_i)$ for some structural steels (with contrast mechanical properties) show that the temperature decrease can reduce the value $\mu^T(\varepsilon_i)$ at elasto-plastic behaviour of the material (up to 3%) by 15 to 20%. Given in the Report experimentally found actual value of the static (secant) modulus of elasticity E_c^T and dynamic modulus of elasticity E_g^T showed their considerable difference. Due to this fact, at the design of metal structures when a limit state is determined according to the criterion of deformativity and at the refined design of static and repeated (low-cyclic) strength at the stage of a crack initiation and propagation (these processes take place with low deformation rates), it is suggested to use an actual value of E_c^T. The actual values of E_g^T with regard for their statistic parameters of spread are found at determination of brittle fracture properties (as well as at the design of dynamically loaded structures), characterised by a high rate at which microplastic deformations, reducing the modulus of elasticity have no time to propagate in large material volumes. In the climatic range of temperatures for specimens made of mild and low-alloy steels, subjected to a long period static and cyclic loading, it was found out that E_g^T was increased by 3% at a spread 1.4%, while E_c^T was increased by 10% and 14%, respectively (maximum values are given).

2.3 Characteristics of static and cyclic strength

The test temperature reduction doesn't alter cyclic elasto-plastic properties of structural steels and their welded joints and a static deformation is plotted at a common curve on relative coordinates ($\bar{\sigma} = \sigma/\sigma_{0,2}$ — $\bar{e} = e/e_{0,2}$) and is restricted by an initial value of plasticity ψ^T at a given temperature. Due to this fact, the influence of low temperatures at the kinetics of a stressed-strained state in the zone of structural stress concentrators is insignificant.

The climatic range of low temperatures practically doesn't reduce the low-cycle strength of structural steels and of separate zones of the welded joint at the stage up to a crack initiation, the strength of the whole welded joint at a rigid loading being limited by the metal of the adjacent to the weld zone (AWZ) or by weld metal (WM). An empirical relationship of the exponent m_e^T from ψ^T in the Coffin's equation is suggested, experimentally verified for a wide range of low temperatures

$$m_e^T = m_e^{20°C} - 0{,}047\left(\psi^{20°C} - \psi^T\right)$$

2.4 Characteristics of static, dynamic and cyclic crack resistance

The analysis of graphical relationships of the fatigue crack propagation rate (FCP) to temperature for structural steels of various strength levels and their welded joints confirms a general tendency to reduction of FCP rate with the test temperature drop, metal of AWZ and WM of low-alloyed steels having the lowest cyclic crack resistance. For some grades of low-alloyed steels a temperature range was experimentally determined (-20C for steel 20ХГСА of increased strength and -20...-40C for steel 09Г2С of medium strength), in which the rate of FCP was 1.3 and 2 times, respectively, increased. At a further temperature decreasing the FCP rate slowed down: for steel 20ХГСА by 1.3 times at -40C and 2 times at -70C, for steel 09Г2С by 2 times at -70C. It has been found out that test temperature reduction slows down the process of a surface crack growth without any influence on its shape. However, at the same time reduction of the critical defect size occurs, that might result in the structural member failure before the crack reaches a stable shape, besides the plastic zone size reduction takes place by 1.8-2.1 times for the deepest point of the surface crack and by 1.5-1.8 times - on its surface.

At temperature decrease down to -70C the values of K_c for ductile steels (low strength low-carbon steel 20К) decreased by more than 2.5 times and for less ductile steels (such as low-alloyed steel 09Г2С of a medium strength) - only by 1.9 times. Lower values of K_c were obtained for the metal of AWZ (20К) and WM (09Г2С), however, the difference between the separate zones of the welded joint is small. At -40C for high-strength steels (20ХГСА and 07Х3ГНМЮА) the value of K_{fc} for the weld metal having pronounced characteristics of a cyclic loss of strength was lower than that of the base metal by 1.7-2.6 times.

It has been established that the test temperature drop down to -70C and increasing of the loading intensity (by 10^6 times) resulted in reduction of resistance to the crack initiation in metal of all welded joint zones ($K_c^{20°C}/K_{gc}^{-70°C} \approx 5$). However, at an embrittled state of steel due to the temperature drop, the additional reduction in fracture toughness caused by high-rate loading, is only 15-40%.

3. CONCLUSIONS

The experimental results obtained are initial data for design of low-temperature cyclic durability of welded structures both in the absence of initial crack-like imperfections, and in the presence of such imperfections as lack of penetration, undercuts, non-metallic inclusions, gas cavities, etc. and structural-technological concentrators having high values of stress gradients. In the first case between cycle numbers prior to final failure N and crack initiation N_o in the absence of stress concentration there is an experimentally found Manson's relationship

$$N/N_o = 1 - 2{,}5\,N^{-1/3}$$

at $N = 10^5$ the ratio $N/N_c = 0.95$, while at $N = 10^4$ the ratio $N/N_o = 0.88$. In this case the stage of crack propagation can be considered as a negligible one. With available stress concentrators of defects these ratios undergo significant changes since the crack appears earlier, the higher is stress and concentration. Thus, the useful life of structural elements after damage detection may be 75-90% of their total durability, depending on the level and the gradient of stresses in the section considered with this contribution increasing as the stress concentration rises.

Design evaluation of the useful life of the cyclic loaded welded structures at the stage of fatigue crack growth showed that using of crack resistance parameters C, n, K_{gc} and K_{fc} with regard for their temperature dependence changed the value of durability from 30 to 50% in comparison with the design based on the constant values of these parameters.

Thus, for the purpose of fuller use of the material bearing capacity and reduction of the amount of metal per structure a possibility is presented to ensure an equal strength from the standpoint of simultaneous failure of differently stressed assemblies, connections and elements of the given structure. One of the ways of creating an equally reliable structure can be a design method based on a probabilistic approach, and allowing at the design stage to take into account real structural stress concentration and technological defects of design sections with the aim to ensure approximately equal durability by crack development criteria or its critical value with regard for temperature dependence of the design parameters.

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Durability Design for Concrete Structures

Durabilité dans la conception de structures en béton

Dauerhafter Entwurf von Betonbauten

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SUMMARY

The concept of durability design for concrete structures is proposed on the basis of comprehensive evaluation of materials, design detailings and construction works under a certain environmental condition. The new concept on durability index has been introduced. The methodology to calculate the index quantitatively has been provided in the proposed recommendation for durability design of concrete structures by the Committee on Concrete in the Japan Society of Civil Engineers.

RÉSUMÉ

La durabilité dans la conception de structures en béton armé repose sur une évaluation globale des matériaux, des détails constructifs et de l'exécution dans un environnement donné. Un indice de durabilité est introduit. La méthode de calcul de cet indice est proposée dans une recommandation de la société japonaise des ingénieurs civils.

ZUSAMMENFASSUNG

Der dauerhafte Entwurf von Stahlbetonbauten beruht auf einer umfassenden Evaluation der Materialien, der Entwurfsdetails und der Ausführungsmethoden für die gegebenen Umweltbedingungen. Es wird ein Dauerhaftigkeitsindex eingeführt. Die Methodik zur Berechnung dieses Indexes ist in einer Empfehlung des japanischen Bauingenieurvereins vorgeschlagen.

1. INTRODUCTION

Although many research works regarding to the durability of concrete have been performed in various fields, there has been very few attempts to treat them comprehensively to establish the design philosophy of durability.

It could be said impossible to realize durable concrete structures under taking consideration only into so-called design. It is not rational to provide requirements on the quality of materials and methods of construction works under no consideration of its relation to so-called design procedures.

The design philosophy should be regarded as important that a required durability in a certain environmental condition can be realized by various combinations of total construction procedures.

2. DEFINITION OF DURABILITY DESIGN

We, the Committee of JSCE on Durability Design of Concrete Structures, would like to define the durability design for concrete structures to design them comprehensively in considering the quality of materials, construction works and structural details to construct the structures in a certain environmental condition for required period without any maintenance and for a certain additional period with easy and economical maintenance.

Two indexes have been introduced to evaluate the total construction procedures comprehensively and the environmental conditions. One is "durability index" and the other is "environmental index".

The environmental index "Sp" is defined as an index calculated by the required period with maintenance-free in a certain environmental condition.

Durability index "Tp" is defined as an index calculated in the designing stage prior to actual construction works by the comprehensive evaluation on construction procedures such as quality of materials, design details and construction methods.

The performance of durability for new concrete structures could be examined by confirming that the durability index is not less than the of environmental index as shown in Eq.(1).

$$Tp \geqq Sp \qquad \text{.... Eq.(1)}$$

3. METHODOLOGY TO PROVIDE THE EQUATIONS FOR EVALUATION OF DURABILITY

If we intend to provide the durability design system quantitatively, we have to face the difficulties that there are tremendously many factors which affect the durability on concrete structures. One of the most difficult problems is how to evaluate the effect of site construction works, because it could be affected by the human behaviors.

A mong many kinds of the methodology to evaluate each factor quantitatively, we have adapted the methodology to assume and set up equations derived from the collected technical informations obtained by many research works and construction reports. Under the discussions within the committee the equations have been brushed up to practically accepted levels. There are full of engineering judgements to construct each equation which shows the Japanese research and engineering level.

Some refinements on the equations should be done in the future, however, the most importance is to provide a durability design system as soon as possible even if it has some incomplete factors.

4. ENVIRONMENTAL INDEX

The procedures for providing the environmental index are as follows.

(1) A certain value of environmental index have been set up. The index is generally assumed as 100 where we intend to realize the concrete structures in moderate environmental conditions for 50 years of maintenance free with 95% confidence.

(2) The index should be increased or decreased according to the required period of maintenance free. For example, the index is to be zero when the required period is 10 to 15 years.

(3) The index should be increased according to the particularly severe environmental conditions such as chloride content atmosphere or freezing and thawing weather conditions as shown in Table. 1.

Table. 1 Increased environmental index, ΔSp

Environmental Conditions	ΔSp
Effect of chloride contents	10～70
Effect of freezing and thawing	10～40

After all the environmental index is generally written as shown Eq.(2).

$$Sp = So + \Delta Sp \quad \text{---} \quad \text{Eq.(2)}$$

Besides the chloride contents or freezing and thawing attacking, there are some other kinds of factors which deteriorate the durability of concrete structures, such as alkali-silica reaction and fatigue by cyclic loading.

In this stage we could not take consideration into these factors because we could not yet set up appropriate ΔSp for them.

5. DURABILITY INDEX

The durability index could be determined by considering comprehensively quality of concrete materials, properties of fresh concrete and reinforcing bars and tendons, design crack width, detailings such as shape and dimensions of reinforcing bars, writing method of design drawings, concretening, reinforcing, formwork and shoring and so on.

The durability index could be computed as in Eq.(3).

$$Tp = 50 + \Sigma Tp(1.J) \quad --- \quad \text{Eq (3)}$$

$\Sigma Tp(I.J)$ are durability points, which are evaluated quantitatively considering the factors affecting the durability of concrete structures shown in Table 2.

Table. 2 Durability point, Tp (I. J)

I	J		Tp(I. J)
1		CONCRETE MATERIALS	
	1	Cement	10～ 0
	2	Water absorption of aggregates	8～-15
	3	Grading of aggregates	0～ -5
	4	Admixtures	20～-15
2		CONCRETE AND REINFORCEMENT	
	1	Workability	35～-15
	2	Strength and permiability	20～-15
	3	Unit water content	10～-25
	4	Amount of chloride contents	5～-30
	5	Quality control on the supplier's plant of concrete	10～-10
	6	Anti-corrosive reinforcing bars and tendons	modify Tp(4. 2)
3		CONSIDERATION TO CRACKS	
	1	Thermal cracking index	10～-20
	2	Flexure crack width	10～-20
4		SHAPE AND DIMENSIONS OF MEMBERS, DETAILING OF REINFORCING BARS AND TENDONS, DESIGN DRAWINGS	
	1	Shape and dimensions of members	Considered in Tp(2. 1)
	2	Concrete cover	30～-30
	3	Clear distance and layers of reinforcing bars and tendons	15～-35
	4	Additional reinforcement	10～ 0
	5	Construction joints	0～-25
	6	Design drawings	0～-30

5		CONCRETING WORKS	
	1	Experience and qualification of a chief engineer in site	20～ -5
	2	Acceptance of supplied concrete	5～ -5
	3	Transportation, placing and compaction	25～-45
	4	Surface finishing and curing	5～-30
	5	Construction of joints	modify Tp(4.5)
6		REINFORCEMENT, FORMWORKS AND SHORING	
	1	Cutting and bending of reinforcing bars	5～ 0
	2	Placing of reinforcing bars	5～-20
	3	Properties of formwork	20～-15
	4	Properties of shoring	5～ -5
7		ADDITIONAL FACTORS FOR PRESTRESSED CONCRETE	
	1	Experience and qualification of site engineers for prestressed concrete structures	0～-5
	2	Mix Properties of grout	5～ 0
	3	Properties of concrete for anchor pockets	0～-5
	4	Quality control for injection of grout	5～-5
8		PROTECTION OF CONCRETE	
	1	Protection of concrete surface	20～ 0

6. EXAMPLES OF EQUATIONS ON DURABILITY POINTS

In the proposed recommendation, computing methods for each durability point are provided as follows.

(1) Tp(2.1)=Tp(2.1.1)+Tp(2.1.2)

Workability of fresh concrete has been defined to evaluate the propertis of the flowability and segregation resistance.

The flowability is evaluated by slump value "B_{10}" and the coefficient of B_{11} which could be deternined from the easiness of pouring and filling fresh concrete everywhere in the various shaped and sized members.

Flowability : $Tp(2.1.1)=2(B_{10}-10)+B_{11}(1-B_{10}/30)$

$B_{11}=(10-8/D_{11})+(5-D_{12}^2)+D_{13}$
D_{11}:minimum lateral sige of members $\geqq$ 0.5(m)
D_{12}:maximum depth of members $\leqq$ 3.0(m)
D_{13}:coefficient regarding to the size of members.

$D_{13}=-5$: if there is a smaller section the checked one.

Segregation resistance : $Tp(2.1.2)=5-B_{12}(B_{10})^2$

B_{12}:coefficient regarding to the segregation resistance, 0.05 in general.

This value can be decreased with the use of viscons agent and is to be zero for indeal high-performance concrete which could be placed everywhere in the formwork without any consolidation processes.

(2) $Tp(2.2)=55-B_2$ where B_2(%) water cement ratio

(3) $Tp(2.3)=0.5(160-B_3)$: $B_3\leq 160$ where B_3(kg/m^3) unit water content
$1.0(160-B_3)$: $B_3>160$

(4) $Tp(2.4)=5-0.5(10B_4)^2$ where B_4(kg/m^3) amount of chlolide content

(5) $Tp(4.2)=30(\sqrt{D_2}-2)$ where D_2(cm) concrete cover

(6) $Tp(4.3)=Tp(4.3.1)+Tp(4.3.2)$
$Tp(4.3.1)=15(1-\sqrt{2D_{30}/D_{31}})$
D_{30} : Number of piled up reinforcing bars and tendons
D_{31} : Clear distance/maximum size of coarse aggregate
$Tp(4.3.2)=0.5(10-D_{32})$
D_{32} : The depth where inner rod-typed vibrators(ϕ60mm) could not inserted.

CONCLUDING REMARKS

The constitution of durability design system must be provided that the progress by the indivisual reasearch work could be easily adapted for the development of the total system. The proposed recommendation could be applied to any concrete structures with various kinds of structural design methods.
The spirit of comprehensive evaluation and the manner of exchanging on the basis of engineering judgements between materials, design details and construction works should be regarded as important for durability design of concrete structures.
By checking not only durability index but also enviromental index on many actual concrete structures, some items and equations should be refined in the future, nevertheless, we are sure this new durability design system could make new concrete structures to be more durable rationally and economically.

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Reliability and Risk Function for Deteriorated Structures

Fiabilité et risque des structures endommagées

Zuverlässigkeit und Risikofunktion geschädigter Bauteile

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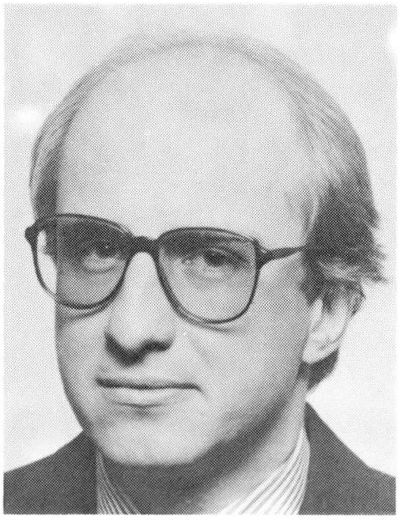

SUMMARY

Some important stochastic degradation models are reviewed and the methods for determining relevant reliability characteristics are given. The concepts for updating reliability characteristics by inspection results are reviewed. Two exemples, one for carbonation of concrete and subsequent spalling of the concrete cover due to corrosion and another for load-induced fatigue illustrate the methodology.

RÉSUMÉ

Quelques modèles d'endommagement sont rappelés et les méthodes de détermination des caractéristiques stochastiques importantes sont présentées. Les principales idées concernant l'utilisation des résultats d'inspection lors de l'analyse de fiabilité sont rappelées. Deux exemples illustrent la méthologie: l'un sur la carbonisation du béton avec prise en compte de la détérioration de la surface par corrosion; l'autre concernant la fatigue induite par des charges.

ZUSAMMENFASSUNG

Einige wichtige Schädigungsmodelle und Methoden zur Festlegung massgebender Zuverlässigkeitscharakteristiken werden dargestellt. Ein Konzept zur Berücksichtigung von Inspektionsergebnissen im Rahmen einer Zuverlässigkeitsanalyse wird erläutert. Anhand je eines Beispiels für die Karbonatisierung von Beton mit nachfolgendem Abplatzen der Betondeckung infolge Korrosion und für lastinduzierte Ermüdung wird die Methodik veranschaulicht.

1. INTRODUCTION

Explicit consideration of durability aspects of building structures is still a non–classical task of engineering, especially in a probabilistic context. Neither is the understanding of the various physical and/or chemical degradation phenomena as developed as, for example, structural mechanics nor can the classical design concepts for mechanical adverse performance states be directly applied. In fact, inspection and maintenance are integral parts of the means to achieve durability. In the following some basic terminology and notions are given first. Then a flexible model for damage accumulation and computational tools for the treatment of deteriorating components are presented with due consideration of inspection and repair. Special emphasis is given to the calculation and interpretation of the risk function.

2. BASIC TERMINOLOGY AND NOTATIONS

Let $\mathbf{X}(t)$ be the vector of uncertain quantities possibly depending on time in a deterministic or stochastic manner. Then, a limit state is defined as $g(\mathbf{x}(\tau)) = 0$ and, by convention, $g(\mathbf{x}(\tau)) \leq 0$ defines the set of failure states. Exceeding a limit state is understood as the transition of the structure into a state with a given utility loss , for example the loss associated with unserviceability or structural collapse. Hence, structural reliability is defined as:

$$R(t) = P(g(\mathbf{X}(\tau)) > 0) \text{ for all } \tau \in [0,t] \qquad (1)$$

The time dependent failure probability is $F(t) = 1 - R(t)$. If T denotes the random time to failure, an equivalent formulation is

$$R(t) = P(T > t) \qquad (2)$$

and this is the formulation most suitable for durability considerations. A structure is said to be reliable if $R(t)$ exceeds a given value $R_0(t)$. Alternatively, a limiting value can be placed on the risk or hazard function defined as:

$$\rho(\tau) = \frac{f(\tau)}{R(\tau)} \quad \text{or by} \quad R(t) = \exp[-\int_0^t \rho(\tau)\, d\tau] \qquad (3)$$

Here, $f(\tau)$ is the probability density of the time to failure. $\rho(\tau)$, if multiplied by a time interval $\Delta\tau$, obviously is the failure probability related to that time interval and to the "population" of structures still existing at time τ or the interval failure probability (failure rate) conditional on the event that the structure has survived up to τ.

3. A FLEXIBLE DAMAGE ACCUMULATION MODEL

A special though flexible type of failure model is when a "demand" process causes "capacity" reductions whose magnitude typically depends on the magnitude of the demand process. These capacity reductions accumulate. Failure occurs when the total capacity reduction exceeds some preselected value or if the demand exceeds the capacity. Abrasion of the pavement on roads due to passing vehicles or the development of cracks in vessels due to variable stresses are typical examples. The simplest but practically important formulation is [1]:

$$\frac{dZ(t)}{dt} = f(Z(t), X(t)) \qquad (4)$$

where $Z(t)$ is some damage indicator and $X(t)$ the demand process. Obviously, the damage increment per time unit is proportional to a function of the total damage at time t and the demand at that time. If, in particular,

$$\frac{dZ(t)}{dt} = g(Z(t))\, h(X(t)) \tag{5}$$

the differential equation can be separated and integrated

$$\int_{Z(t_0)}^{Z(t)} \frac{dz(\tau)}{g(z(\tau))} = \int_{t_0}^{t} h(X(\tau))d\tau \tag{6}$$

$$\Psi(Z(t)) - \Psi(Z(t_0)) = Y(t_0,t) \tag{7}$$

from which

$$Z(t) = \Psi^{-1}[Y(t_0,t) + \Psi(Z(t_0))] \tag{8}$$

Here, $Y(t_0,t)$ is a random variable obtained by integration of the random process $h(X(\tau))$. If $h(X(\tau))$ is strictly non–negative, the damage indicator is monotonically increasing. Of central importance is the additive character of the right–hand side of eq. (6) as it allows the application of the law of large numbers and even the central limit theorem under certain conditions. For example, assume that $X(\tau)$ is a stationary and ergodic process and $h(X(\tau))$ has finite variance. Then, for large t the following approximation can be found for the random variable $Y(t) = Y(t_0,t)$:

$$Y(t) \approx E[h(X(\tau))]\,(t-t_0) \tag{9}$$

In this asymptotic version the time–variation of the demand process is no more present.

There are a number of prominent applications a few of which are presented below with $y(0) = 0$. For example, let $g(Z(t)) = 1$ and $h(X(t)) = X(t)$ where $X(t)$ has mean μ and a covariance function described by the variance σ^2 and the correlation length τ_0. Then, $Z(t)$ is a Gaussian process with mean $t\,\mu$ and variance $t\,\tau_0\,\sigma^2$. It is clear that this model is suitable for the abrasion of a road pavement in time. Also, the corrosion depth of steel surfaces in splash zones can be described with this model. In both cases μ and σ may also be random functions of spatial coordinates. Next, let $g(Z(t)) = Z^m(t)$ and $h(X(t)) = X^n(t) = X^{m/2}(t)$. One finds in making use of eq. (9):

$$\ln(Z(t)) - \ln(Z(t_0)) \approx \mu_{X^n} t \quad \text{for } m = 1 \tag{10a}$$

$$\frac{1}{1-m}\left(Z^{1-m}(t) + Z^{1-m}(t_0)\right) \approx \mu_{X^n} t \quad \text{for } m = 2,3,. \tag{10b}$$

If one now interprets the function $Z(t)$ as crack length and $X(t)$ as the effective stress range we have, apart from some constants, precisely the formula for Paris–Erdogan's crack propagation law. For $m = 2$, $Z(t)$ has a lognormal distribution. Further, let $g(Z(t)) = C/Z(t)$ and $h(X(t)) = X(t)$. One determines:

$$\frac{Z^2(t)}{2C} \approx \mu t \tag{11}$$

If, on the other hand, $g(Z(t)) = (C_1/Z(t)+C_2)$ and $h(X(t)) = X(t)$, then:

$$\frac{Z(t)}{C_2} - \frac{C_1}{C_2^2}\ln(1 + C_2\, Z(t)/C_1) \approx \mu t \tag{12}$$

Inspection shows that the last two results describe the carbonation depth of concrete after continuous attack of carbon dioxide from the concrete surface according to [2] and [3] with $X(t)$ the randomly varying humidity of the outer concrete layer which changes the diffusion "constant" accordingly. Both models appear to have certain physical deficiencies but it is out of the scope of this paper to discuss those.

More general models can be generated by solving less specialized stochastic differential equations but we can not pursue this any further. Experience shows that it frequently is not the randomness of the time–variant demand process but the (time–invariant) uncertainty in the parameters in these equations, at least if t can be considered as large. Therefore, it is admissible to ignore the variability of the right–hand side of the equations in many cases.

4. FAILURE CRITERIA AND FAILURE EVENTS

The computation of R(t) under sufficient general conditions for the process **X**(t) and the shape of g(.) is by no means trivial and considerably more involved than simple time–invariant reliability problems. The same is true for the risk function. The state function most frequently is formulated in the so–called damage indicator space but it is also possible and sometimes necessary to use other formulation spaces. If damage accumulation is strictly positive and the damage indicator formulation is chosen one has to solve:

$$R(t) = P(T \leq t) = P(g^{-1}(\mathbf{X}(t);Z(t)) - t \leq 0) \tag{13}$$

Application of FORM/SORM [4] yields

$$R(t) \sim \Phi(\beta_E(t)) \tag{14}$$

where $\beta_E(t)$ is the so–called equivalent safety index defined by $\Phi(-\beta_E(t)) = P(\mathbf{X}(t) \in V)$ where V is the failure domain and Φ is the standard normal distribution function. The risk function can be determined by:

$$\rho(t) = -\frac{\varphi(\beta_E(t))}{\Phi(\beta_E(t))}\frac{\partial\beta_E(t)}{\partial(t)} \sim -\frac{\varphi(\beta(t))}{\Phi(\beta(t))}\frac{\partial\beta(t)}{\partial(t)} \tag{15}$$

The last derivative term is nothing else than the so–called parametric sensitivity factor available in most FORM/SORM computation schemes [5]. φ is the standard normal density.

The reliability calculation is much more involved if the failure criteria cannot be formulated in the damage indicator space. A typical example is failure due to instable crack propagation. Changing notations to the ones usual in this area and assuming linear–elastic fracture mechanics a crack grows "stable" as long as there is $K_{IC} > K(\tau) = C\,S(\tau)\sqrt{(\pi a(\tau))}$ with K_{IC} the fracture toughness, $S(\tau)$ the far field stress in the component and $a(\tau)$ the actual crack length which grows proportional to the effective stress ranges $\Delta S(\tau)$ raised to the power of m according to eq. (10). C and m are material constants. It is clear that failure, i.e. crack instability can also occur when $a(\tau)$ is still moderate but $S(\tau)$ is large. The difficulty lies in the fact that one is not interested that the component is in a failure state at some time but in the event when this occurs for the first time. Unfortunately, very few solutions exist for this problem and those are widely of asymptotic nature. A relatively general method is the so–called outcrossing approach for which certain regularity conditions concerning the disturbance and the damage accumulation process must be assumed. Let

$$\nu^+(\tau) = \lim_{\vartheta \to 0} 1/\vartheta\, P(\{g(X(\tau),Z(\tau),\mathbf{q}) > 0\} \cap \{g(X(\tau+\vartheta),Z(\tau+\vartheta),\mathbf{q}) \leq 0\}) \tag{16}$$

be the outcrossing rate with $g(X(\tau),Z(\tau),\mathbf{q})$ the structural state function and **q** an uncertain time–invariant parameter vector. If the disturbance process is sufficiently mixing, i.e. becomes independent for two times τ and $\tau + \vartheta$ when $\vartheta \to \infty$, the reliability function can be shown to be:

$$R(t\,|\,\mathbf{q}) \sim \exp[-\int_0^t \nu^+(\tau\,|\,\mathbf{q})\,d\tau] \tag{17}$$

For the technical details of the calculation of the outcrossing rate we must refer to the literature [6].

5. UPDATING BY INSPECTION OBSERVATIONS

The above failure models are as mentioned distinct from the failure in classical reliability as they directly adhere to the physical damage accumulation process. For the estimation of their parameters not only failure times can be used but also measurable damage indicators and the disturbance (loading) parameters as well as material parameters which frequently can be measured independent of the damage state of the component. This enables reliability updating after inspection by use of Bayes' theorem. Let t_1 be the first inspection time and denote by B the set of observations collected up to and during inspection. Then, the updated reliability is:

$$R(t|t_1, B, ...) = \frac{P(\{T > t\} \cap \{T > t_1\} \cap B)}{P(\{T > t_1\} \cap B)} \tag{18}$$

B contains events of the type $\{X(t_1) \leq \hat{x}(t_1) + \epsilon\}$ or $\{Q \leq \hat{q} + \delta\}$, where $\hat{x}$ and $\hat{q}$ are the observations and ϵ and δ the corresponding measurement errors (error vectors). Again FORM/SORM techniques facilitate numerical calculations [7].

6. EXAMPLES FOR RELIABILITY AND RISK FUNCTIONS

As a first approach the time–variant carbonation process according to eq. (12) with constants $C_1 = b_s/a$ and $C_2 = D_{A.B}\, c_o/a$ is studied where $D_{A.B}$ is the diffusion coefficient of carbon dioxide for concrete, c_o the concentration of carbon dioxide in the air, a the amount of carbon dioxide for complete carbonation and b_s a parameter which collects the retarding effects. $D_{B.A}$ and b_s are taken as uncertain with given distributions. With the exception of c_o the parameters can be related to concrete strength and the specific exposure conditions. The limit state function is formulated according to eq. (13) by assuming that regional carbonation is a necessary condition for longitudinal cracks and subsequent spalling of the concrete cover due to corrosion. Failure is assumed to occur when a certain percentage α of the reinforcement is reached by the carbonation front. In the following α is chosen to be 0.3. Furthermore, concrete cover and a model uncertainty parameter are considered as random variables [8].

Fig. 1 shows results of the reliability calculations. The risk function $\rho(t)$ is given for a concrete C15 under outdoor conditions but not subjected to rain with cover of 25 mm and 30 mm respectively . The dotted line represents the probability of failure $P_F(t)$. It is seen that up to a certain time the risk function is essentially zero. At this time the carbonation front reaches the reinforcement and failure is most probable. Beyond this time the risk function decreases reflecting the fact that the carbonation front has not reached the reinforcement before for a reduced population. Therefore inspections are most effective if they are performed just before this "discontinuity point". It follows that the planning of inspections must be affected by the characteristics of the risk function. Further on the quantification of the actual degradation state is of special importance. As visual inspections rarely are reliable sampling strategies should be developed on the basis of an optimization of the amount and the timing of inspections.

If structural components experience cumulative damage due to fatigue they have to be inspected and if necessary repaired. The risk function shows a somewhat similar behavior as shown in figure 2 which is based on Paris–Erdogan's crack propagation law and the crack instability criterion mentioned just below eq. (15). Again it is first increasing and then moderately decreasing beyond a certain point in time. It is worth noting that cost considerations specify about the same time as the optimal first inspection time (see [9]). The inspection results can be used to update the knowledge about the structural state resulting in new risk and failure probability functions (dashed lines). In the example the observed crack length was larger than estimated a priori which results in a more rapid increase of both functions. However, at this optimal inspection time the risk function and failure probability have reached rather large values (i.e. $P_F(t_1) \approx 0.3$) which may be considered as to high so that earlier inspections might be required for safety reasons. It thus is shown that both the risk and the failure probability function provide the necessary information for planning inspection times and possible maintenance actions.

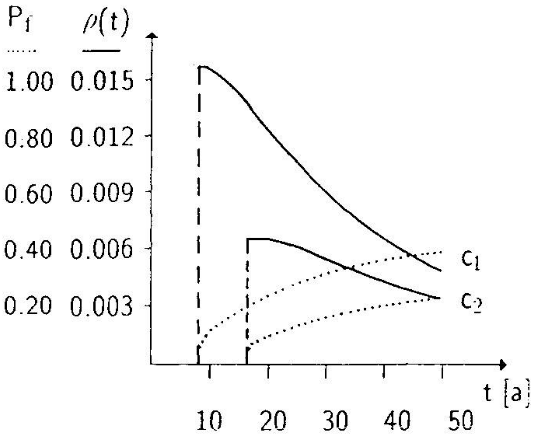

Figure 1: Hazard rate and failure probability for carbonation of concrete C15, cover c_1= 25 mm and c_2= 30 mm

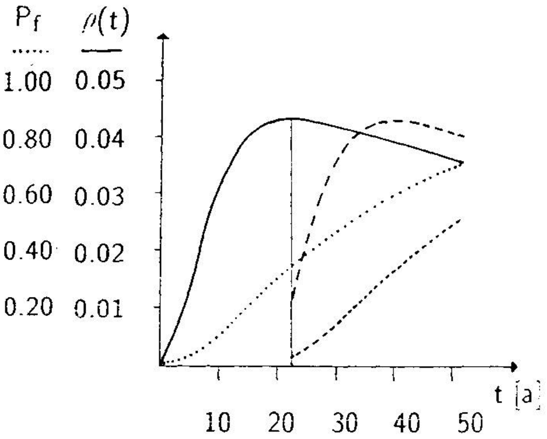

Figure 2: Hazard rate and probability of fatigue failure of a component in a steel structure, reliability updating after 22 years

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Durability Provisions for Prestressed Concrete

Réglementations pour la durabilité du béton précontraint

Dauerfestigkeitsbestimmungen für Spannbeton

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SUMMARY

The consequences of durability — related damage to structures comprising prestressed concrete are potentially greater than for those comprising reinforced concrete. This paper discusses the differences between durability behaviour of prestressed and reinforced concrete. Recommendations of codes and code provisions for durability of prestressed concrete are given. Results of a survey investigating the durability provisions of European, Australian and American concrete codes of practice are presented.

RÉSUMÉ

Les conséquences de dommages relatifs à la durabilité du béton précontraint sont potentiellement plus sérieuses que dans le cas du béton armé. Ce document examine les différences entre le comportement à long terme du béton précontraint et armé. Les recommandations et réglementations des codes sur la durabilité du béton précontraint sont présentées. Les résultats d'une enquête examinant les réglementations européennes, australiennes et américaines de durabilité sont donnés.

ZUSAMMENFASSUNG

Die Folgen von Dauerfestigkeitsschäden an Bauwerken aus Spannbeton sind unter Umständen grösser als diejenigen an Bauwerken aus Stahlbeton. Diese Studie behandelt die Unterschiede im Langzeitverhalten von Spannbeton gegenüber Stahlbeton. Empfehlungen von Normen und deren Bestimmungen für die Festigkeit von Spannbeton sind gegeben. Die Ergebnisse einer Zusammenstellung, welche die Dauerfestigkeitsbestimmungen für europäische, australische and amerikanische Betonnormen untersucht, werden dargestellt.

1. INTRODUCTION

In spite of the generally more detailed design and construction phases of prestressed compared to reinforced concrete, durability provisions for the former are often not given sufficient consideration.

Corrosion of prestressing tendons appears to be very much less common than for ordinary reinforcement and there have certainly been very few documented cases of failure due to severe durability problems in prestressed concrete components. Despite this, corrosion of prestressing cables in prestressed concrete construction presents a high risk of building failure. As a consequence of the high tensile stresses present in the small diameter prestressed wires, progressive loss of cross-sectional area due to corrosion induces rapidly increasing tensile stresses.

Despite the agreement of most researchers that the consequences of durability related damage to prestressed concrete are generally far greater than for reinforced concrete, most of the research work into durability of concrete structures has been carried out for ordinary non-prestressed reinforcement and many codes of practice throughout the world do not recognise a difference in the durability behaviour of the two types of construction.

2. MAJOR FACTORS AFFECTING DURABILITY [1],[2],[3]

2.1 Concrete Cover to Reinforcement or Tendons

Cover provides both chemical and physical protection to the steel. "Quality of concrete cover" is also essential.

2.2 Water/Cement Ratio of the Concrete Mix

Concrete permeability, and thus the rate at which carbon dioxide or aggressive agents such as chlorides can penetrate the concrete, increases as the water/cement ratio increases. Thus, concrete "quality" largely depends on the water/cement ratio.

2.3 Cement Content of the Concrete Mix

Maximising the cement content of the concrete (without causing other problems) greatly contributes to its "quality". Reducing the cement content of a mix reduces its chloride binding capacity and also its neutralising capacity against the effect of CO_2 ingress.

2.4 Characteristic Compressive Strength at 28 days (fc_{28})

It is generally agreed that durability is more dependent on the previously mentioned mix design factors and construction practice than on fc_{28} alone. This is due to the possibility of producing an adequate fc_{28} with an inadequate value of cement content or water/cement ratio as far as durability is concerned.

3. DIFFERENCES BETWEEN DURABILITY BEHAVIOUR OF PRESTRESSED & REINFORCED CONCRETE

3.1 Design

In prestressed concrete higher quality materials need to be used, with corresponding attention to quality assurance, to ensure durability.

The latest Australian Concrete Structures Code AS 3600 reflects the international trend towards "limit state" design. This provides a unified approach to the design of prestressed and reinforced concrete structures, but does not highlight any differences in durability behaviour between these two forms of construction.

Differences in durability should be emphasised at the design stage.

3.2 Materials [2]

3.2.1 Grouts and Grouting

Grouts and grouting for post-tensioned structures present a special aspect of concrete technology. Portland cement grouts have been found to be extremely efficient in preventing corrosion. This is dependent on the ducts being completely filled, since corrosion can occur in the cavities of improperly grouted ducts. Such cavities have been studied and it has been found that:

"a) Voids form more readily at higher flow velocities.
b) More voids form at high steel-to-duct area ratios.
c) Voids in the grout tend to disappear when grouting pressures are maintained constant after the grouting is completed.
d) Voids can be caused by the presence of bleed water in pockets. This bleed water is reabsorbed after the grout hardens, thus leaving a void in the structure."[2]

Non-grouted systems (popular in North America) rely on other than the passivating protection of the grout. The procedure is to grease the tendons, sheathed in a plastic duct, and seal the anchor assembly with a mortar plug.

3.2.2 Prestressing Steel

The types of corrosion that are of greatest concern are pitting, stress corrosion and hydrogen embrittlement.

Pitting is similar to the severe corrosion of reinforcement found in reinforced concrete structures. Stress corrosion results from a combination of stress and corrosion, and can lead to delayed fracture of the prestressing steel. Hydrogen embrittlement results from the embrittlement of steel by hydrogen, and can also lead to delayed fracture.

The delayed fracture mechanisms mentioned above are restricted to prestressed concrete, and cause brittle failure of the steel, often without any significant corrosion of the steel surface. Stress corrosion and hydrogen embrittlement are intrinsically more dangerous than pitting corrosion in that they may cause sudden failure, without any prior signs of distress.

4. CODE PROVISIONS AND A CODE COMPARISON

4.1 Introduction

The main aim of this paper was to carry out a study and comparision of various codes of practice for concrete structures to review how they ensure durable prestressed concrete structures and whether they recognise differences in the durability requirements for prestressed compared to reinforced concrete.

For this survey we have chosen what researchers generally believe to be the four most important factors which affect the durability of prestressed and reinforced concrete structures. These factors were introduced in Section 2 of this paper. The results of the code comparision are presented in Tables 1 and 2 for exterior (Ext.) and interior (Int.) environments. These results are summarised in the following sections.

4.2 Cover

Of the codes compared in Table 1, the only ones which recognise a difference in the minimum cover required for prestressed compared to reinforced concrete structures are: Australian Standard AS1481 (1978), ACI318M (1983), CEB-FIP MC78 (1978), FIP Recommendations (1984) and Danish Standard DS 411 (1984). The ACI and CEB-FIP Codes regard stressed tendons as "reinforcement sensitive to corrosion", this is consistent with the belief of most researchers. The ACI Code recognises the corrosion of highly stressed tendons as such a serious problem that where the extreme fibre concrete tensile stress exceeds the allowable value of $\sqrt{fc_{28}}/2$, the minimum cover for prestressed concrete members increases by 50%.

TYPICAL STRUCTURE IN METROPOLITAN SYDNEY (within 1km to 50km from coastline), fc_{28} = 32MPa													
CODES	REINFORCED CONCRETE (Reinf. bar diam 36mm) COVER (mm) i), ii)						PRESTRESSED CONCRETE (POST-TENSIONED) COVER (mm) i), ii)						COMMENTS
	BEAM		SLAB		WALL		BEAM		SLAB		WALL		
	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	
Australian AS1480(1982)	40	25	30	20	30	20							Cover to all reinforcement.
Australian AS1481(1978)							40	25	40	25	40	25	Cover to duct.
Australian AS3600(1988)	40	20	40	20	40	20							Cover to all reinforcement.
							40	20	40	20	40	20	
American	40	40	40	20	40	20							Reinf. bar diam. < 20mm) cover to all]
	50	40	50	20	50	20							Reinf. bar diam. ≥ 20mm) reinf't] iii).
ACI318M							40	40	30	20	30	20	Extreme fibre tensile stress ≤ $\sqrt{fc_{28}}/2$) cover]
(1983)							60	60	45	30	45	30	Extreme fibre tensile stress > $\sqrt{fc_{28}}/2$) to duct]
British	40	15	40	15	40	15							Cover to all reinforcement.
CP110(1980)							40	15	40	15	40	15	Cover to tendons.
British BS8110 (1985)	40	25	40	25	40	25							Cover to all reinforcement.
fc_{28} = 34MPa							40	20	40	20	40	20	Cover to duct.
European	25	15	25	15	25	15							Cover to all reinforcement.
CEB-FIP							35	25	30	25	35	25	Cover to sheath around tendon.
MC78 (1978)							Greater than b,h/2 or the values for reinforced (but ≯ 40)						b = width)) of duct h = depth)
European FIP (1984)	25	15	25	15	25	15							Cover to all reinforcement.
							35	25	35	25	35	25	≮ duct diam., ≯ 40. Cover to duct.
Danish DS411(1984)	20	10	20	10	20	10							Cover to all reinforcement.
							35	30	35	30	35	30	Cover to tendons.

Notes:
i) Unbundled reinforcement
ii) Covers generally to be not less than the reinf. bar or tendon diam. to which the cover is measured or the max. nominal aggregate size.
iii) Prestressing with unbonded tendons is common practice.

Table 1 A Code Comparison for Cover

4.3 Water/Cement Ratio

Referring to the code comparision for w/c ratio carried out in Table 2, it can be seen that none of the codes studied specify a lower w/c ratio (and hence less permeable concrete) for prestressed compared to reinforced concrete, as recommended by many researchers.

4.4 Cement Content

The code comparision for cement content carried out in Table 2 indicates that the British (CP110 and BS8110) and CEB-FIP (MC78) Codes are the most consistent with research recommendations requiring an increase in cement content for prestressed (compared to reinforced) concrete, to ensure improved durability.

4.5 Characteristic Compressive Strength at 28 Days (fc_{28})

The code comparision for fc_{28} carried out in Table 2 indicates that only the British (CP110 and BS8110) and CEB-FIP(MC78) Codes require a higher fc_{28} for prestressed compared to reinforced concrete to ensure higher quality concrete,as generally recommended for prestressed construction.

TYPICAL STRUCTURE IN METROPOLITAN SYDNEY (within 1km to 50km from coastline)																																				
	MAXIMUM WATER/CEMENT RATIO												MINIMUM CEMENT CONTENT (kg/m^3)												MINIMUM CHARACTERISTIC COMPRESSIVE STRENGTH AT 28 DAYS (fc_{28}) (MPa)											
CODES	REINFORCED						PRESTRESSED (POST-TENS.)						REINFORCED						PRESTRESSED (POST-TENS.)						REINFORCED						PRESTRESSED (POST-TENS.)					
	BEAM		SLAB		WALL		BEAM		SLAB		WALL		BEAM		SLAB		WALL		BEAM		SLAB		WALL		BEAM		SLAB		WALL		BEAM		SLAB		WALL	
	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.	Ext.	Int.
Australian AS1480(1982)	0.6	0.6	0.6	0.6	0.6	0.6							—	—	—	—	—	—							—	—	—	—	—	—						
Australian AS1481(1978)							0.6	0.6	0.6	0.6	0.6	0.6							—	—	—	—	—	—							20	20	20	20	20	20
Australian	—	—	—	—	—	—							—	—	—	—	—	—							32	20	32	20	32	20						
AS3600(1988)							—	—	—	—	—	—							—	—	—	—	—	—							32	20	32	20	32	20
American	0.5	—	0.5	—	0.5	—							—	—	—	—	—	—							25	—	25	—	25	—						
ACI318M(1983)							0.5	—	0.5	—	0.5	—							—	—	—	—	—	—							25	—	25	—	25	—
British	0.5	0.7	0.5	0.7	0.5	0.7							360	250	360	250	360	250							21	17	21	17	21	17						
CP110(1980)							0.5	0.7	0.5	0.7	0.5	0.7							360	300	360	300	360	300							25	25	25	25	25	25
British	0.55	0.65	0.55	0.65	0.55	0.65							325	275	325	275	325	275							34	25	34	25	34	25						
BS8110 (1985)							0.55	0.60	0.55	0.60	0.55	0.60							325	300	325	300	325	300							34	30	34	30	34	30
European CEB-FIP	0.5 i)	0.7	0.5 i)	0.7	0.5 i)	0.7	0.5 i)	0.7	0.5 i)	0.7	0.5 i)	0.7	240	240	240	240	240	240							16	16	16	16	16	16						
MC78 (1978)	0.6 ii)	0.7	0.6 ii)	0.7	0.6 ii)	0.7	0.6 ii)	0.7	0.6 ii)	0.7	0.6 ii)	0.7							270	270	270	270	270	270							25	25	25	25	25	25
European	—	—	—	—	—	—							—	—	—	—	—	—							—	—	—	—	—	—						
FIP (1984)							—	—	—	—	—	—							—	—	—	—	—	—							—	—	—	—	—	—
Danish	0.6	0.6	0.6	0.6	0.6	0.6							375	—	375	—	375	—	375	—	375	—	375	—	25	15	25	15	25	15						
DS411(1984)							0.6	0.6	0.6	0.6	0.6	0.6	iii)		iii)		iii)		iii)		iii)		iii)								25	15	25	25	25	15

Notes: i) Thickness of concrete: 100mm to 400mm. ii) Thickness of concrete > 400mm iii) Cement and find sand (grain size < 0.25mm).

Table 2 Code Comparisons for Water/Cement Ratio, Cement Content and Characteristic Compressive Strength at 28 days

4.6 Other Factors

Some of the codes of practice used in this comparision also limit the sulphate and chloride ion contents in the concrete mix and the allowable crack widths. Those codes which limit the latter two of these generally halve the limit for prestressed compared to reinforced concrete while those which specify a maximum sulphate content (AS3600, CP110 and BS8110) do not differentiate between the two types of construction.

5. CASE STUDIES AND SURVEYS OF DURABILITY PROBLEMS IN PRESTRESSED CONCRETE STRUCTURES IN SERVICE

5.1 U.S. Army Corps of Engineers. Durability and Behaviour of Prestressed Concrete Beams [4]

In June 1961, 20 air-entrained, post-tensioned concrete beams were placed at the Treat Island, Maine, exposure station at mean tide level and have undergone twice daily tidal inundations and an average of 129 cycles of freezing and thawing each winter. In September 1973, December 1974 and January 1983 a number of the beams were evaluated to determine the extent of corrosion that had occurred.

5.2 The Berlin Congress Hall Collapse [5]

The Berlin Congress Hall was built in 1957. On May 21, 1980, the southern overhanging portion of the roof collapsed without warning. The roof was a prestressed concrete shell structure with tendons lying within the roof membrane. The roof panels were resting on bituminous paper on top of the tensioning ring. Several tubes covering tensioning tendons were in contact with this paper. "Humidity and carbon dioxide were able to penetrate via this paper to the tensioning elements, causing severe corrosion." See Fig. 1.

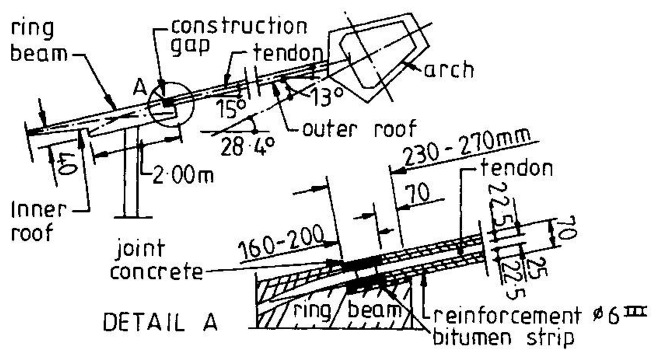

Fig.1 Detail of Arch and Ring Beam Construction

5.3 Humidification Chamber [6]

5.3.1 Introduction

The humidification chambers constructed in 1970 are used in Australia to store hardboard at a temperature of 175°C. Each chamber is 15m x 1.72m x 5.73m high. Warm air at 65 - 90°C and 92% to 95% RH is circulated through the gallery.

5.3.2 Construction

In order to facilitate speed of construction the consulting engineers devised a precast system, whereby two of the chambers were formed using precast slabs spanning between the walls of the other insitu chambers. The whole construction was held together by transverse and longitudinal prestressing tendons of 2/12.5mm dia. imparting a uniform prestess of approximately 1MPa.

Epoxy mortar was used for all horizontal joints and as a filler around duct joints. All internal concrete was protected by fibreglass.

5.3.3 Failure

During 1980 concrete began spalling off the ends of the suspended slabs separating the upper and lower chambers due to extensive corrosion of the prestressing tendons in these slabs.

Following extensive investigations the mechanism for failure was finally ascertained. At the operating temperature fibreglass is porous and allowed water, which failed to drain properly from the intermediate slab, through the epoxy mortar joints into the prestressing ducts (which had not been properly grout filled). The epoxy mortar was not resistant to the operating regime and consequently perished. The tendon ducts finally became water-logged and the tendons were consumed by a weak acid solution.

6. CONCLUSION

Durability problems in concrete structures result from inadequate detailing and specification at the design phase and/or poor work practices during the construction phase of a structure.

The corrosion of prestressing steel is fraught with more danger than that of normal non-prestressed reinforcement. The corrosion of prestressed steel proceeds at a faster rate than that of non-prestressed reinforcement under identical conditions, and presents a higher risk of building failure.

As steel is more susceptible to corrosion when stressed, prestressed concrete should require stricter durability control than reinforced concrete. Of the Codes of Practice for the Structural Use of Concrete reviewed in this paper several do not recognise a difference in the durability behaviour of the two types of construction.

Codes of Practice should reflect the recommendations of researchers by establishing stricter durability provisions for prestressed compared to reinforced concrete, particularly in the areas of cover, water/cement ratio, cement content, 28 day compressive strength, chloride ion content and sulphate ion content.

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Fair-Face Concrete Durability in Tropical Environments

Durabilité de surfaces en béton dans un environnement tropical

Dauerhaftigkeit von Betonoberflächen in tropischer Umgebung

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SUMMARY

A description of two case studies concerning the ageing of exposed fair-face reinforced concrete structures at the University of Dar es Salaam in Tanzania and at the University of Malawi at Zomba. The paper outlines the investigation of "thermal distress" as part of the mechanism of deterioration and places it in the context of other environmental factors involved, the characteristics of constituent materials and the workmanship applied to them.

RÉSUMÉ

Deux études de cas d'altération de surfaces de béton exposées aux Universités de Dar-es-Salaam (Tanzanie) et Zomba (Malawi) sont décrits. L'article présente l'analyse des sollicitations thermiques avec la prise en compte des autres facteurs extérieurs, des matériaux utilisés et de la qualité de l'exécution comme éléments du mécanisme de détérioration.

ZUSAMMENFASSUNG

Zwei Fallstudien zur Alterung exponierter Betonflächen bei den Universitäten Dar-es-Salaam (Tanzania) und Zomba (Malawi) werden erläutert. Der Beitrag beschreibt die Untersuchung der thermischen Beanspruchungszustände im Umfeld der übrigen Umwelteinflüsse, der verwendeten Baustoffe und der Ausführungsqualität als Teile des Zerfallsmechanismus.

1. Introduction

It is a commonplace assumption that concrete is a permanent material which ages slowly and requires no maintenance. It is taken for granted that concrete can survive casual disregard for good-practice guidance embodied in national codes and standards. Nothing could be further from the truth. Durable concrete is not easily produced and experience shows that, in a tropical environment, durability calls for higher standards of care and control than in temperate zones; yet it is to the latter that most established works of reference relate.

2. Factors causing deterioration

Close examination of a 30 year heritage of fair-faced reinforced concrete structures at two universities in Africa leads us to believe that "thermal distress" of exposed elements is an important and under-rated component of deterioration. Chemical contamination of constituent materials and of a completed structure is frequently discerned. The adverse effects of elevated temperatures and swift evaporation on the mixing and placing of fresh concrete are usually apparent and inadequate attention to the curing regime is always a crucial factor. These important aspects are rendered the more damaging when diurnal fluctuations in surface temperature propagate a fracture system, thereby increasing moisture penetration to liberate and transport aggressive substances, and promoting the advance of the carbonation front. Unless all reinforcement has been provided with consistently adequate concrete cover, corrosion inevitably proceeds on its destructive course.

3. Effect of surface temperature variations

Our studies indicate that surface temperatures attained under the influence of solar radiation can be extremely high on exposed concrete, particularly where conductance or re-emission of radiation are impeded by insulation. Thermal shock induced by the passage of clouds and wind or sudden rainfall swiftly extends any crack pattern initiated by imperfections in casting or curing techniques. Restraint of the member will set up stresses and deformations which, by repeated fluctuation and reversal, will induce fatigue fracture patterns and impose them upon the existing stress regime.

4. Surface temperatures at Zomba

At Zomba, in a tropical upland zone 15 degrees 23' South and 964 metres above sea level a maximum air-shade temperature of 37 degrees can be accompanied by a conservatively calculated roof surface temperature of about 84 degrees. The incident angle upon walls and other units has an ameliorating effect; northwest and northeast facing concrete louvres were found to attain 68 degrees C and 70 degrees C respectively whilst adjacent wall surfaces were 10 degrees C less. Accompanying ambient night-time temperatures were in the region of 20 degrees C, though a minimum of 7 degrees C has been registered during cooler seasons of the year.

5. Temperature propagation in concrete

Marked changes in temperature can provoke stress cracking, particularly where high temperatures are swiftly quenched. Prolonged exposure causes heating to a considerable depth. Assessment of the velocity of propagation during a 24 hour period through homogeneous concrete has been made and 35 mm per hour was taken as a reasonable average.

6. Calculation of "sol-air" temperature equivalent

For assessment purposes the heating effects of incident solar radiation and convective warm air immediately adjacent to the surface can be combined by employing the "sol-air" temperature concept. This involves the establishment of a temperature (Te) that would create the same thermal effect as the incident radiation in question, which value can then be added to the shade-air temperature (To). The composite figure (Ts) is not strictly accurate, for the radiant portion is more complex in composition than this simple relationship would suggest and a physicist would be dissatisfied with the precision of the result. Nevertheless, it is our experience that application of the solar-excess temperature by this means corresponds with on-site measurement with sufficient accuracy to be accepted as valid for the inexact science of building.

Thus Ts = To + Te
where Ts = "sol-air" temperature equivalent, in degrees C,
To = outside shade-air (dry-bulb) temperature, in degrees C,
Te = solar excess temperature,in degrees C,
= Q.a/fo
where Q = intensity of radiation heat flow, in W/sq.m
W = Watt = Joule/second
a = absorptivity of the surface; a dimensionless proportion which with reflectance r equals unity
= 0.65 for normal grey concrete (0.35 reflectance)
and fo = outside surface (film) conductance, increased by air movement across the surface,
= 12 W/sq.m degrees C for dense concrete in still air.

7. Assessment of radiation intensity

To establish the solar flux (Q) it is necessary first to obtain, for the latitude required, a solar chart with the stereographic projection of months and daylight hours on it. By superimposing upon the chart an appropriately graduated solar radiation overlay to the same azimuth scale (one each for horizontal, normal and vertical surfaces, plotted to the orientation of the building in question) it is possible to plot a sunrise to sunset maximum flux diagram for any particularly vulnerable portion of the structure. A day which corresponds with the period of maximum shade-air temperature as recorded by a local meteorological station should be selected. Drawing counterpart graphs for concurrent solar-excess and shade-air temperature throughout that day make it possible to establish a likely maximum combination.

8. Temperature gradient profiles

Simple calculator routines enable linear and non-linear temperature gradient profiles for regular intervals through the day to be constructed. Variations in shade-air temperature are slow enough for normal propagation through the structure to smooth out differentials. However, solar-excess temperature builds up rapidly on the sunlit face, propagating through the structure to emit from the shaded face by re-radiation and convection many hours later. Re-emission from the sunlit face reaches a significant level only when solar radiation wanes. The thermal "wave" causes a marked oscillation of bending and stressing, particularly if the element is firmly restrained by the structure.

9. Deformation and stresses derived

By employing well-documented standard relationships it is possible to assess likely unrestrained deformation or restrained stresses in each element. The vulnerability of exposed restrained portions of a continuous structure then becomes apparent. Tensile stresses develop which are well in excess of maxima recommended in established codes for concrete, particularly in cyclic flexure. When the cyclic nature of the diurnal thermal regime is considered certain aspects of fatigue failure come into play. One of the earliest findings of research into fatigue in concrete was that tensile cracking occurs at lower levels of cyclic or repeated load than under sustained static load. Extending consideration to the behaviour of a complex building structure becomes difficult. Observed distress in brickwork walls supporting long buildings has been explained by investigating longitudinal and transverse displacements of brickwork and concrete.

10. Recommendations for thermal durability

Our findings lead us to the following general recommendations. Movement joints designed to accommodate the extreme range of seasonal ambient shade-air temperature are sufficient to cater for the likely overall thermal response of the structure but additional attention should be given to providing more closely spaced thermal stress-relieving joints in cladding elements exposed to solar-gain. All joints should be simple and open with bearings able to accommodate countless reversals of movement. "Strong points" should be centrally located or isolated by joints. Reinforcing bars should be evenly distributed in each direction on all faces and not widely spaced. Cover should be adequate but not over-generous.

11. Other hazards determined.

Good quality, dense and durable concrete, when sensibly detailed, will withstand thermal effects without distress. However, at both universities cyclic thermal stressing can be seen to have increased deterioration originating from other causes. Zomba exemplifies an inland tropical climate of moderate altitude and humidity, notable for clear skies and drying winds. Here the main hazards are rapid water loss from the fresh mix and inadequate curing. Aggregates are derived from quarried quartz granulite and lake-shore sand. They are of reasonable quality though angular and harsh. Dar es Salaam, by contrast, lies within sight of the sea and barely 100 metres above it, both temperature and humidity are usually high. Here the main hazards are the high and irregular water demand of crushed coralline limestone aggregates and the presence of chlorides, both "historic" from within the coral and as an aerosol from the Indian Ocean monsoon winds. Sand supplies are derived from seasonal river beds, frequently silty and contaminated.

12. Hot weather concreting precautions

In both nations, as so frequently elsewhere in the tropics, many hot-weather concreting precautions are overlooked:
- testing the water supply for chloride and sulphate content,
- cooling mixing water; shading stockpiles,
- damping down shutters well in advance and erecting wind-breaks around them,
- assuming an adequate supply and consistent origin for cement and sand,
- incorporating a water-reducing additive to restrain the water-cement ratio and enhance workability, painting mixers, bins and barrows a light colour,

- mixing, transporting, placing and compacting the concrete in one swift operation during early morning or evening hours,
- covering the concrete immediately it is finished,
- curing it for a sufficient period with more than a single dribbling hose during working hours only.

Getting these right is the exception rather than the rule, yet it is these precautions that produce durable concrete with an inherent resistance to the attrition of tropical conditions.

13. Carbonation and sulphation

Research during recent years has established that a world-wide increase in atmospheric carbon dioxide above its normal 0.03 percent is taking place. It is postulated that deforestation and desertification bear as much responsibility as industrialisation. If that postulation is true, tropical Africa is affected equally with other parts of the globe. The university buildings of the Malawi and Tanzania testify to the action of this natural "pollutant" causing carbonation of concrete and the neutralization of its high alkaline passivation of reinforcement to a degree similar to that of structures in the United Kingdom. Natural carbonization of rainwater leads to "acid rain" and natural biological and bacterial action in surface moulds promote the production of sulphates and catalyse carbon, further neutralizing the alkalinity of concrete and leading to surface cracking. The higher ambient temperatures of the tropics tend to promote these chemical reactions.

14. The visual ageing of concrete

The main symptoms of ageing are colour change (including dirt collection), organic growths (algae, lichens), inorganic growth (efflorescence, lime deposits), stains from interactions with other materials, crazing, spalling and crumbling. Most of these symptoms are purely visual but the latter three can swiftly jeopardize the structural performance of the member.

15. Repair of cracking

Before embarking upon any specific treatment it is important to be aware of why the cracks have occurred, otherwise an inappropriate and ineffective, even damaging, repair method might be selected. Remedial work is normally undertaken only when one or more of the following points adversely affect the performance of the structure;
- the structural safety, loadbearing capacity or stability,
- the durability; wide cracks allow access of air and moisture to the reinforcement,
- the appearance; of importance on aesthetic grounds only.

16. Types of crack

Cracks can be divided into three categories and it is important to be sure which category applies before repairs are undertaken;
- dead cracks, caused by a past event not expected to recur. These can be filled with a rigid repair material,
- live cracks, which do not remain constant in width but open and close as the structure sustains load or temperature change. These should only be filled with a flexible repair unless the movement can be eliminated or accommodated elsewhere.
- growing crack, which increase in width because of a continuing defect, for instance, corrosion of reinforcement. These should only be repaired when the cause has been eliminated.

17. Choices of crack repair method

The location and environment of the cracks affect the choice of repair material and method. Techniques which rely upon gravity to introduce material into the crack may be used successfully on horizontal surfaces but are rarely effective elsewhere. The presence of moisture, liquid water or contaminants must be taken into consideration. In our experience, the skill of available operatives is an important consideration; elegant "hi-tech" systems call for specialist expertise.

18. Normal range of choice of repair material

Repair methods available can be summarised as follows;
- resin injection, epoxy and polyester resins are commonly used for crack injection, they exhibit high strength (usually excessive) and can be formulated to resist all commonly encountered conditions except very high temperatures. They are relatively rigid (particularly so the epoxies) and thus not suitable for live cracks. Resin injection calls for the presence of a specialist contractor.
- vacuum impregnation, involves encapsulating the structure, creating a vacuum within it and employing natural air pressure to force resin into the cracks. Though effective the method is difficult to apply and, as before, calls for the presence of a specialist contractor.
- polymer emulsions, applied as suspensions of polymer in water and appropriate to gravity applications only. Less robust than epoxies and polyesters but with more strain tolerance. Some are susceptible to damp conditions. They are more appropriate for application by unskilled labour and can penetrate fractures as fine as 0.1 mm.
- cement based materials, are suitable for the wider "dead" cracks, and when an appropriate polymer bonding agent is incorporated they can be applied by semi-skilled labour to a large proportion of repairs by trowel. By adjusting the formulation it is possible to apply them by gravity or injection.

19. Surface coatings

A wide range of surface coatings can be applied to concrete, from thin purely cosmetic emulsions to thick membranes. If cracking has become stable then a coating can usually be applied but our experiences have not been entirely satisfactory. Vapour permissivity is an important characteristic to avoid detachment yet, when it is achieved, liquid water, oxygen and carbon dioxide seem ready to pass through also. Recent advances in molecular organic chemistry have resulted in a wide range of reactive siliconates which utilize residual alkaline components of the concrete to form "permanent" bonds. They do not line the pores but become unified with the pore structure. Unfortunately they all employ volatile alcohol bases and are therefore sensitive to high application temperatures; a serious impediment to the reaction process and almost unavoidable in the tropics. New siloxanes appear to be solving the problems we have experienced and may offer a practical treatment for the future.

Remedy to Loss of Workability in Hot-Weather Concreting

Remède à la perte de la maniabilité lors du bétonnage par temps chauds

Gegenmittel zur Minderung der Verarbeitbarkeit von Betonieren bei heisser Witterung

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SUMMARY

This paper presents a study of the change in the degree of workability as influenced by variation of the initial temperature and water content of concrete mixes. Emphasis is placed on how to offset the loss of workability of a concrete mix cast in extremely hot weather. Apart from the limitation imposed by the material conditions relating to this work, the generality of certain findings is worth noticing.

RÉSUMÉ

Cet article traite de la mise en oeuvre influencée par la variation de la température initiale et de la teneur en eau des mélanges de béton. L'attention est portée sur les moyens de compenser la perte de maniabilité d'un mélange de béton coulé par temps extrêmement chauds. A part la limitation imposée par les conditions des matériaux en relation avec ce travail, le caractère général de certains résultats obtenus mérite d'être mentionné.

ZUSAMMENFASSUNG

Dieser Beitrag behandelt die Abhängigkeit des Verarbeitbarkeitsgrades von Betonmischungen von Anfangstemperatur und Wassergehalt. Es wird dargelegt wie die Verminderung der Verarbeitbarkeit infolge heisser Witterung vermieden werden kann. Die Allgemeingültigkeit der Zusammenhänge ist bemerkenswert.

1. INTRODUCTION

Freshly mixed concrete must be kept workable during the entire placing period to permit satisfactory compaction and finishing. Also, it must be kept plastic for a sufficient period so that succeeding lifts can be placed without development of cold joints.

Hot-weather concreting could result in loss of workability or increased water demand, premature setting, formation of plastic shrinkage cracks and loss of strength [2,5,7]. The increased water demand and placing problems can lead to highly permeable concrete which is undurable. To counteract the loss of workability and rapid setting encountered in hot weather conditions, it is imperative to use a suitable set-retarding and water-reducing admixture in the concrete mixture [1,6,8].

Based on wide experimental data, this paper investigates the way by which workability is affected and could be treated at high environmental temperatures. The British compacting factor test was chosen in the present work to measure the degree of workability. The test bears close relation to the definition of workability; it is more sensitive than the slump test for stiff concrete mixes, and is more suitable for field use than the remoulding and Vebe tests.

2. TESTING PROGRAMME, MATERIALS AND PROCEDURES

Concrete mixes were generally designated either with admixture or without admixture. Mixes containing the admixture were classified according to the dosage level into three classes: normal, above normal and high. The normal dosage amounts for 0.2 ℓ per 50 kg of cement; the other two levels are 0.25 and 0.3 ℓ, respectively. For simplicity, these dosages will be denoted as 0.4%, 0.5% and 0.6%, respectively.

Mixes being so classified were prepared and tested at three different ranges of ambient temperatures, viz: from 22 to 24°C (at laboratory conditions), from 30 to 33°C, and from 40 to 44°C (outdoors). The corresponding initial temperature of the fresh concrete itself ranged from 24 to 25°C, 28 to 29°C, and from 33 to 34.5°C, respectively.

As regards the water content, the concrete mixes fell into three categories: 190, 300 and 210 kg per m^3 of concrete. The water-cement ratios ranged from 0.4 to 0.65, and were varied in 0.05 increments. Well-proportioned crushed coarse ggregate with a maximkum size of 19 mm, and ordinary Portland cement were used in all the mixes. The fine aggregate had a fineness modulus of 2.4.

The admixture used was the FEBFLOW Retarding Concrete Plasticiser, manufactured by FEB (Great-Britain) Ltd [4]. The admixture, a non air-entraining, water-reducing, set-retarding admixture, is a concentrated aqueous solution of lignosulphonic base, and free from added chlorides and nitrates. It complies with Type D of ASTM C 494 [3].

Non-laboratory concrete mixes were mixed and tested under an open-air shelter. No special precaution was taken to control the evaporation of the mix water during testing. The intention was to have conditions similar to those prevailing at the job sites.

3. PRESENTATION AND INTERPRETATION OF TEST RESULTS

The results of the compacting factor test for the concrete mixes (with and without the retarding/reducing admixture) having different water contents, different water-cement ratios and different initial temperatures are given in Table 1.

Water content, kg/m³ of concrete	Water cement ratio	Without admixture						With a set-retarding, water-reducing admixture added in (by wt. of cement)																	
								0·4%		0·5%		0·6%		0·4%		0·5%		0·6%		0·4%		0·5%		0·6%	
		C.F.	I.T. °C	C.F.	I.T. °C	C.F.	I.T. °C	C.F.	I.T. °C	C.F.	I.T. °C	C.F.	I.T. °C	C.F.	I.T. °C	C.F.	I.T. °C	C.F.	I.T. °C	C.F.	I.T. °C	C.F.	I.T. °C	C.F.	I.T. °C
190	0.40	0·892	24·5	0·875	29	0·867	33·5	0·924	24·5	0·941	24	0·945	24	0·914	28	0·924	28	0·925	28·5	0·898	34	0·915	34	0·920	33
	0.50	0·895	24·5	0·880	28	0·859	34	0·925	24·5	0·938	24	0·938	24·5	0·909	28	0·929	28	0·931	29	0·905	34	0·912	34	0·922	33
	0.60	0·887	25	0·883	29	0·865	34	0·929	24	0·932	24	0·940	25	0·926	28·5	0·919	28	0·928	29	0·895	33·5	0·920	34	0·914	34
	0.65	0·893	24·5																						
200	0.40	0·920	24	0·910	28·5	0·900	33	0·940	25	—	–	0·951	24							0·923	34			0·932	34
	0.50	0·924	24	0·916	28·5	0·895	33	0·941	25	0·953	24	0·958	24	0·934	28·5	0·941	28·5	0·943	29	0·925	34	0·927	34	0·936	33·5
	0.60	0·918	25	0·908	29	0·891	33·5	0·946	24·5	—	–	0·947	24·5		–					0·920	34	—	–	0·930	34
	0.65	0·926	24																						
210	0.40	0·941	24	0·926	28·5	0·892	33·5	0·953	25	0·962	24	0·97	24·5	0·944	28	0·953	28	0·957	28.5	0·912	33	0·937	33·5	0·955	34
	0.45	0·950	24	0·929	28·5	0·910	33	0·952	25	0·967	24	0·968	25	0·950	28	0·948	28	0·962	28·5	0·917	33	0·933	33	0·948	34·5
	0.50	0·943	24	0·931	28	0·893	34	0·948	25	0·964	24	0·963	25	0·945	28	0·950	29	0·955	28·5	0·914	33	0·930	33	0·951	34·5
	0.55	0·938	24·5	0·925	28	0·913	33	0·957	25	0·958	24	0·975	24·5	0·940	29	0·954	28·5	0·960	28	0·923	34	0·932	33	0·952	34
	0.60	0·942	24	0·932	28	0·900	33·5	0·960	24·5	0·970	24	0·971	25	0·937	28·5	0·952	29	0·961	28	0·928	34	0·930	33·5	0·946	34·5
	0.65	0·945	24	0·925	28·5	0·910	33	0·952	24·5	0·955	25	0·967	24	0·942	28·5	0·945	28.5	0·952	28	0·925	33·5	0·927	33·5	0·945	34·5

Table 1 Compacting factor (C.F.) of concrete, with and without set-retarding, water-reducing admixture, at different initial concrete temperatures (I.T.) and for various water contents and water-cement ratios

On reviewing these results, the following observations can be made:

- The effects of the initial concrete temperature and the water content are particularly significant. The compactin factor decreases considerably as the initial concrete temperature increases; the same tendency is detected when the water content is decreased.
- For a given water content and at a specific initial temperature the compacting factor increases with the addition of the retarding/reducing admixture. This increase is greater at higher admixture dosages.
- The effect of the water-cement ratio appears to be insignificant when the values of the water content, the initial temperature and the admixture dosage are kept unchanged.

Based on the last observation, the average value for the compacting factor was considered the representative value at a certain initial temperature, water conent and admixture dosage. Making use of these results, the relationship between the compacting factor and initial concrete temperature is shown in Fig. 1 for different values of water content and dosage level. The figure explicitly shows that the respective relationship is linear, and that the straight lines are almost parallel to each other. It is also apparent that the said linearity is maintained fairly well for a range of initial temperatures extending between 20°C and 37.5°C. This range, fortunately, covers in practice the overwhelming majority of concrete temperatures encountered in the tropical and subtropical countries throughout the whole year. Referring to Fig. 1, the linear relationship may be mathematically expressed as:

$$CF = -mT + k \tag{1}$$

where CF is the compacting factor; T is the initial concrete temperature in degrees Celsius; m is the slope of the line, and k is the intercept.

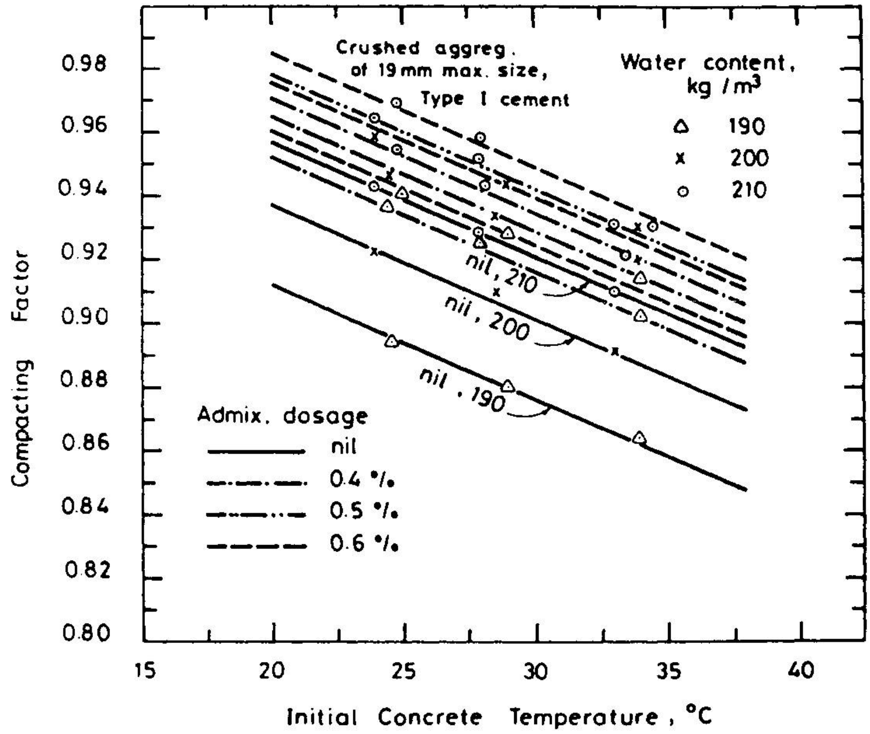

Fig. 1 Compacting factor as influenced by initial concrete temperature, water content and dosage of retarding/reducing admixture

The coefficient k is dependant on the water content and dosage level. Since all the lines are parallel, the slope m is independent of the values of water content and admixture dosage. The value of m represents, in fact, the value of the rate of drop in the compacting factor (or loss of workability) due to a unit rise in the initial concrete temperature. The latter tends to be higher in hot weather.

The value of m has been found equal to 0.0037, and hence Eq. 1 becomes:

$$CF = k - 0.0037T \tag{2}$$

Making use of Fig. 1 and/or Table 1, the values of the coefficient k were calculated and are listed in Table 2.

Water content, kg/m^3	Coefficient k			
	Zero dosage	0.4% dosage	0.5% dosage	0.6% dosage
190	0.985	1.026	1.03	1.034
200	1.01	1.038	1.045	1.048
210	1.03	1.044	1.051	1.058

Table 2 Values of coefficient k at different water contents and dosage levels

By virtue of the data of Table 2, Fig. 2 was plotted. The figure illustrates the relation between the coefficient k and the water content at dosage levels 0, 0.4% and 0.6%; the relationship tends to be linear for all the dosage levels. However, not all the family of lines are parallel. Evidently, the figure permits a reliable linear extrapolation for water contents down to 180 kg/m^3 and up to a 220 kg/m^3. Between the above two limits every value of the water content used in practice lies.

Figure 2 reveals distinctly that the lower the water content in a concrete mix, the more influential the role of the admixture dosage in improving the

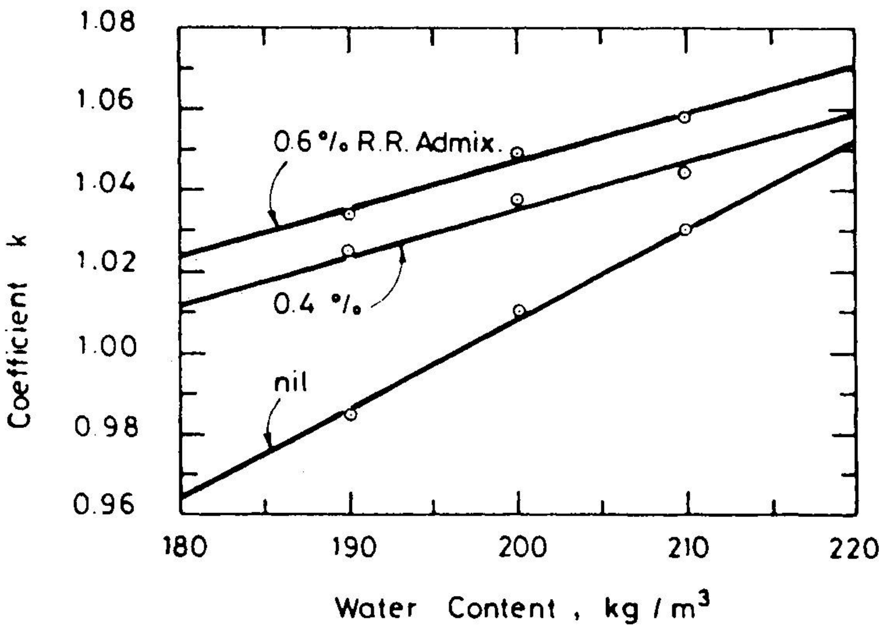

Fig. 2 Coefficient k versus water content at various dosage levels

workability of the mix. Further examination of Fig. 2 indicates that for a specific admixture dosage, the rate of increase in the value of the coefficient k is almost constant over the whole range of practical water-contents.

These observations suggest that when the water content is low, the optimal dosage of admixture intended to offset the loss of workability at hot-weather temperatures could be taken about 0.4%, whereas a dosage of 0.6% would be optimum at high levels of water content. Intermediate dosages would best suit moderate water contents. Use of admixture dosages greater than 0.6% is not a recommended practice, because the likely relative gain in the compacting factor value is very limited and accordingly is not justified by the increase involved in cost. To this end, the importance of Eq. 2 and Fig. 2 becomes evident. Their utilization is advantageous in the following cases:

- Prediction of the compacting factor, if the values of the initial concrete temperature, the water content and the dosage level are known.
- Determination of the water content, when the initial temperature is known and the compacting factor and dosage level are decided upon.
- Selection of the dosage level in case the values of initial temperature, compacting factor and water content are given.

The latter case is of great help when designing concrete mixes in hot weather while applying standard methods of mix design. It should be remembered, however, that utilization of Eq. 2 and Fig. 2 remains within the limits of the materials used.

4. CONCLUSIONS

Based on this study and within the limits of the materials used and the conditions covered, the following conclusions can be drawn:

- The initial concrete temperature and not the ambient temperature has a pronounced effect on the workability of concrete mixes; the workability degree significantly falls with the increase of this temperature. Nevertheless, the initial concrete temperature rises in hot weather.
- A loss in the compacting factor of about 0.0037 per 1°C rise of initial temperature was found.
- In Kuwait and similar hot regions, concrete cast in summer can be expected to have a lower compacting factor of about 0.05 than a similar mix cast in winter.
- Addition of water-reducing and retarding admixture can favourably offset the loss in workability of concrete cast at high temperatures. The lower the mixing water content, the more effective the role of an admixture dosage in improving the workability.

  Dosage of FEBFLOW Retarding Concrete Plasticiser would be optimally recommended at about:

  0.4% for mixes of originally low workability,
  0.5% for mixes of originally medium workabilty,
  and 0.6% for mixes of originally high workability.
- For a given water content, the water-cement ratio seems to have no significant effect on the compacting factor of well-designed concretes with or without water-reducing and retarding admixtures and irrespective of the concrete temperature.

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Hygrothermic Behaviour and Durability of Vertical External Walls

Comportement hygrothermique et durabilité des parois des bâtiments

Hygrothermisches Verhalten und Dauerhaftigkeit vertikaler Aussenmauern

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SUMMARY

The reliability requirement and the technological requirement of hygrothermic behaviour will fully answer the demands of durability of vertical external walls, if the intervention is carried out only during use but also during the design program, by means of a data input which approaches as nearly as possible the conditions of use which shall actually take place. This study aims at determining, on the basis of several hypothesized models of users behaviour, the maximum quantity of vapour produced one hour in the case of families with the same number of components, but with different characteristics and in the case of families with a variable number of components. The research results are shown in a diagram providing simple use for designers.

RÉSUMÉ

Pour que la fiabilité et la qualité technologique du comportement hygrothermique répondent parfaitement à l'exigence de durabilité des parois, il faut intervenir dans la phase de projet avec des données aussi proches que possible des conditions d'utilisation. Cette contribution essaye de déterminer, sur la base de différentes hypothèses des modèles de comportement des usagers, la quantité maximale de vapeur produite durant une heure dans des familles avec un même nombre de personnes, mais de caractéristiques différentes ainsi que dans des familles avec un nombre variable de personnes. Les résultats de la recherche sont représentés dans un diagramme aisé à consulter par l'ingénieur.

ZUSAMMENFASSUNG

Solange die Vertrauenswürdigkeit und die Technologie des hygrothermischen Verhaltens für die Dauerhaftigkeit massgebend sind, müssen diese nicht nur beim Gebrauch, sondern auch bei der Planung möglichst realistisch berücksichtigt werden. Hier hat man, aufgrund verschiedener Verhaltensmodelle der hypothesierten Bewohner, die maximale Menge des in einer Stunde erzeugten Dampfes, für Familien derselben Grösse, jedoch mit verschiedenen Charakteristiken und für Familien verschiedener Grösse, festzulegen versucht. Die Ergebnisse sind in einer Tabelle und einem Diagram enthalten, welche beim Projektieren verwendet werden können.

1. PRELIMINARY REMARKS

The reliability requirement (see UNI specification 7959) intended as the ability or attitude of external vertical walls, their single parts and their components, to keep quality constant in time, according to clearly defined conditions of use and by control , prevention and maintenance operations, seem to answer fully to the requirement of durability; but with a view to satisfying the durability requirement the intervention is not requested only during the use but also during the design program with a data input approaching as nearly as possible the conditions of use, that shall take place.

2. INTRODUCTION

It is well-known that the production and migration of water vapour is determinant in the shaping of the behaviour model of external walls. In the study of the technological requirements of the hygrothermic behaviour, in order to avoid the interstitial and/ or superficial condensing, it was found necessary to introduce into the construction design program, among the other measures to be taken, the study of the production of water vapour in such buildings as: halls (theatres, cinemas, etc.), hospitals, industrial premises, civil buildings, etc., in relation to the great number of people and to the particular activities that are carried out in them and wherever a severe check of hygrometric conditions could be required.
With a view to examplifying the proposed method, a study has been introduced, which regards the production of water vapour in flats; this method analyzes the causes determining such a production, the behaviour of the users, in relation to their activities carried out in flats and to any other parameter useful for the calculation of the water vapour quantity that is produced in a fixed time's unit.
Therefore the research has set up a methodology of study and investigation useful for the definition of behaviour models of users with a number of hypotheses on the activities of users, on their presence even in relation to their external activities, so that to become a useful support in design stage, supplying an analytical description, whose necessity of further investigation was not felt in the past, in order to prevent condensing that is one of the most frequent pathologies of vertical external walls.
Such a methodology of study and investigation is to become a useful element in the second stage of this research in order to determine an operational metodology intended to extrapolate notions of general use, starting from experimental measurements taken on significant representatives. The choice of exemplification fell on civil buildings because of the availability of data relative to them, in the hypothesis of the consideration that it could be always possible to extend such a methodology of investigation also to other buildings of interest for the research.

3. THE PRODUCTION OF WATER VAPOUR IN FLATS

3.1 Causes

The production of water vapour inside flats is essentially due to:
- the presence of users (breathing, sweating);
- the type of activity carried out (cooking, drying, etc.).

The production is discontinuous both for the intermittent presence of users varying the number of the components and the hours, per day, of their presence, and also for their type of activity.
Consequently it is not right to make an average of water vapour production resulting from a cycle of the occupation of rooms during the whole day because the high productions of vapour are temporary and only inside a few rooms.
It is therefore important to determine the quantity of vapour produced in a short period (an hour).
The parameters determinant of the vapour quantity depend on several factors than can be collected as follows:
- characteristics and composition of the family unit, i.e. if the components are mostly adult, elderly, or children; it is in general interesting to know the composition as to age ranges of users;
- economic and social level of the family unit;
- labour activity of the components of the unit outside and inside the flat and for how many hours;

- free time, hobbies and time devoted to sleep;
- habits at home: daily use of shower, bath, etc.;
- presence o absence of household appliances, such as dishwashers, washing-machine, drying-machine,etc..

The above listed factors that influence the vapour production in each single flat depend, as is easy to understand, on the life habits of each single component so that it appears impossible, in view of the extreme variability of the reference parametres, to determine a model of behaviour to which to refer in the design stage.

As ti is clearly impossible to theorize a sole behaviour model of reference, the research aims at the defining of a set of behaviour models from which to deduce, with simple calculation, useful indications in the design stage for an input of data closer to the actual situation.

3.1.1 Presence of users

The quantity of vapour depending on the kind of activity is expressed in table 1.

USERS	KIND OF ACTIVITY			
	Rest	Light activity	Light working	Heavy working, play and phisical training
ADULT	50 gm/h	100 gm/h	200 gm/h	400 gm/h
CHILDREN	25 gm/h	50 gm/h	100 gm/h	200 gm/h

Table 1 The quantity of vapour produced on the basis of activity (gm/h).

The values shown in the table No 1 and the following are expressed in gm/h, instead of Kg/s as required by the S.I., thus being more significant.

3.1.2 Activities

Consider that the following activities are carried out:

1) Cooking and washing, altogether:

breakfast	700	gm	from	7 to 8	a.m.;
lunch	700	gm	from	12 to 13	p.m.;
snack	350	gm	from	17 to 18	p.m.;
supper	1000	gm	from	19 to 20	p.m..

2) Clothes-washing and drying:

the washing machine but not the drying machine is there; the drying takes place (in winter time) in the toilet and are foreseen:

- No 2 weekly washings (5 kilos each);
- No 2 hand-washings (1kilo each);

time of drying: 12 hours in a room where t=20°C and H.R. = 40%.

Production of water vapour: 200 gm/h.

3) Toilet and bath:

- hot bath 400 gm/h;
- hot shower 2000 gm/h;
- other 200 gm/h.

Consider that each component has at least 2 showers and 1 bath a week, on average.

4) Other works:

- floor washing: 1500 gm in 1/2 hour 2 times a week;
- watering of plants: 400 gm 2 times a week.

Besides consider that fish basins and water vessels, etc. are absent and natural ventilation (opening of windows, draughts, etc.) is not taken into account. It was not considered the hygrothermic role played by hygroscopic materials (furniture,tiling and coating, etc.) that absorb the molecules of water vapour in case of an increase of the air H.R. (Relative Humidity) and give back to the air in the opposite case. The quantity of water vapour absorbed could even not be negligible.

3.2 Exemplification

The author wants to assess for a flat of 190 cubic mèters (70 net square meters) the vapour quantity produced by a family unit composed of four people; six different types of family units are hypothesized with the following characteristics:

Ex. No 1: father (8 working hrs. outside);
mother (8 working hrs. outside);
1 child (attending a full time school);
1 child (attending a school 5 hrs.);

Ex. No 2: father (8 working hrs. outside);
mother housewife;
1 adult son (8 working hrs. outside);
1 child (attending a school 5 hrs.);

Ex. No 3: father (8 working hrs. outside);
mother housewife;
2 children from 0- 3 years of age;

Ex. No 4: retired father;
mother housewife;
2 adult sons (each working 8 hrs. outside);

Ex. No 5: father (continous working, 6 hrs.);
mother (part-time working, 4 hrs.);
1 child (attending a school 5 hrs.);
1 child (from 0-3 years of age);

Ex. No 6: 1 elderly;
2 adults (8 working hrs.);
1 child (attending a school 5 hrs.).

For each component of the unit it is summed up in a table, on the basis of both presence and of the kind of activity carried out, the quantity of water vapour expressed in gm/h which has been produced, in the different hours of the day and in a week's time.

Table No 2 is compiled for each component of the family unit. From the summation of all the values tabulated is obtained the vapour production for each day of the week for the whole family unit.

FATHER																								
Day	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour
	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	10-11	11-12	12-13	13-14	14-15	15-16	16-17	17-18	18-19	19-20	20-21	21-22	22-23	23-24
Mo	50	50	50	50	50	50	50	100						100	50				700	100	100	100	100	50
Tu	50	50	50	50	50	50	50	100						100	50				700	100	100	100	100	50
We	50	50	50	50	50	50	50	100						100	50				700	100	100	100	100	50
Th	50	50	50	50	50	50	50	100						100	50				700	100	100	100	100	50
Fr	50	50	50	50	50	50	50	100						100	50				700	100	100	100	100	50
Sa	50	50	50	50	50	50	50	50	100	100	100		100	100	50				200	100	100			50
Su	50	50	50	50	50	50	50	50	100	100	100		100	100	50				200	100	100	100	100	50
MOTHER																								
Day	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour
	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	10-11	11-12	12-13	13-14	14-15	15-16	16-17	17-18	18-19	19-20	20-21	21-22	22-23	23-24
Mo	50	50	50	50	50	50	50	100						100	100				100	100	100	100	100	50
Tu	50	50	50	50	50	50	50	500						100	100				100	100	100	100	100	50
We	50	50	50	50	50	50	50	100						100	100				100	100	100	100	100	50
Th	50	50	50	50	50	50	50	500						100	100				100	100	100	100	100	50
Fr	50	50	50	50	50	50	50	100						100	100				100	100	100	100	100	50
Sa	50	50	50	50	50	50	50	100	600	200	200	400	100	100	100				100	100	100	100	100	50
Su	50	50	50	50	50	50	50	50	100	100	100	200	100	100	100	100	100	100	100	100	100	100	100	50
CHILD																								
Day	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour
	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	10-11	11-12	12-13	13-14	14-15	15-16	16-17	17-18	18-19	19-20	20-21	21-22	22-23	23-24
Mo	25	25	25	25	25	25	25	50										200	200	50	25	25	25	25
Tu	25	25	25	25	25	25	25	50										200	200	50	25	25	25	25
We	25	25	25	25	25	25	25	50										200	200	50	25	25	25	25
Th	25	25	25	25	25	25	25	50										200	200	50	25	25	25	25
Fr	25	25	25	25	25	25	25	50										200	200	50	25	25	25	25
Sa	25	25	25	25	25	25	25	50					50	50				200	200	50	25	25	25	25
Su	25	25	25	25	25	25	25	25	25	100	100	100	50	50	50			200	200	50	25	25	25	25
CHILD																								
Day	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour
	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	10-11	11-12	12-13	13-14	14-15	15-16	16-17	17-18	18-19	19-20	20-21	21-22	22-23	23-24
Mo	25	25	25	25	25	25	25	50						50	50	50	50	50	50	50	25	25	25	25
Tu	25	25	25	25	25	25	25	50						50	50	50	50	50	50	50	25	25	25	25
We	25	25	25	25	25	25	25	50						50		50	50	50	50	50	25	25	25	25
Th	25	25	25	25	25	25	25	50						50	50	50	50	50	50	50	25	25	25	25
Fr	25	25	25	25	25	25	25	50						50	50	50	50	50	50	50	25	25	25	25
Sa	25	25	25	25	25	25	25	50						50				200	200	50	25	25	25	25
Su	25	25	25	25	25	25	25	25	25	100	100	50	50	50	50	50	50	200	200	50	25	25	25	25
HOME ACT.																								
Day	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour
	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	10-11	11-12	12-13	13-14	14-15	15-16	16-17	17-18	18-19	19-20	20-21	21-22	22-23	23-24
Mo								700						1500				300	40	1040	40	40	40	
Tu								700						1500				300		1000				
We								700						1500				300	1900	1200	200	200	200	200
Th	200	200	200	200	200	200	200	700						1500				300		1000				
Fr								700						1500				300	40	1040	40	40	40	
Sa								700				1900	200	1900	200	200	200	500	200	1200	200	200	200	
Su									700					1700				300		1000				
Sum. Ex 1																								
Day	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour	Hour
	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	10-11	11-12	12-13	13-14	14-15	15-16	16-17	17-18	18-19	19-20	20-21	21-22	22-23	23-24
Mo	150	150	150	150	150	150	150	1000						1750	200	50	50	550	1190	1340	290	310	310	150
Tu	150	150	150	150	150	150	150	1400						1750	200	50	50	550	1150	1300	250	250	250	150
We	150	150	150	150	150	150	150	1000						1750	150	50	50	550	3050	1600	450	350	350	150
Th	350	350	350	350	350	350	350	1400						1750	200	50	50	550	1150	1300	250	250	250	150
Fr	150	150	150	150	150	150	150	1000						1750	200	50	50	550	1190	1340	290	290	290	150
Sa	150	150	150	150	150	150	150	1000	700	300	300	300	450	2100	750			900	1000	1500	450	450	450	150
Su	150	150	150	150	150	150	150	150	250	500	500	500	300	2000	250	150	150	800	800	1300	250	250	250	150

Table 2 Quantity of vapour produced by a family unit

In the histogram shown in Fig.1 are emphasized the maximum values of water production in the case of example No1.

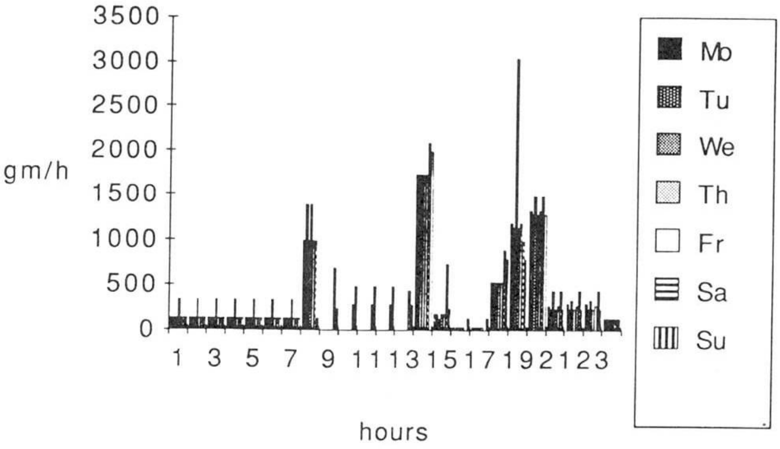

Fig. 1 Histogram of a week's days

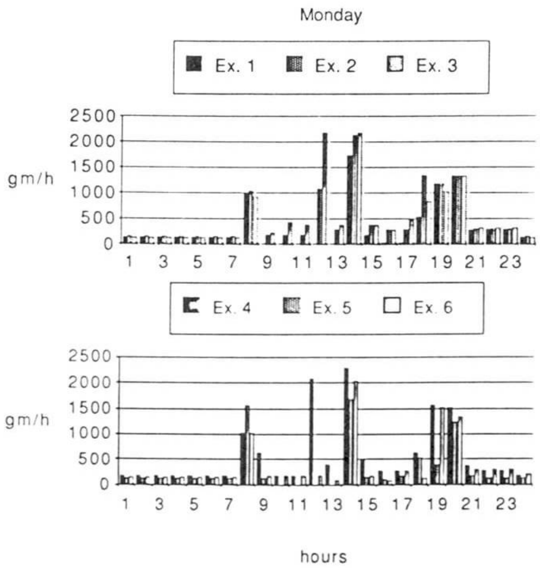

Fig.2 Histograms of three family units in a week's day

Re-processing the same calculation also for the other five hypotheses of composition of the family unit are obtained by comparison indications as to what behaviour model brings in vapour productions most concentrated.

The behaviour model that in the end is the one that gives the highest vapour production in an hour's time is No 4.

Till now it was considered the family unit as composed always by four users in the same flat of 190 cubic meters; let us now consider how vapour production varies for a number of people from 2 to 6 users in the selfsame flat before taken into consideration.

Now are examined the five different situation as follows:

Ex. A: 2 adults (1 working outside, 1 housewife);

Ex. B: 2 adults + 1 child (1 working outside, 1 housewife, 1 child from 0-3 years of age);

Ex. C: 2 adults + 2 children (see Example No 3);

Ex. D: 2 adults + 3 children (Ex. C + 1 child from 0-3 years of age);

Ex. E: 2 adults + 3 children + 1 elederly person.

From the comparison between the different situation analyzed the following diagram, shown in Fig. 2, is obtained that gives useful indications to the designer, in case of over-crowding in the considered flat, on the range of variation of the reference values for an adequate design of vapour suction devices.

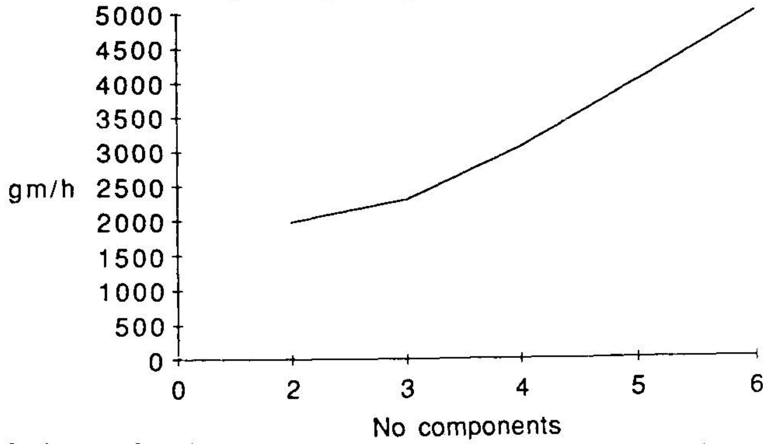

Fig.3 Diagram of maximum values assessed for family units with one to six components

4. FINAL REMARKS

Theanks to an adequate design, in the final use there shall be fewer maintenance interventions in order to ensure standard quality; or better, quality is surely maintained through a certain established period and so it is actually achieved the correspondence of the reliability requirements (ability to maintain constant in time standard quality under predetermined use conditions) and of technological requirements of hygrothermic behaviour to the exigency of durability, as for instance in the reviewed case of vertical external walls.

The presence of such a pathology as condensing causes unforseen damages and faults that require interventions of repair not easily forseen or programmed in advance as in the interventions of a real maintenance course.

To prevent pathologies means even to have global costs better defined and forseen in advance.

The global cost is the summation of the cost of settlement, building costruction, maintenance and operation for the lasting life of the buildings, and besides demolition and reuse at the end of their life (either positive or negative values).

The results of the research are also useful during a recostruction course, when for instance faulty conditions due to condensing depend directly on overcrowded flats or on the peculiar habits of users. In fact this research methodology can be used also for the acquisition of input data so that to give precise indications for the interventions that must be carried out.

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Factors Affecting Steel Corrosion in Concrete Bridge Substructures

Facteurs influant sur la corrosion de l'armature des structures porteuses de ponts en béton

Einflussfaktoren zur Bewehrungskorrosion in Brückenunterbauten aus Stahlbeton

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SUMMARY

Some 45 different areas of concrete on 15 bridge substructures were selected for detailed survey. Some were sound, others showed corroding reinforcement. Both site survey measurements and laboratory analysis of cores were made. Bridges less than 25 years old were of better concrete and had better cover than those more than 50 years old, but showed more serious reinforcement corrosion because the modern bridges were more susceptible to chloride from de-icing salt. The relationship between carbonation and concrete quality is shown by the survey.

RÉSUMÉ

Plus de 45 endroits différents de la structure porteuse de ponts en béton ont été choisis pour une inspection détaillée. Certains étaient sains, d'autres révélaient une armature corrodée. Les analyses ont été effectuées aussi bien in situ qu'en laboratoire. Les ponts de moins de 25 ans présentent un meilleur béton et un meilleur enrobage, mais une plus forte corrosion de l'armature, car ils sont plus sensibles aux chlorures des sels de déverglaçage. L'examen montre aussi la relation entre carbonation et qualité du béton.

ZUSAMMENFASSUNG

Um die 45 verschiedene Stellen von Brückenunterbauten aus Beton wurden für eine detaillierte Inspektion ausgewählt. Einige waren intakt, andere wiesen korrodierte Bewehrungen auf. Es wurden sowohl an Ort als auch im Labor Analysen vorgenommen. Brücken von weniger als 25 Jahren wiesen besseren Beton und bessere Ueberdeckungen auf, zeigten aber stärkere Bewehrungskorrosion, da sie auf Chloride von Tausalzen anfälliger sind. Die Untersuchung zeigt auch den Zusammmenhag zwischen Karbonatisierung und Betonqualität auf.

1. INTRODUCTION

1.1 Standards for the specification of concrete to make durable structures are founded on long experience. However, the evolution of modern cements and cement blends, and changes in the required performance of modern concrete to cope with changed environments and construction methods, suggest that traditional standards are not necessarily adequate for concrete for contemporary and future structures. There have already been suggestions that the changes, particularly of cement, have resulted in less durable concrete, and there is no doubt that corrosion of steel in structures only a decade or so old is all too common. To design concrete which will be adequately durable for present and future conditions it is necessary to understand the basic factors that control durability, and the work described in this paper is a contribution to that understanding.

1.2 This paper is based on a survey made for the Bridges Department of the Transport and Road Research Laboratory of Great Britain. The details of that investigation are reported in Reference [1].

1.3 For this study 15 bridges were selected from a broad survey of over 100. The bridges ranged in age from 11 to 68 years. On each bridge a number of areas of concrete, usually on the substructure and about $4m^2$, were chosen for detailed study. There were 45 areas in all. Some areas included corroding reinforcement (evidenced by spalling, cracking or rust stains) and others were apparently corrosion free. As well as visual examination of each area, the depth of cover was measured and corrosion activity assessed by half-cell survey. Core samples were taken and from these the depth of carbonation and the chloride profile in each area determined. Core samples were also used to assess the quality of the concrete (porosity, permeability and so on) and to estimate the original mix proportions.

2. RESULTS OF SURVEY

2.1 Durability and age of bridge

2.1.1 The structures surveyed tended to fall into two groups; those more than 50 years old and those between 22 and 11 years old. For a first analysis it is convenient to group the different measurements into two blocks, 'old' and the 'modern', and compare these to see what quantitative changes have happened over the more than 30 years between them.

2.1.2 The minimum depth of cover in each area tended to be less for old than modern bridges, with more than half the 'old' areas having cover of 20mm or less and less than a quarter of the 'modern' areas with this range.

2.1.3 The concrete quality was measured in a number of different ways, and these may be grouped broadly into estimates of original mix proportions, and material properties. For mix proportion the analyses give the modal values for cement content as 250 to 300 kg/m^3 for concrete from 'old' areas and 300 - 350 kg/m^3 for 'modern' ones, with distributions such that whilst over half the old areas had less than 300 kg/m^3, only one of the 18 'modern' concretes was in that range. Clearly, modern concretes generally have a higher cement content than older ones. As would be expected from this, estimates of water/cement ratios show lower values for modern concretes.

2.1.4 The material properties tend to conform to expectation from the estimated mix proportions. Capillary porosity and water absorption are less for modern concrete, and although the modal values for permeability are about the same for old and modern concretes the ranges of results are such that the very high permeabilities are associated with old, not modern material.

2.1.5 A simple comparison of the depth of carbonation of old and modern concretes shows, as expected, that the old ones generally have carbonated to greater depths. It is commonly assumed that depth of carbonation is proportional to the square root of age and this rule was used to 'normalise' the data to a standard age of 55 years. When this is done, it is found that, while 40% of the old concretes exceed 5-15mm depth, only 10% of modern concretes do. The performance of modern concrete is, then, generally better than old, and this conforms with the mix proportions and material property observations.

2.1.6 One core sample from each area was used to establish the chloride profile from the surface into the concrete. In this survey, where appreciable chloride was found, there was always a gradient from the surface inwards, indicating the chloride came from de-icing salt. The distribution of surface chloride levels for old and modern bridges are shown in Figure 1. Only 10% of modern areas showed less than 0.15% Cl^- by mass of cement compared with 60% of the old areas. On modern bridges 45% of the areas showed more than 0.5% Cl^- by mass cement compared with 15% of old ones.

2.1.7 This finding is sufficient to account for the relatively poor durability reputation of modern concrete bridges: the problem is not in the material being less durable but because the exposure conditions are more harsh for modern than for old bridges. While the reasons for this are not fully understood, two factors undoubtedly have a major influence:

a) Modern bridges tend to be on motorways which in Britain are heavily salted.
b) Modern construction tends to favour simply supported spans which require movement joints at the supports. These joints do not behave well but frequently leak, leading to salt water pouring over the substructure. The majority of cases of damage studied could be associated with such leaks. Older structures, which were generally of in-situ and more continuous forms of construction tended not to have such joints.

2.2 Depth of carbonation and concrete quality

2.2.1 Although in the U.K. the most severe durability problems result from exposure to chloride, carbonation must not be forgotten. In this survey there were a number of areas where corrosion was not the consequence of chloride penetration, and in general these were areas where the depth of carbonation had reached the reinforcement. The depths of carbonation were very variable but if they can be related simply to concrete quality some progress will have been made to understanding and control of carbonation related durability problems.

2.2.2. The complicating effect of age can be avoided for this analysis by

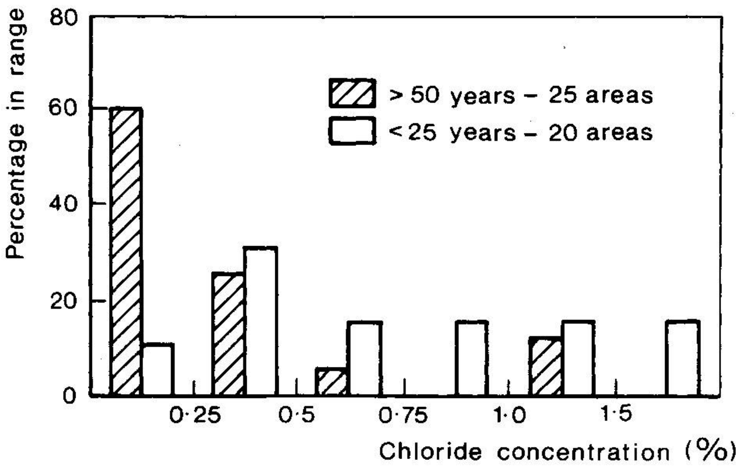

FIGURE 1

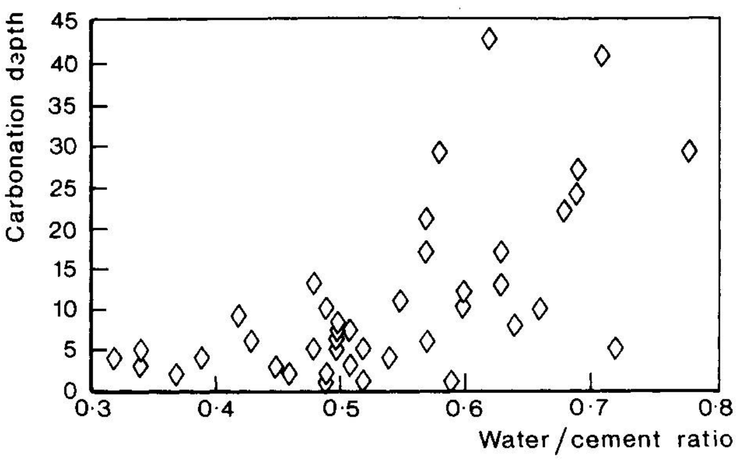

FIGURE 2

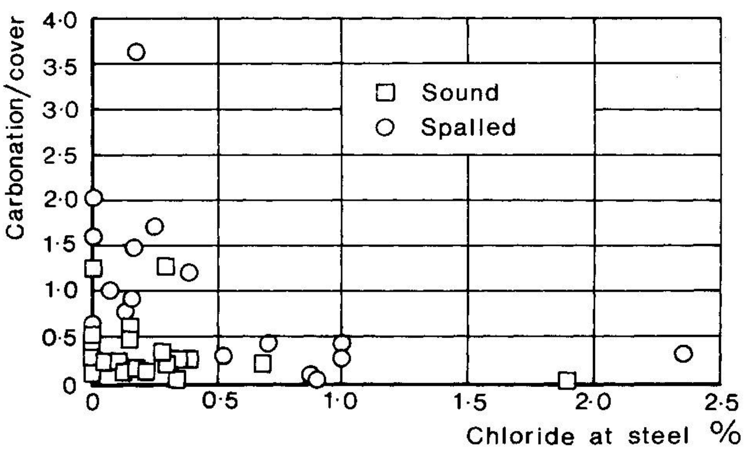

FIGURE 3

considering only the old concrete samples - their range of age is negligible compared with their starting age of 50 years. Figure 2 shows the typical depth of carbonation and the water/cement ratio for this group. There is a clear tendency for high carbonation depths to be associated with high water cement ratios and no concrete with less than 0.55 w/c showed more than 15mm carbonation. The values shown are 'typical' depths and are the mean value from all samples in the area. 'Maximum' values, the highest value in each area, were also noted and these show more variability than the typical ones but, as with the typical values, the mean depth of carbonation for areas with more than 0.6 water/cement ratio is about 10mm more than those areas below this ratio. At all ages there are a significant number of areas with effectively zero carbonation, and the wide scatter of results within the carbonated area of Figure 2 suggests that some factor additional to concrete quality is present. Since it is well recognised that carbonation rate is very moisture dependent, this factor is likely to be the micro-environment of the concrete.

2.3 Corrosion

2.3.1 Figure 3 shows that concrete is usually sound unless the carbonation depth is more than about 0.8 of the cover or the chloride level at the steel is more than 0.5% Cl^- by mass cement. In view of the uncertainty of the inferred depths on spalled concrete the carbonation/cover ratio of 0.8 is not incompatible with the expected value of 1. The critical chloride level of 0.5 compares with the maximum of 0.4% Cl^- by mass cement in British Standard 8110.

2.3.2 The effect of factors such as age or concrete quality cannot be deduced from this survey since all but two areas corroded when chloride was above the threshold. It is interesting, however, that the two areas which did not show spalling despite the high chloride level had better cover (48 and 70mm) than most of the spalled areas.

3. DISCUSSION

3.1 Carbonation induced corrosion need not be a problem with bridges. In this survey only two areas were found with depths of carbonation more than 30mm, and that after 50 years. Carbonation should not be forgotten, however. If carbonation depth increases proportionally to the square root of age then the modern concretes sampled in this report will have carbonated to a range of 5 to 40mm in the 120 year design life time of a bridge, and this is greater than many of the minimum covers found. Whilst most areas of bridges will be protected, the probability of areas of poor cover coinciding with poor concrete must increase if there is any deterioration in the general quality of the concretes specified or of the standards for achieving adequate cover. For confidence in the future there is a need for more information on the interelationship between micro-climate, bridge design and carbonation of concrete.

3.2 Chloride is a much more difficult problem. The simple conclusion from this survey is that if the chloride level is more than 0.5% Cl^- by mass cement, corrosion is very likely; and these high levels can be achieved at the depth of the reinforcement in little more than a decade. The survey does suggest, however, that since high chloride levels occurred primarily where surfaces do not drain well, and where cover was not very high, a combination of design for drainage and protection of areas susceptible to de-icing run off could increase service life significantly. A study of the interelationship between the structural details, the micro-climate, and the chloride levels and their rate of build up would be valuable both for new design and to assess maintenance needs on existing bridges.

4. CONCLUSIONS

4.1 A comparison of the performance of concretes more than 50 and concretes less than 25 years old shows that modern concretes give better protection to reinforcement than do the old ones. The corrosion which does occur on modern bridges is almost always the consequence of chlorides from de-icing salt: modern bridges seem to have a higher exposure to salt than do old ones.

4.2 Carbonation is not a problem in the short term for bridges. Concretes of reasonable quality will not carbonate more than 40mm in 120 years, and average carbonation would be less than 15mm in this time.

4.3 If chloride is present at the steel with more than 0.5% Cl^- by mass cement then corrosion is almost certainly occurring. Chloride from de-icing salt can penetrate the concrete very quickly: From this survey more than 30mm in two decades is not exceptional. The durability of bridges is likely to be improved more by changes of design to keep chloride levels low and cover high than by improvements in the quality of the concrete.

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New Building Designs Incorporating Lessons from Failures

Conception de bâtiments en tenant compte des leçons tirées de dommages

Bauwerksentwürfe unter Berücksichtigung der Lehren aus Schadenfällen

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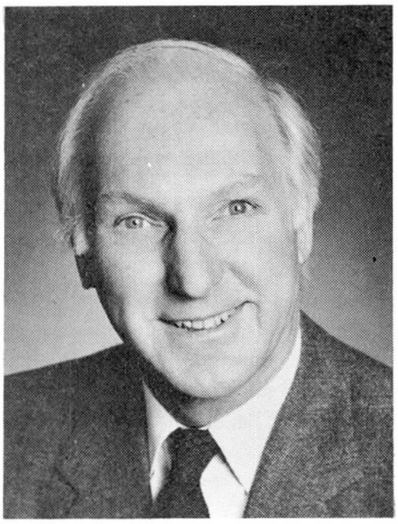

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SUMMARY

The presentation discusses some of the factors which lead to deterioration of buildings, and, in particular, parking structures. Examples are given to illustrate designs and details which have performed poorly in service, and other designs and details which perform well. Some figures are given for the apparent 'savings' made in construction costs and the real costs incurred subsequently for restoration. The general inadequacy of feedback to designers, from buildings in service, is discussed, along with some reflections on the roles of Codes and Standards.

RÉSUMÉ

Cet artcile traite de quelques facteurs contribuant à la détérioration de bâtiments, plus particulièrement de parkings. Des exemples, bons et mauvais, de conceptions et de détails constructifs sont donnés. Des comparaisons entre les économies lors de la construction et les frais de réparations sont faites. Les projeteurs sont trop peu informés du comportement en service de leurs structures. Le rôle des normes est discuté.

ZUSAMMENFASSUNG

Der Beitrag behandelt einige Einflussfaktoren auf die Schädigung von Gebäuden, speziell von Parkgaragen. Es werden Beispiele gegeben, welche ein unbefriedigendes Verhalten zeigen und andere, welche sich bewährt haben. Zahlen über die vermeintlichen Einsparungen beim Bau werden mit den Instandstellungskosten verglichen. Die mangelnde Rückkoppelung zwischen Konstruktion und Bauschäden wird diskutiert, zusammen mit einigen Gedanken zur Rolle von Normen.

1. INTRODUCTION

There has been a phenomenal boom in building construction in North America during the past 30 years. During the past 5 years there has developed a phenomenal boom in the business of repairing and restoring buildings.

Our firm now undertakes about 300 new design projects a year, and over 100 projects on the investigations and restoration of existing buildings. Some buildings suffer premature deterioration due to loading or environmental conditions that could not reasonably have been forseen at the time of design.

Most of the problems, however, could have been avoided, at very little extra cost, by better attention to design, details, specifications and construction practice.

In most cases that we have investigated, problems have resulted from a lack of judgement or care.

Many areas of Canada and the northern United States suffer extremes of climate, and significant atmospheric pollution. Salt is used extensively throughout long winters. These conditions provide very rapid tests for structural systems and materials. We hope that some of the lessons we have learned may help those practicing in other regions.

Problems occur in buildings of all categories, but parking garages, as a category of buildings, show the most widespread, conspicuous and generally costly troubles. We have designed about 400,000 square metres of parking decks, and investigated and repaired about 1,200,000 square metres of parking decks.

This paper will discuss three types of parking structures as examples. Many of the lessons from these most vulnerable structures, however, can be applied to other structures which have less severe exposure in service. The paper will deal with precast concrete, post-tensioned concrete and conventional reinforced concrete construction.

During the preparation of this paper a tragedy occurred in Vancouver, British Columbia, when the roof of a shopping centre, which was designed as a parking deck, collapsed within a few weeks of its completion. The cause of this failure has been identified by a public enquiry, as being a basic design error. The design engineer failed to consider lateral stability of the unrestrained bottom flange of a steel girder which was continuous over the supporting columns. The error was not caught by checking within the design office, by the building officials in reviewing the drawings, by the steel fabricator who produced fabricating details, nor by a second firm of consulting engineers who were called in to check the structure during construction, before the failure. The enquiry panel recommended, among other things, that structural engineers be subjected to more stringent examination before being allowed to practice. The extremely low fees negotiated for the consulting structural engineers, in this case, were criticised. It was recommended that a fee scale should be enforced with a minimum level that was sufficient to allow consultants to provide adequate time and effort to the design of building structures.

It will be interesting to see how that recommendation fares, as it is out-of-step with the present march towards deregulation. Fortunately, basic design errors which lead to tragic failure are very rare.

This paper is intended to address deficiencies which are very common.

2. THE SCOPE OF THE PARKING STRUCTURE PROBLEM IN CANADA

Various estimates have been made regarding the scope of the problem of premature deterioration of parking structures in Canada, and the approximate cost of rehabilitation and replacement.

It is believed that there are about 5,000 framed parking structures, including parking levels beneath buildings. Most of these have been constructed in the past 30 years.

As an order of magnitude indication, one study in 1987 estimated that the costs to deal with premature deterioration, as distinct from normal maintenance, may be around $3 billion in Canada alone. Even when these garages are "dealt with", they can rarely be put into a really sound condition. Some contamination remains. Although the subsequent useful life expectancy may be increased, on-going maintenance and repairs are likely to be higher than for a structure which was well built in the first place.

Table I gives data on 3 structures investigated by the author to indicate the costs involved for repair and protection on a per square metre basis.

3. ILLUSTRATIONS OF DESIGNS AND DETAILS WHICH HAVE LED TO FAILURE AND CORRESPONDING DESIGNS AND DETAILS WHICH PERFORM MORE SATISFACTORILY

Slides will be shown to illustrate each of these structures, showing details of failures or premature deterioration.

Failures illustrated include:

Corrosion of reinforcement due to:

- Inadequate concrete cover, depth and quality.
- Failure or omission of surface protection systems.
- Inadequate protection of post-tensioning tendons, unbonded, in plastic sheaths or paper wrappings.
- Inadequate protection of anchorages for post-tensioning tendons.
- Inadequate sealing systems at joints.
- Poor details leading to entrapment and concentration of contaminants.

Structural distress due to:

- Excessive deflection and displacement. e.g. creep deflection, thermal movement.
- Inadequate provision of expansion and control joints.
- Movement due to earth pressure or ice formation.
- Impact.
- Corrosion of embedded electrical conduits.

Corresponding slides will be shown to illustrate equivalent structures in which details and protection have been better engineered to provide durability in service.

4. FEEDBACK

In Canada, the construction of large numbers of parking structures began in the late 1950's, mostly for apartment buildings and office buildings. Large parking decks for shopping centres began to appear in the 1960's. Some of these were designed and built with care and consideration for exposure conditions, but many were not.

By the late 1970's serious problems were obvious in many of these structures. Effort was quickly put into investigation and rehabilitation techniques by a few firms.

In hindsight, it is both remarkable and distressing that so much new construction was completed throughout this period, and into the 1980's, without recognition of the lessons that these failures should have taught.

Developers were generally unable or unwilling to appreciate that lowest initial cost did not always mean lowest life-cycle costs. Projects were often built and sold off, so that the original developers did not have to face the subsequent repair costs.

There was fierce competition between the proponents of various systems to increase their market share by lowering initial costs. Bonded post-tensioning tendons gave way to unbonded tendons because they were $2.00/square metre less expensive. Failures in parking decks with unbonded tendons are widespread. The author is not aware of any significant failures of decks reinforced with bonded tendons.

Many precast parking structures for shopping centres were designed and built by contractors who had no experience of conventional cast-in-place structures, and were not aware of the hazards that arise in service. When leaking and corrosion did develop in precast decks, we find that owners and operations managers called in other contractors to apply sealants and to try to treat the symptoms. The original designers were rarely made aware of the service failures, and they repeated past details, or devised even less expensive ones, genuinely in ignorance of their deficiencies.

5. CODES AND STANDARDS

Until 1987, we have had very little guidance from National Codes or Standards on the design and protection of structures exposed to severe environments.

Our 1970 National Building Code stipulated that "Special attention shall be given to the spacing of expansion joints, the details of construction joints, the amount of shrinkage steel provided and the amount of protection afforded the reinforcing steel in structures in which danger of steel corrosion is increased due to the presence of salt or acid solutions or vapours."

Concrete cover requirements were stipulated for only two situations - surfaces exposed to "the weather or to be in contact with the ground", or "surfaces not exposed to the ground or weather". Parking decks, especially below-grade, were most commonly categorised as if they were not exposed to the weather - despite their severe exposure to salty water and slush brought in by vehicles.

In 1987, the Canadian Standards Association published their first Standard on Parking Structures, CAN/CSA-S413-87.

This is a landmark publication. It sets out specific recommendations for design, detailing and construction of parking garages, over and above the general requirements of the national Standards for reinforced and prestressed concrete construction. It includes particular guidance on concrete toppings, tendon protection, epoxy-coated reinforcement, protective surface membranes, construction and expansion joints, slopes and drainage. Minimum protection systems are given for light use and heavy use areas on different structural systems.

This Standard was published over ten years after serious deficiencies in general practice had become apparent. The "industry" was very slow to formalise the lessons that should have been learned from inadequate performance in practice.

This Standard has now been incorporated into the mandatory Building Code of the Province of Ontario, and all new parking structures are required to comply with its requirements.

To meet these requirements, a parking deck probably costs about $30.00/square metre more than the cost in today's dollars of the poorest practices which were commonly followed by developers two or three years ago. There have been strong complaints from suppliers of some systems which were not incorporated into the Standard. We feel there is a justifiable fear that an exclusive Standard may inhibit the introduction of new and possibly improved materials and methods. But better mouse-traps do eventually force their way onto the market.

A new Standard is a great help towards the assurance of durable structures, but diligence and sound judgment by the design engineer is needed in the application of all Codes and Standards. There is no substitute for the experience, and the opportunities taken to learn from failures, problems and successes of previous building designs.

TABLE I

GARAGE	A	B	C
STRUCTURE TYPE	Concrete flat slab, normal reinforcing, nominal 20 mm cover, no surface protection, nominally flat	Precast concrete TT units on precast columns and prestressed girders, in-situ topping, no surface protection	Post-tensioned cast-in-place slabs on normally reinforced columns and beams
Framed area	60,000 m^2	18,000 m^2	22,000 m^2
Year completed	1975	1975	1972
Year major repair work begun	1980	1987	1982
Direct costs to date for repairs and protection	\$80.00/m^2	\$64.00/m^2	\$374.00/m^2 (Plus approximately \$400.00/m^2 indirect costs)
Estimated "avoided" costs to provide better details and protection in accordance with knowledge available at the time of construction	Less than \$10.00/m^2	Less than \$10.00/m^2	Less than \$12.00/m^2

Analysis of Bridge Beams with Jointless Decks

Dimensionnement de poutres de ponts à tablier continu

Berechnung von Brückenträgern mit kontinuierlicher Fahrbahnplatte

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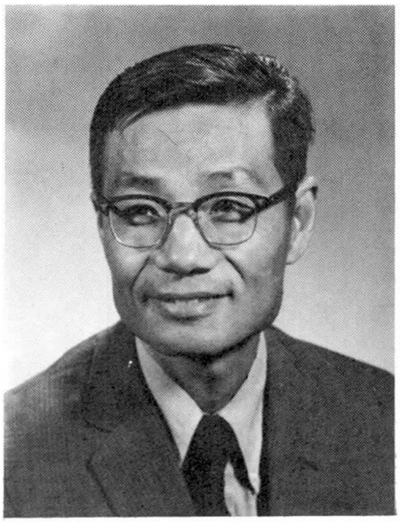

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SUMMARY

The use of beams with jointless decks is presented as an alternative solution for the construction and rehabilitation of multispan bridges. A finite element numerical approach is used for the determination of the instantaneous and time-dependent responses and for the strength analysis of such beams. Analytical results are shown which demonstrate the effectiveness of the method and conclusions are drawn concerning their overall performance.

RÉSUMÉ

L'usage des poutres de ponts, sans joint, à tablier continu présente une alternative pour l'exécution et la réparation de ponts à plusieurs travées. Une méthode numérique, avec des éléments finis, est employée pour la détermination des réponses instantanées et dépendantes du temps et pour l'analyse des états-limites ultimes de ces poutres. Des solutions analytiques mettent en évidence l'efficacité de la méthode. Quelques conclusions sont données sur le comportement de ces structures.

ZUSAMMENFASSUNG

Die Verwendung von fugenlosen Brückenträgern mit kontinuierlichen Fahrbahnplatten stellt eine alternative Lösung für den Bau und die Instandsetzung von Brücken über mehrere Felder dar. Ein finites Elementmodell wird zur Berechnung von anfänglichem und zeitunabhängigem Verhalten verwendet, welches schliesslich zur Analyse der Bruchzustände solcher Träger führt. Analytische Ergebnisse werden dargestellt, die den Erfolg der Methode beweisen und einige Schlussfolgerungen über das Tragverhalten dieser Träger erlauben.

1. INTRODUCTION

The use of jointless construction [1,2] for composite bridge beams has been considered as a possible solution for the persistent bridge maintenance problems due to the existance of expansion joints [3,4]. Expansion joints, regarded as an indispensable design requirement for the proper behavior of bridges, have always been a cause for deterioration of such structures.

Jointed bridges may become uneconomical due to the presence of joints and the ensuing maintenance problems resulting therefrom. The concept of a jointless structure, however, even though demanding some higher effort as far as design and analysis are concerned, has presented numerous advantages on its construction, performance and maintenance.

The idea of eliminating structural joints in the bridge deck, presents yet another interesting possibility. Partial continuity could also be obtained by simply casting a fully-continuous deck over the simply supported girders [2]. Such an unconventional constructional procedure,referred herein as "Deck-Continuous Beams", may prove to be an economical solution not only for the construction of new bridges but also for the rehabilitation of old ones.

Simple and fully-continuous beams, composite or not, under linear and uncracked conditions, can be analyzed satisfactorily by standard methods of theory of structures. Under non-linear and cracked conditions, however, numerical solutions are generally necessary. When only partial continuity is obtained, as is the case of deck--continuous beams, conventional analytical methods are no longer applicable, even for the most simple situations.

The purpose of this paper is to present the results of a full-range analysis of deck-continuous beams [5]. For such analysis a finite element numerical approach is developed. The beams may also be composite, continuous or not, in steel, reinforced or prestressed concrete. Girders may be pre- or post-tensioned, fully or partially prestressed,with bonded tendons. Various constructional sequences may be studied,supporting conditions may be varied, and different loading arrangements assumed. Instantaneous and time-dependent responses are obtained. Time-dependent material properties may be varied, and temperature effects can be included.

2. ANALYTICAL MODEL

A deck-continuous bridge beam may be composed of cast-in-place or precast girders of reinforced or prestressed concrete or steel girders, topped by a concrete deck-slab. Construction sequences and techniques may be varied, as well as cross-sectional shapes and material properties.

In the analysis, the structure is modeled by two distinct elements: a two-noded isoparametric beam element represents cross-sectional and material properties of the girders and deck, whereas the deck portion, connecting the adjacent girders, is modeled by a two noded, uniaxial, spring-like element. Both elements have their stiffness matrices modified to account for a variable nodal position, imperative in representing the actual supporting conditions of a general beam. The presence of pre- or post-tensioned tendons is obtained by a matrix superposition and three levels of mild reinforcement are also considered.

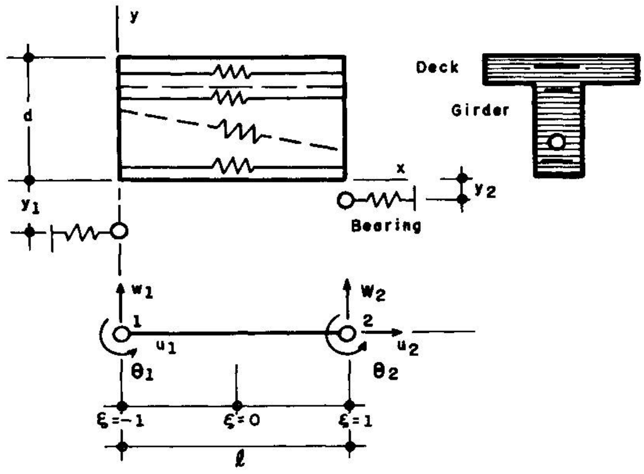

Fig. 2.1 - Beam element

Cracking is assumed through the Smeared Cracking Model and two different constitutive relationships are used for both girder and deck concretes.

The solution for instantaneous, static loading, is obtained by either increments of load or displacement, using the tangent stiffness matrix of the system and covering both the linear and nonlinear ranges of behaviour of the members. The effects of support displacements and temperature variation are also included in the instantaneous analysis. Time-dependent analysis considers the effects of aging of concrete, differential creep, differential shrinkage and prestressing steel relaxation. A time incremantal procedure is assumed and the suggested models of ACI Committe 209 [6] and PCI Committe on Prestress Losses [7] are adopted for both concrete and steel properties, respectively. Different loading sequences and construction stage may be predefined and solved in one single analysis, for a general beam type of any number of spans.

The model is validated by the analysis of eighteen different beam cases, results have shown in very close agreement with analytical and measured data, as shown in details in reference [5].

3. RESULTS

A deck continuous beam, as shown in Fig. 3.1 for only two spans may, under vertically applied loading, behave in two different ways: compression or tension may be induced in the deck connection between two adjacent girders. Such situations are primarily dependent on the supporting conditions and each dictates a different behavior of the structures. As illustrated in Fig. 3.2, a combined bending and rigid-body movement of the girders, supported at their bottom flanges, may produce either a pulling or squeezing effect on the deck connection, condition that can be set as a design variable.

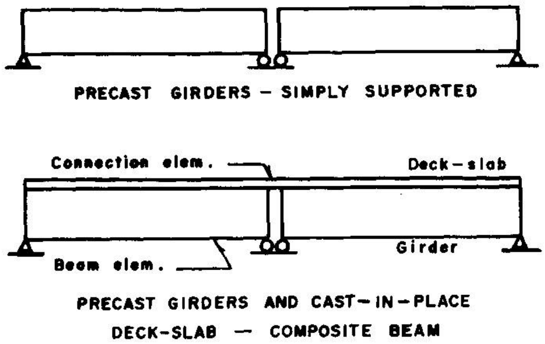

Fig. 3.1 - Deck-Continuous Beam

Compression in the deck-connection is obtained by allowing both girders to move inwards as in Fig. 3.2(b). Such situation produces an extra compressive force component which enhances the stiffness capacity of the member under bending. Should the structure be overloaded, however, such increasing compressive effect may overcome the compressive strength of the concrete connection, inducing an early failure of the member without enough ductility capacity (see Fig. 3.3, case c). Here failure is assumed when the ultimate compressive strain of the concrete is reached.

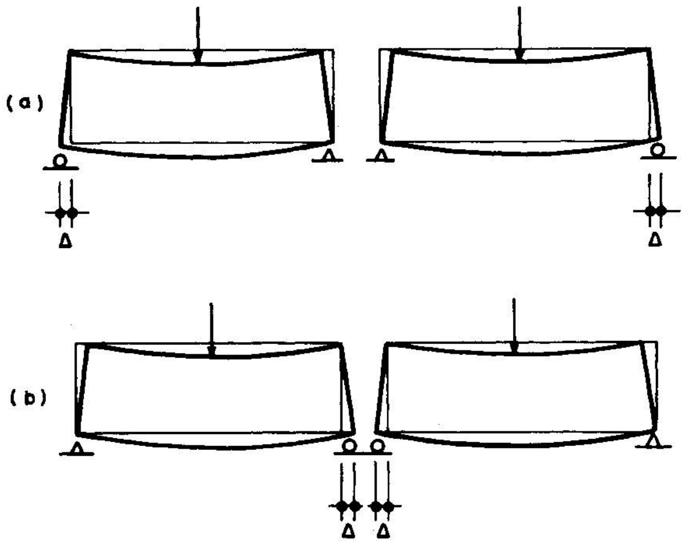

Fig. 3.2 - Girder movements under different supporting conditions.

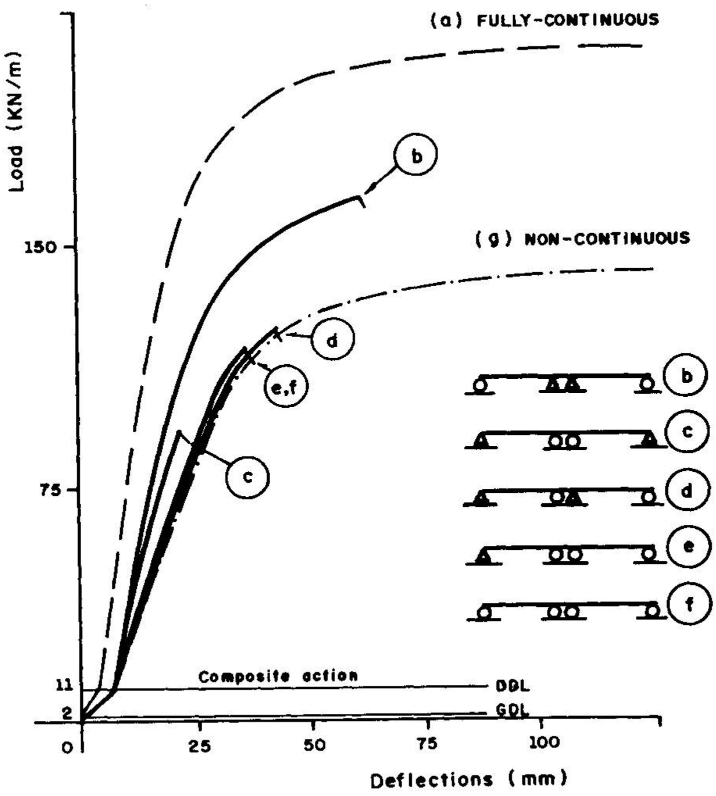

Fig. 3.3 - Load-deflection responses for various support arrangements, under unshored deck construction and full-span loading.

By permitting both girders to move apart from each other, when supported as in Fig. 3.2(a), tension is induced in the deck connection, equilibrated by coupled reactive forces at the interior supports. In this situation, the negative moment created at the connection also helps increasing the structure's bending stiffness. It makes use of the high tensile capactity of the negative mild reinforcement rather than the weak contribution from the concrete material.

Both cases, under compression and tension of the deck connection, are compared with the upper and lower bound cases of full and no continuity, respectively. As seen in Fig. 3.3, several load--deflection relationships are shown, corresponding to the different cases: girders and deck are fully-continuous (a), the deck is fully--continuous but over two simple girders (b,c,d,e and f) and there is no continuity whatsoever (g), both composite beams are simply supported. All three cases correspond to a two 15,25m span beam with W33 x 118 steel girders and a 2,13m by 0,18m reinforced concrete deck slab. Unshored construction is assumed and the load is uniformly distributed.

As seen in their instantaneous load-deflection responses, the tensile behavior of the concrete deck connection (case b) presents a reamarkable enhancement in strength and ductility, with slightly greater bending stiffness, as compared to the compressed connection (case c). Failure, in this case, is likely to be determined by yielding of the mild reinforcement in the connection.

Similar behaviors are found for beams containing more then two spans, with either reinforced, prestressed or steel girders. The effects of temperature variation and the time-dependent effects from creep and shrinkage have been observed to produce, in the deck-continuous beams (case b) , the same type-behavior that would be found if the beams were fully-continuous, as in case (a).

By having the girders simply supported at all aupports, i. e. by providing elastomeric bearing pads or rollers, as in case (f),the structure's response is observed to be slightly stiffer than the one presented by the jointed beams (case g). Cracking at the deck connection is expected under overload conditions; however, this seems to be less demaging to the structure than the presence of a joint. Should failure occur, by extensive yielding or rupture of the deck connection, the structure will provide all the ductility capacity as the jointed beams. This situation is likely to be found when jointed bridges are rehabilitated, by casting a fully--continuous deck-slab over the still serviceable girders.

4. CONCLUSIONS

The use a fully-continuous, jointless deck-slab has been presented as an alternative solution for the construction of composite bridge beams with cast-in-place or precast girders, as well as for the rehabilitation of jointed bridges. The behavior of deck-continuous beams has been observed to be very satisfatory under dead and service load stages. The behavior is primarily affected by the external boundary conditions. Under some specific conditions of the beams, a conservative and simplified design approach could be used, as to consider the different spans as independent simply supported composite beams. Should a more realistic design approach be needed the structure shall be analyzed by some appropriate numerical procedure, such as the one presented herein [5].

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Quality Inspection of Concrete Bridges and Wharfs in Norway

Auscultation de ponts et quais en béton en Norvège

Qualitätsuntersuchungen von Betonbrücken und Werften in Norwegen

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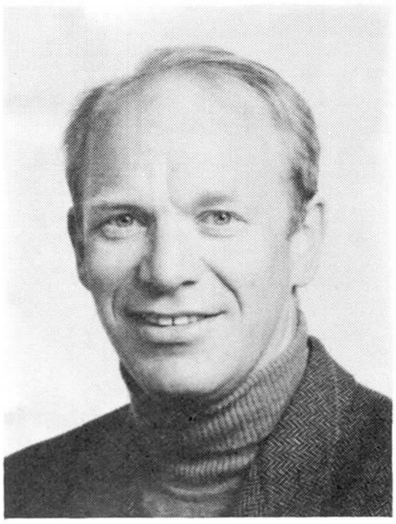

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SUMMARY

Results from the quality inspection of 35 bridges and 27 wharfs in Norway, varying in age and location, are reported. The results may be used as a basis for improvement of quality assurance, better design and detailing, changing of specifications and planning of maintenance and rehabilitation.

RÉSUMÉ

Les résultats de l'auscultation de 35 ponts et 27 quais d'âges et d'emplacements divers en Norvège sont présentés. Ils peuvent servir de base à l'amélioration de la qualité, à de meilleurs projets et détails constructifs ainsi qu'à l'adaptation des prescriptions et de la planification de l'entretien et de la remise en état.

ZUSAMMENFASSUNG

Die Resultate von Qualitätsuntersuchungen an 35 Brücken und 27 Werften verschiedenen Alters und Standortes in Norwegen werden vorgestellt. Sie können als Basis zur Verbesserung der Qualitätssicherung, des Entwurfsund der Konstruktionsdetails sowie zur Anpassung von Ausschreibungen und zur Plannung von Unterhaltung und Instandsetzung verwendet werden.

1. INTRODUCTION

Concrete bridges and wharves have been built for more than 80 years in different locations and exposed to different environments in Norway. The structures have been built according to existing codes and standards. The main purpose with the quality inspection has been to look for deterioration and analyse the reasons. The information will be used to improve the quality of future structures by introducing better quality assurance systems, by better design and detailing and by changing codes and specifications. For existing structures the information may be used for planning maintenance and rehabilitation.

2. TEST PROGRAM

2.1 Inspection

The inspection at the structure included a general visual survey to give an overall condition, a more detailed examination of deteriorated areas, a half cell surface potential mapping for detecting the corrosion situation of the rebar and, rebar cover measurements using a covermeter. From different locations at the structures, cores were drilled for further examination and testing in the laboratory.

2.2 Laboratory testing

The laboratory testing of cylinder cores included measurement of compressive strength, capillary adsorption, carbonation depths and chloride content. Capillary adsorption is of more present interest than water permeability. Carbonation was measured by the phenolpthalein method and chloride content by the Quantab test.

3. RESULTS

3.1 Bridges

Bridges from two areas in Norway have been inspected. In the western county of Hordaland, the survey included 20 bridges built in the period from 1930 to 1975 and located in the environmental zones outward and inner coast, inner fjord and inland. Most of the bridges were located in the costal zone. In the eastern county of Telemark, 15 bridges built in the period from 1940 to 1975 were inspected. The bridges were located in the environmental zones inner fjord, inland and higher inland. Some interesting information is shown in Table 1. More detailed information is given in /1 and 2/.

3.2 Wharves

27 wharves along the Norwegian coast, most of them in the northern part of Norway, have been inspected. The wharves were built in the period from 1920 to 1984. Some interesting information is shown in Table 2. More detailed information is given in /3 and 4/.

4. DISCUSSION

In spite of the relatively high number of structures, the variables are so many that a detailed discussion is impossible. More general

Table 1. Test results from bridges

No	Location 1)	Building-period	Strength (MPa)	Carb min/max (mm)	Max Cl⁻ close to surface (% of concr)
1	H - OC		41	0/ 8	0.05
2	H - IC	1930-39	39	0/15	0.07
3	H - OC		64	1/15	0.22
4	H - I		28	10/32	0.15
5	H - IC		56	2/32	0.08
6	H - IC	1940-49	33	2/30	0.14
7	T - I		32	3/ 7	0.08
8	T - I		47	1/10	0.06
9	T - HI		40	0/22	0.04
10	H - IF		41	2/13	0.18
11	H - IC		69	0/ 0	0.05
12	H - OC		90	0/ 2	0.19
13	H - OC		72	0/ 8	0.20
14	H - I	1950-59	23	0/ 8	0.11
15	T - IF		37	8/10	0.02
16	T - IF		40	4/22	0.07
17	T - HI		71	0/ 4	0.18
18	H - IF		61	0/ 8	0.11
19	H - OC		27	0/16	0.05
20	H - OC		55	0/ 0	0.13
21	H - IC	1960-69	24	12/31	0.05
22	H - OC		50	0/ 1	0.27
23	T - IF		46	3/20	0.08
24	T - IF		45	4/ 7	0.09
25	T - I		45	0/ 5	0.12
26	T - HI		44	0/ 4	0.06
27	T - HI		46	8/ 8	0.17
28	T - HI		49	0/ 3	0.07
29	H - IF		54	3/ 4	-
30	H - OC		73	2/ 6	0.05
31	H - OC		33	0/15	0.06
32	H - OC	1970-79	77	0/ 1	0.07
33	T - IF		48	0/ 5	0.03
34	T - I		64	0/ 9	0.14
35	T - I		42	8/ 9	0.02

1) H - Hordaland
T - Telemark
OC - Outward coast
IC - Inner coast
IF - Inner fjord
I - Inland
HI - Higher inland

Table 2. Test results from wharves

No	Building-period	Strength (MPa)	Max Cl⁻ close to surface (% of concr)
1	1920-29	-	-
2	1930-39	-	0.19
3	1950-59	55	0.52
4		52	0.13
5		57	0.14
6		47	0.21
7		38	0.12
8		45	0.20
9		44	0.06
10	1960-69	58	0.10
11		53	0.47
12		-	-
13		-	-
14		-	-
15		65	0.28
16		-	0.23
17		50	0.36
18		46	0.10
19		55	0.23
20	1970-79	70	0.18
21		51	0.40
22		53	0.10
23		44	0.44
24		50	0.27
25	1980-82	53	0.31
26		59	0.48
27		45	0.13

trends, however, are of great interest.

4.1 Bridges

The general deterioration problem of the bridges is reinforcement corrosion due to high chloride content. Carbonation and frost deterioration were og minor importance.

The compressive strength was in the majority of the structures higher than specified. However, as shown in Table 1, the strength values varied quite a lot.

The environmental zone seems to have a consistant effect on chloride penetration. The most severe environment is outward coast (OC), diminishing towards the inland. However, in some cases the bridge slab in inland bridges has a high chloride content due to summer salting in order to reduce dust on gravel roads. Also high chloride content, probably due to the use of accelerators during construction, have been found.

Carbonation rate is found to be highest in the inner cost zone. Bridges built in the period 1940-49 have the highest carbonation depts due to lack of cement during and after the second world war. This resulted in a higher w/c-ratio and a poorer quality. The correlation between carbonation dephts and concrete quality was as expected.

The concrete cover was found to vary quite a lot. In most of the bridges, the measured cover was satisfactory with respect to existing code during construstion. However, it is clear that specified cover has been too low. In the new Norwegian code, the specified cover in the actual environmental class is increased to 40 mm and 50 mm in the splash zone. This seems to be enough when combined with increased demand on concrete composition (reduced w/c-ratio to 0.45) and improved quality control.

The visual inspection revealed some common weak details in the structures. The most common was insufficient drainage systems from the top of the bridges. Drainage pipes with diameter 75 mm or lower were filled with scrap and blocked. Lack of protruding pipes under the bridges resulted in local high water content with freezing deterioration and mis-colouring. Reinforcement corrosion was most commonly found along the rim of the bridge slab sides. Insufficient concrete compaction had in many structures resulted in washing out of the hardened concrete, leaving white areas of lime. In structural details like sharp edges, the risk of deterioration was found to be very high. Also the fixing of steel railing to the bridge slab was found to be weak points where corrosion and concrete scaling were common. It is reasonable that freezing also may be a reason for the deterioration in such local areas.

4.2 Wharves

The main deterioration problem in concrete wharves is also reinforcement corrosion, first of all due to chloride ingress. The wharf slabs were commonly more deteriorated than beams and columns. Generally, the most deteriorated part of the slab was the inner part underneath due to splashing sea water. Therefore, the orientation of the wharves compared to the main wind direction is of importance. Heavy sea water splashing resulted in high chloride content and low electrical resistivity in the concrete, an ideal situation for rebar corrosion. Rebar corrosion was also found as a result of damage due to ship collision. This is not a material but may be a structural problem. Wharves should be designed so that the risk of damage due to ship collision is reduced or so that such structural parts may be replaced.

The compressive strengths were in most cases higher than specified, but the variation was relatively high as shown in Table 2.

Frost damage is a smaller problem than expected in spite of the fact that air entrainment is used in very few structures, especially in structures built before the middle of the fifties. The reason may be that the frost load is low due to the fact that the minimum temperature is relatively high close to the unfrozen sea water. Frost damages were located to special details like drainage pipes with insufficient protruding, along the lower rim where dripping noses were insufficient and along railroad tracks where deicing salts had been used.

Carbonation is found to be no problem in wharves. The reason seems to be a combination of a moist environment and a relatively high concrete quality.

The measured concrete cover varied a lot and the minimum values were frequently lower than specified. In general, the measured cover were lower in the bottom of soffit slabs than in beams and columns. This is in correlation with the most severe deterioration in the wharf slabs.

Cracks due to different reasons were observed in the majority of the wharves. The most common reason seems to be plastic and drying shrinkage, thermic cracking, deformation of the base and overloading compared to design specification.

The criteria for designing and detailing have primarily been based on strength requirements. From a durability point of view, this is normally not sufficient. The reasons for deterioration are mostly due to environmental and not to static loads. Important keywords are detailing like water drainage, location/direction of the wharf in the environment, concrete quality and good workmanship.

In some of the newer structures, silica fume has been used. The number of wharves and exposure times are too limited to draw conclusions, however, based on numerous research reports it is expected that the use of silica fume will reduce the ingress of chlorides considerably.

5. CONCLUSIONS

35 bridges and 27 wharves in Norway have been inspected and tested the last few years. From the results it can be concluded that during design, more attention has to be paid to durability, environmental loads and detailing.

Concrete cover was in most cases too low. In the new Norwegian code, the specified cover is increased to 40 mm and 50 mm in the splash zone for the actual environmental class. This seems to be sufficient in most cases when combined with the specified increased material quality. The new Norwegian code specifies a w/c-ratio lower than 0.45 in the actual environmental class. When carbonation is the limiting factor, this is sufficient requirements. However, regarding chloride penetration, the w/c-ratio should not exeed 0.40 which also is specified in the new design code from the Norwegian Public Road Administration. This specification is also recommended for wharves. In a planned submerged floating tube for public trafic across a fjord in Norway, the specifications may be even stronger.

A combination of a blended cement and silica fume as well as entrained air is recommended, especially where the structure is exposed to saline water.

Quality assurance and quality control both during design and construction are of great importance in order to achieve a satisfactory result.

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Design of High Masts Needing no Maintenance

Projet de mâts élevés ne nécessitant aucun entretien

Projektierung von unterhaltungsfreien hohen Masten

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SUMMARY

It is not very often you are faced with requirements of structures which need no maintenance for 50 years. However, when it is a question of a new transmitter network for the Danish television comprising twelve 300 m high masts a serious effort is made. Planning and design of these masts ensure such a durability. The article describes structures, choice of material and protection against corrosion as well as the databased information system which records all information on the masts and antennas from fabrication and through the whole operation phase.

RÉSUMÉ

Des conditions qui comportent une durée de vie de cinq décennies sans entretien sont peu ordinaires pour les structures. Cependant lorsqu'il s'agit d'un nouveau réseau de télévision de 12 mâts, chacun d'une hauteur de 300 m, de telles exigences sont à l'ordre du jour. La planification et la conception des mâts pour la nouvelle chaîne de télévision danoise, assure une telle durée de vie. L'article traite des structures, du choix des matériaux et de la protection contre la corrosion, ainsi que du système d'informations informatisé, qui enregistre l'ensemble des données relatives au réseau, depuis la fabrication et pendant toute la phase d'exploitation.

ZUSAMMENFASSUNG

Eine Forderung, dass Konstruktionen 50 Jahre lang unterhaltungsfrei bleiben, ist nicht alltäglich. Handelt es sich indessen um ein neues Fernsehmastennetz von zwölf 300 m hohen Masten, wird eine ganz ausserordentliche Leistung erforderlich. Planung und Projektierung der neuen dänischen Fernsehmasten sichern eine solche Dauerhaftigkeit. Der Artikel beschreibt Konstruktionen, Wahl von Materialien und Korrosionsschutz sowie das databasebasierte Informationssystem, das alle die Anlagen betreffenden Bedingungen — von der Fabrikation durch die ganze Betriebsphase hindurch — registriert.

1. INTRODUCTION

In 1986 the Danish Government enacted a law for the establishment of a transmitter net-work for a second national television channel. The net-work is being established from 1987 to 1989.

In the planning, design and fabrication of the supporting steel structures the stringent precautions have been payed to long durability. Moreover it has been the aim to design masts which were easy to inspect and needed practically no maintenance.

Due to the requirements of high reliability great importance has been attached to the durability. At the same time it has been possible to keep the construction costs and the maintenance costs at a minimum.

The Danish Teleadministrations has awarded the contract for the detailed design of the structural works to the consulting engineers Rambøll & Hannemann.

2. DESCRIPTION OF THE PROJECT

The new transmission net comprises 12 new stations, each with a 300 m high mast carrying the TV 2 antenna.

The overall application of round bars makes the form of the very high masts remarkable. The structure is simple and appears on the whole light and elegant. Furthermore we have designed a structure which needs practically no maintenance and at the same time introduced the World's best guys. All details have been carefully analysed to maximize the structure to fulfil all requirements with regard to function (static, dynamic and access), fabrication, erection and maintenance.

3. FUNCTIONAL REQUIREMENTS

The basic requirements to function of the masts are simple :

- The UHF-antenna supported inside a 18 m high glass-reinforced plastic cylinder, 1.6 m in diameter shall be placed 300 m above ground.
- A hoist for 3 persons/500 kg shall run from bottom to top.
- A ladder with safety cage shall be installed from base to top.
- Various antennas may be installed all over the mast.
- Besides the 2x5 inches feeders for the UHF-antenna, cables, feeders, waveguides, etc. will be needed for the other antennae.
- Easy and safe conditions for working in the mast must be fulfilled.
- The masts shall be able to withstand a rupture of one guy.

The economical requirements are even more simple :

- The total construction costs shall be the lowest possible, and
- The maintenance costs shall be the lowest possible

Cross Section

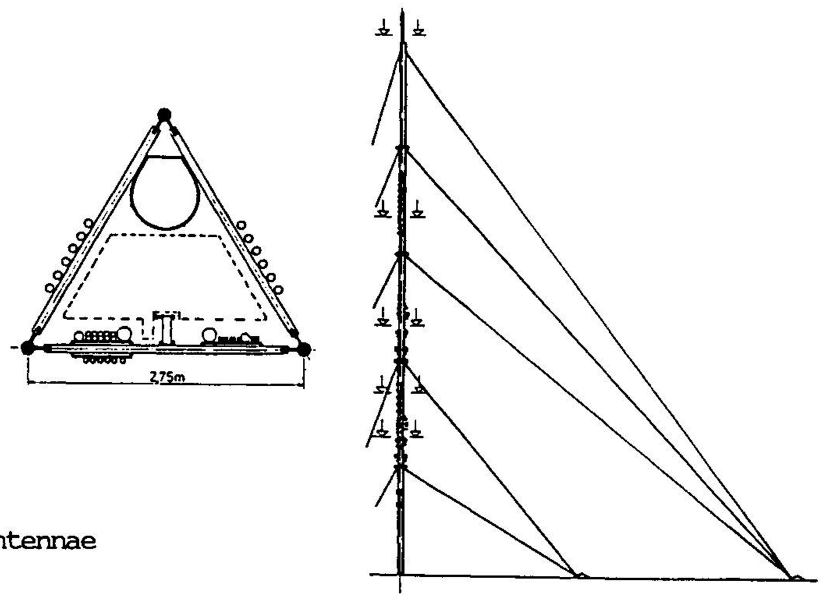

Fig. 1 300 m MAST, Antennae

4. STRUCTURAL OUTLINE

The latticed mast has a triangular cross-section with a face width of 2.75 m to the top. It is guyed in three directions at five levels, and the base is pinned to the mast foundation. A continuous ladder with backguards is placed inside the cross section. If you don't feel like climbing the 1000 steps, you can use the diesel-hydraulic hoist, which runs inside the cross section and may be stopped at arbitrary levels. From the working platform on top of the hoist installation and inspection works may be undertaken.

The leg members are solid rounds with a diameter of 150 mm from base to top.

The diagonals in the V-bracing are solid rounds with a diameter of 85 mm. At the bottom 150 m of the shaft horizontals reduce the buckling length of the legs.

The guy ropes are spiral stands. The diameter of the guys is 43 mm except for the second lowest set, where the diameter is 55 mm.

5. DURABILITY OF THE MAST

When planning and designing the mast special attention is paid to maximize the protection against corrosion, so that the masts need no maintenance for a period of at least 50 years.

We decided on a latticed mast of round bars as the best solution :

- The surface of the structural steel-work is smooth and has been reduced compared to other steel sections. Therefore there is no collection of dirt or moisture anywhere or risk of condensates as e.g. is the case in tubes.

- The guys and their attachments have been subject to careful analysis, one of the reasons being the various collapses due to guy ruptures and break of the guy attachment.

- Wind loads on the mast are reduced to a minimum and the mast is at the same time dynamically completely stable.

6. PROTECTION AGAINST CORROSION

Maintenance of the surface treatment of guyed masts to ensure proper protection against corrosion is extremely expensive. It is very often necessary after some years to clean, sandblast and paint the masts. Maintenance of guy ropes is extremely difficult and replacement of worn-out guys is often the best solution. Therefore special attention is paid to the surface treatment and choise of material.

All structural steel-work is hot-dip galvanized with a heavy layer of zinc. A minimum thickness of 250 microns (approximately 1800 g/m^2) is achieved by specification of the chemical compositon of the steel and the dipping time in the zinc bath. Painting is avoided. The ladder with backguards, cable ladders, bolts, nuts, clamps etc. are made of stainless steel. Thus the mast needs no maintenance for a period of at least 50 years in normal aggressive surroundings.

The problem left is the guy ropes. Stainless steel wires are too expensive. Until a few years ago the only available solution was hot-dipped galvanized steel wires with a rather thin (approx. 300 g/m^2) cover of zinc. Recently heavy galvanized wires - approx. 600 g/m^2 - were available at reasonable prices. With a lifetime of 25-40 years such guys are almost satisfactory and have been used in Denmark for 5-6 years.

The guy ropes for the 300 m masts have an even better protection. The wires are hot dipped in an alloy of 95% zinc and 5% aluminium resulting in a cover of 400 g/m^2. This alloy "GALFAN" is at least 2-3 times more effective than pure zinc. Furthermore during the stranding the individual wires are layed in a special compound, "NYROSTEN", to ensure that the finished rope is without any hollow parts. After the stranding the whole surface of the rope is covered with the same compound. Various tests are undertaken during the fabrication of the thickness and adhesion of the surface treatment. It is expected that the lifetime of these guys is 40-60 years.

The guys for the new 300 m masts are the first of this type in the World and definitely quite outstanding. It is to be mentioned that the technical specifications as well as the demands on surface treatment were set up by Rambøll & Hannemann. This new design has literally contributed to raising the level of the quality of the guys and at competitive prices.

Also the diesel hydraulic hoist is designed to have a maximum life time. The cabin is made of stainless steel plates and the working platform on top of the cabin is of alumimium.

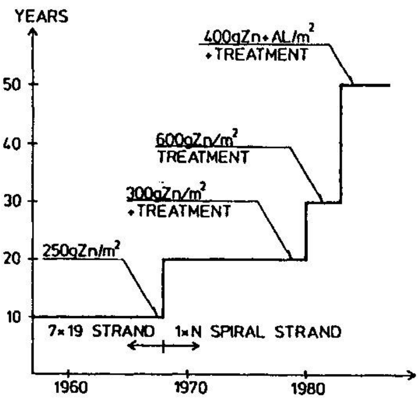

Fig. 2 Lifetime of Guy Ropes

As an extra security the structure is analysed for the dynamic forces immediately after one guy suddenly breaks due to unforeseeable stresses or errors.

7. FUTURE INSPECTION

The antennas in the 300 m masts is such a vital part of the national communication system and reliability is a must. Therefore systematic inspections with intervals of 3-5 years are carried out.

In the new 300 m masts inspection work is easy and quick.

First access to the mast is extremely favourable. From the platform on the hoist almost everything in the mast can be inspected, as the hoist can be stopped at arbitrary levels. The guys can be inspected from a manned chair which can be drawn up and down the guy.

Second very few irregularities will occur, as normally no maintenance is necessary.

8. COMPUTERIZED INFORMATION SYSTEM

A data based information system has been developed to secure that all useful and updated information on the masts and antennas is systematically registrated and stored safely and well-planned in a computer.

The information includes e.g.:

- Basis and design data
- Control reports and inspection data from the fabrication phase
- All registrations from the regular inspection work and operation phase
- Maintenance work, if any, or alterations of structure.
- Information with exact detail drawings of all antennas and cables in the mast

In the operation phase the information system ensures a safe and efficient maintenance work, e.g. the computer tool quickly produces a view of similar elements in all the masts (antennas, cables, connections etc.) if an inspection on one location shows that special attention is demanded.

Also the system makes it possible regularly to analyse loads and structures if changes are wanted and thereby fully secure the durability aimed at.