THEME 1

Fatigue Codes and Design Concepts

Normes et concepts de dimensionnement à la fatigue

Normen und Konzepte für die Ermüdungsbemessung

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Background of American Design Procedure for Fatigue of Concrete

Principe de la méthode américaine de dimensionnement à la fatigue du béton

Über das amerikanische Ermüdungsbemessungsverfahren für Betontragwerke

W. GENE CORLEY
Director
Portland Cement Assoc.
Skokie, IL, USA

JOHN M. HANSON
President
Wiss, Janney, Elstner & Assoc.
Northbrook, IL, USA

THORSTEINN HELGASON
Associate Professor
University of Iceland
Reykjavik, Iceland

SUMMARY
The procedure currently used in North America for considering fatigue of reinforcement in concrete highway bridges was adopted in 1975. The procedure was based on an extensive experimental investigation to determine the fatigue strength of U.S. manufactured, hot rolled, deformed, reinforcing bars. This paper summarizes results of the test program and presents the statistically developed design procedure. Additionally, an example of the use of this provision in American practice is provided.

RESUME
La méthode couramment utilisée en Amérique du Nord pour prendre en considération la fatigue des aciers d'armature dans les ponts en béton fut adoptée en 1975. Cette méthode est basée sur une vaste étude expérimentale, entreprise en vue de déterminer la résistance à la fatigue des barres d'armature fabriquées aux USA, en acier laminé à chaud et déformé. Cet article résume les résultats expérimentaux et présente la méthode de dimensionnement développée sur une base statistique. On présente en outre un exemple pratique d'application de cette méthode.

ZUSAMMENFASSUNG
Das zur Zeit in Nordamerika angewandte Verfahren zur Berücksichtigung der Ermüdung der Stahleinlagen in Betonbrücken kam erst 1975 in Gebrauch. Das Verfahren wurde aufgrund einer umfangreichen experimentellen Untersuchung zur Ermittlung der Dauerfestigkeit der in den USA hergestellten Rippenstähle entwickelt. Die Ergebnisse der Versuche und das statistisch entwickelte Verfahren werden im Aufsatz beschrieben. Im weiteren wird ein Beispiel zur Anwendung des Verfahrens für amerikanische Verhältnisse angegeben.

1. INTRODUCTION

Early bridge design specifications did not need to consider fatigue because 125 to 140 MPa allowable design stresses were too low to present a danger of fatigue fracture. To date, no fatigue damage of a concrete bridge in regular service has been identified. However, Grade 40 bars were placed in two reinforced concrete test bridges [1] in a Road Test conducted by the American Association of State Highway Officials (AASHO) in the late 1950's. Some of these bars fractured in fatigue after repeated application of very heavy loads to the bridges following completion of the planned field tests.

More recently, high yield stress reinforcing bars have come into usage, load factor design methods are permitted, and heavier trucks are allowed on North American highways. Together, these factors result in repeated stresses that approach those known to cause fatigue fracture in reinforcing bars.

In 1974, American Concrete Institute (ACI) Committee 215 published a state-of-the-art report on fatigue of plain and reinforced concrete [2]. Based on data available at that time, the Committee recommended that the stress range for straight deformed reinforcing bars be limited to 145 MPa. This limit was adopted by AASHTO in the 1974 Interim Specifications [3].

In 1976, results of an extensive investigation [4] to determine the fatigue strength of U.S. manufactured hot rolled deformed reinforcing bars were published. This investigation was carried out by the Portland Cement Association (PCA) and sponsored in part by the National Cooperative Highway Reseach Program (NCHRP). The work included a review of the literature, 353 tests on bars embedded as a single reinforcing element within a concrete beam, and a statistical analysis of the resulting data.

2. TEST VARIABLES AND PROCEDURE

Bars shown in Fig. 1, from five U.S. manufacturers, were tested. One of the manufacturers was represented by nominal 16, 25, and 33 mm bars having guaranteed yield stresses of 276, 414, and 517 MPa and by nominally 19 and 32 mm bars having a guaranteed yield stress of 414 MPa. The other manufacturers

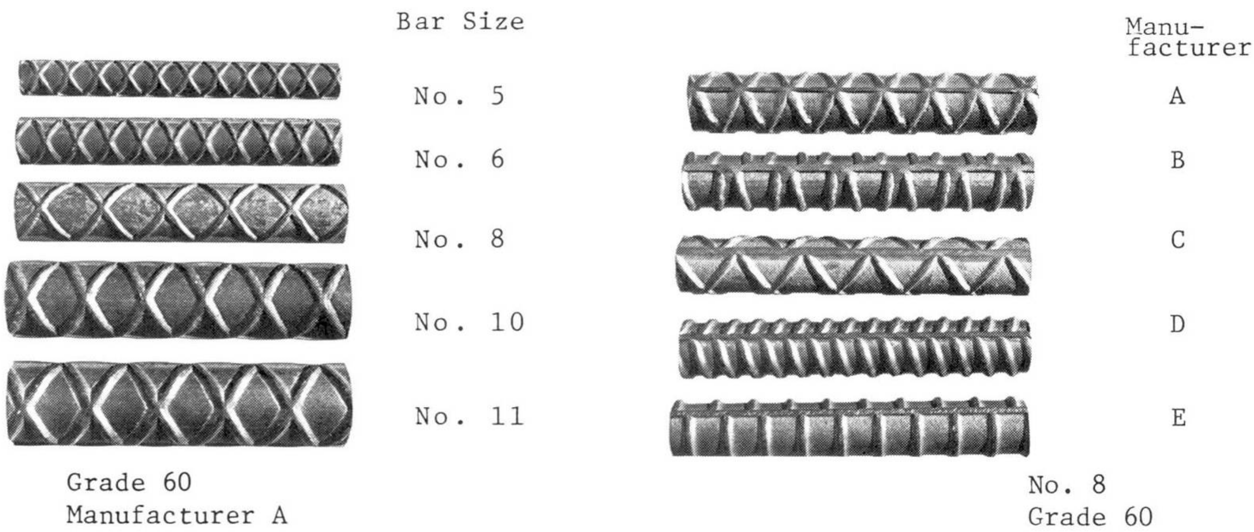
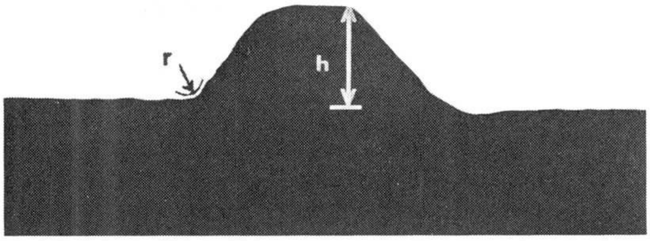


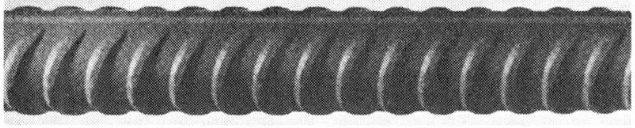
Fig. 1 Reinforcing Bars Used in Test Program

were represented only by nominally 25 mm bars having a guaranteed yield stress of 414 MPa. Each manufacturer's bars had a distinctive rib pattern.

At the time that the investigation was carried out, hot-rolled deformed bars were made by approximately 50 manufacturers in the U.S. These bars have a wide variety of deformation patterns. Typically, the patterns include two longitudinal ribs and transverse lugs either perpendicular to the ribs or inclined at an angle of not less than 45 degrees with the longitudinal axis of the bar. Bars that were tested represented a range of transverse lug geometries. Lug geometry was assessed by means of the ratio of base radius to rib height, r/h, defined in Fig. 2. This ratio ranged from 0.17 to 0.39 for the five manufacturer's bars.



(a) Lug Profile



(b) Lugs and Rib

Fig. 2 Base Radius r and Lug Height h

Bars were tested by embedding them in a concrete beam and subjecting the beam to repeated loads. Each test beam was rectangular or T-shaped in cross section and had a nominal effective depth of 150, 250, or 450 mm. In Fig. 3, one of the beam tests is shown.

Loads were applied to the test beams using either one or two 100 kN capacity Amsler rams, depending on beam size. Loads varied sinusoidally and were applied at fixed nominal rates of either 4.2 or 8.3 Hz. In each case, applied loading produced a constant moment region in the central 1/3 of beam span. Each test beam was simply supported on rollers.

Tests were carried out in two series. In each series, order of testing and selection of bars was randomized to obtain a statistically valid test program.

Stress range in bars of Series 1 tests was varied to obtain an S-N curve in the finite life region for each test condition. Minimum stress levels were nominally 41 MPa compression, 41 MPa tension, and 124 MPa tension in finite-life tests of Series 1. When minimum stress in a test bar was compression, external post-tensioning was applied to the test beam. The post-tensioning system consisted of a pair of steel rods held at the level of the beam reinforcement and passed through steel springs butting against one end of the test beam. Prestress force was measured by load cells.

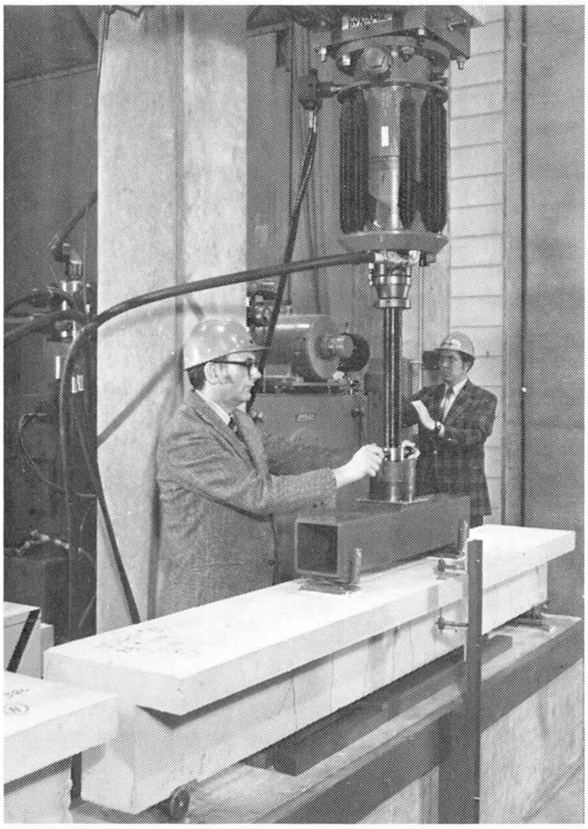


Fig. 3 Test Setup

In Series 2, stress range in bars was varied in increasing or decreasing steps of 7 MPa to obtain a series of staircase results around a fatigue limit of 5 million cycles. Stress range and rib geometry were the only variables in the staircase test series. All tests of Series 2 were conducted on nominally 25 mm bars, each embedded at an effective depth of about 250 mm and subjected to a nominal minimum stress of 41 MPa tension. In the staircase tests, stress range applied to a specific manufacturer's test bar depended in each case on results obtained in the immediately preceeding test on that manufacturer's bars. Thus, a runout at 5 million cycles resulted in a nominal 7 MPa increase in stress range for the succeeding test. Conversely, a fatigue fracture in a test bar resulted in a nominal 7 MPa decrease in stress range for the succeeding test.

3. TEST RESULTS

3.1 Finite-Life Tests

Stress range was found to be the predominant factor affecting fatigue life of each reinforcing bar. Statistical analysis of the test data showed that next to stress range, minimum stress level was the variable of greatest significance. Increasing minimum stress from compression through tension caused a statistically significant reduction in fatigue life. Effects of these and other test variables are presented in detail in Ref. 4.

Considering the effect of stress range alone, the relationship between the logarithm of fatigue life and stress range was found to be:

$$\log N = 6.9690 - 0.0383 f_r \tag{1}$$

where: N = fatigue life

f_r = stress range at centroid of reinforcing bar during stress cycle

This relationship explained 76.8% of the variation in test data. The standard deviation for the regression was 0.16557.

During the tests, 33 mm bars having a yield stress of 414 MPa fractured in fatigue after 1,250,000 cycles when subjected to a stress range of 147 MPa and a minimum stress of 121 MPa. This is the lowest stress range at which a fatigue fracture has been obtained in a straight U.S. manufactured bar. Fatigue fractures have been obtained at lower stress ranges in bent or welded bars.

Tests conducted at low stress ranges indicated that there is a limiting stress range, the fatigue limit, above which a bar is certain to fracture in fatigue, and below which a long fatigue life is possible. Subsequent tests [5] have confirmed that below the fatigue limit, a reinforcing bar may be able to sustain a virtually unlimited number of cycles of loading without fracture.

3.2 Staircase Tests

For the five manufacturer's bars tested, mean fatigue limit at 5 million cycles was found to range from 159 to 197 MPa. This variation had a strong correlation to the rib geometry factor, r/h. Assuming a normal distribution of data around each mean fatigue limit, upper and lower tolerance limits were established, with 95% probability that 95% of all possible test results on a particular manufacturer's bars would fall within the limits. For the five manufacturer's bars, the lower tolerance limit ranged from 136 to 184 MPa.

A linear regression analysis performed on the results of the staircase analysis, using fatigue limit, f_f, as the dependent variable, resulted in the following relationship:

$$f_f = 7.88 + 52.85(r/h) \tag{2}$$

However, this expression may place an undue emphasis on the effect of bar geometry since the effects of other potential influencing factors such as minimum stress level could not be considered.

4. FATIGUE DESIGN PROVISION

The current design provisions [6] of the American Association of State Highway and Transportation Officials (AASHTO) and the ACI-ASCE Committee on Bridge Design [7] require that stresses at service loads in reinforced concrete bridges shall be limited to the following:

4.1 Concrete maximum compressive stress shall not exceed $0.5f'_c$ at sections where stress reversals occur caused by live load plus impact at service load. This stress limit shall not apply to concrete deck slabs.

4.2 Reinforcement range between a maximum tension stress and minimum stress in straight bars caused by live load plus impact at service load shall not exceed:

$$f_f = 145 - 0.33f_{min} + 55(r/h)$$

where: f_f = stress range, MPa

f_{min} = algebraic minimum stress level, tension positive, compression negative, MPa

r/h = ratio of base radius to height of rolled on transverse deformation; when actual value is not known, use 0.3

Bends in primary reinforcement shall be avoided in regions of high stress range.

An example of application of these design provisions is given in the Appendix. Provisions apply only to straight hot-rolled bars with no welds and with no stress raisers (including manufacturers marks) more severe than deformations meeting the requirements of American Society for Testing and Materials (ASTM) Designation: A615.

5. DESIGN EXAMPLE

Application of the fatigue design provision is illustrated in the following partial design calculations for the main reinforcement in a bridge superstructure. For simplicity, a slab bridge was selected.

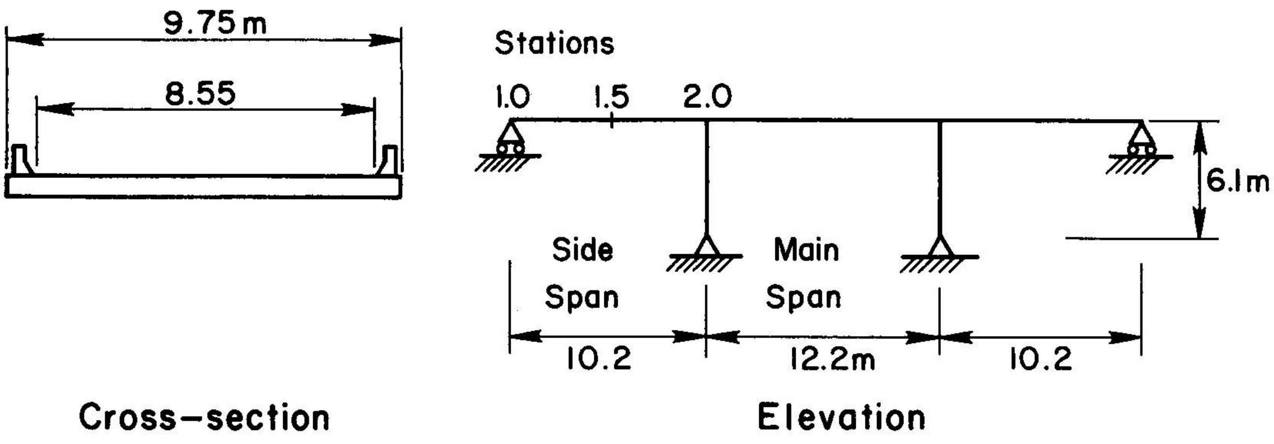


Fig. 4. Example Bridge

This 2 lane highway bridge is designed for HS 20-44 loading 6 . Material properties used in the design are f'_c = 20 MPa for concrete and f_y = 413 MPa for deformed reinforcing bars, assumed to have a lug base radius to lug height ratio, r/h, of 0.3. A 450 mm slab thickness was selected on the basis of maximum reinforcement and deflection criteria. From this and the HS 20-44 loading with an appropriate impact factor, dead load and live load effects at various stations along the span were calculated.

Concrete cover was 50 mm over negative moment reinforcement to account for severe exposure and 25 mm over positive moment reinforcement to account for moderate exposure. Reinforcement at maximum moment locations was selected on the basis of:

$$a = \frac{A_s f_y}{0.85 f_c b} \qquad M_u = \phi A_s f_y (d - 0.5a)$$

where ϕ is a capacity modification factor equal to 0.90 for flexure. At the centerline of the main span, station 2.50, the initial bar selection was:

$$25 \text{ c/c } 195 = 2518 \text{ mm}^2/\text{m}$$

$$a = \frac{2518 \times 413}{0.85 \times 20 \times 1000} = 61.2 \text{ mm}$$

$$M_u = 0.90 \times 2518 \times 413 \times (412 - 0.5 \times 61.2) = 357.0 \text{ kNm/m}$$

This selection was then checked against crack control requirements and found to be satisfactory. Next, stress range at service loads was calculated:

$$f_{sr} = f_{smax} - f_{smin} = \frac{M_{wmax}}{A_{s}jd} - \frac{M_{wmin}}{A_{s}jd}$$

$$= \frac{190.4 \times 10^6}{2518 \times 0.902 \times 412} - \frac{29.7 \times 10^6}{2518 \times 0.902 \times 412} = 203.5 - 31.7 = 171.8 \text{ MPa}$$

and checked agaist the allowable stress range:

$$f_f = 145 - 0.33 f_{smin} + 55 \text{ r/h}$$

$$= 145 - 0.33 \times 31.7 + 55 \times 0.3 = 151.0 \text{ MPa} < 171.8$$

The required reinforcement is:

$$A_s = \frac{2518 \times 171.8}{151.0} = 2865 \text{ mm}^2/\text{m}$$

an increase of 14% from the requirement for strength alone:

Similarly, the required reinforcement in the side span was:

$$22 \text{ c/c } 130 = 2923 \text{ mm}^2/\text{m}$$

providing a moment capacity of 411.2 kNm/m. There, the allowable stress range was exceeded at stations 1.3 to 1.6, for the original bar selection. Due to lack of symmetry in the side span moment diagrams, the critical fatigue location cannot be presumed to coincide with the critical strength location.

Selection of the reinforcement may be summarized in tabular form. Appropriate extensions for development must be provided. Minimum reinforcement and bar spacing considerations permit the main span positive moment reinforcement to be reduced by thirds. Every third bar may be terminated for strength at station 2.31. The cut bars are adequately developed from that point and crack control criteria are satisfied. Service load moment reversal takes place at station 2.31 with:

$$M_{wmin} = 11.4 \text{ kNm/m} \qquad M_{cr} = 93.5 \text{ kNm/m}$$

Therefore, the stress state can be determined as:

$$f_{smax} = \frac{M_{wmax}}{A_{sjd}} = \frac{139.8 \times 10^6}{1949 \times 0.912 \times 414} = 190.0 \text{ MPa}$$

$$f_{smin} = \frac{nM_{wmin}(d - h/2)}{I_g} = \frac{10 \times (-11.4) \times (414 - 225)}{7594} = 2.8 \text{ MPa}$$

$$f_{sr} = f_{smax} - f_{smin} = 190.0 - (-2.8) = 192.8 \text{ MPa}$$

$$f_f = 145 - 0.33 \times (-2.8) + 55 \times 0.3 = 162.4 \text{ MPa} \quad 192.8$$

The location where the bar may safely be cut for fatigue can be determined by trial and error or estimated from:

$$\frac{M_{wmax}}{A_{sjd}} \; 1 - 2.4\, n \; \frac{M_{wmin}}{M_{wmax}} = 145 + 55\, r/h$$

which is derived by setting $f_{sr} = f_f$ and making liberal use of the approximation $d = 0.9h$. Using momemt from station 2.31:

$$\frac{M_{wmax}}{1949 \times 0.912 \times 414} \; 1 - \frac{2.4 \times 10 \times 4.71 \times (-11.4)}{1000 \times 139.8} = 145 + 55 \times 0.3$$

$$M_{wmax} = 117.8 \text{ kNm/m}$$

which is found, by linear interpolation, to occur at station 2.277. A check shows that at station 2.275, f_{sr} exceeds f_f by only 0.5%, which is satisfactory. Other cut offs may be determined similarly.

In summary, checks for fatigue must be made at every stage of the design process. Greatest economy in design effort is obtained by bringing fatigue control directly into the reinforcement selection process, as illustrated here. Fatigue requirements may result in a need to increase reinforcement area beyond that required for strength and/or to extend bar cutoff locations.

6. CONCLUDING REMARKS

The design provision for fatigue in the current AASHTO specifications was initially adopted in 1974. This provision was based on an extensive investigation summarized in this paper. In this provision, the limiting stress range in reinforcing bars depends on the minimum stress level and the ratio of base radius to height of the transverse lugs.

7. REFERENCES

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American Concrete Institute Considerations for Fatigue

Considérations sur la fatigue par l' "American Concrete Institute"

Studien des "American Concrete Institute" bezüglich Ermüdung

NEIL M. HAWKINS
Professor
University of Washington
Seattle, WA, USA

SURENDRA P. SHAH
Professor
Northwestern University
Evanston, IL, USA

SUMMARY
Consideration relevant to the high cycle fatigue design of concrete structures have been developed by the American Concrete Institute's Committee 215 on Fatigue, 357 on Offshore Structures, and 443 on Concrete Bridge Design. The bases for those recommendations are described and findings from recent investigations that are likely to influence future recommendations are summarized.

RESUME
Des considérations relatives au dimensionnement à la fatigue des structures en béton, pour un nombre élevé de charges répétées, ont été faites par différentes commissions de l' "American Concrete Institute": commission 215 sur la fatigue, 357 sur les structures "offshore" et 443 sur le dimensionnement des ponts en béton. Les bases pour ces recommandations sont décrites et les conclusions de ces récentes recherches, qui vont probablement influencer de futures recommandations, sont résumées.

ZUSAMMENFASSUNG
Folgende ACI-Kommissionen haben Studien bezüglich der Ermüdungsbemessung von Stahlbetonkonstruktionen ausgearbeitet: 215 "Ermüdung", 357 "Offshore-Konstruktionen" und 443 "Stahlbetonbrücken". Im Beitrag werden die Grundlagen für die Empfehlungen des ACI beschrieben und Erkenntnisse aus neueren Untersuchungen, die voraussichtlich zukünftige Empfehlungen beeinflussen werden, zusammengefasst.

1. INTRODUCTION

Few structural failures attributable to fatigue have been reported in the U.S.A. Nevertheless, there is an increasing concern with repeated loading effects due to: (1) Increasing use of strength design procedures and higher strength materials; (2) Increasing use of concrete in marine environments, railroad bridges, crane girders, and other applications involving aggressive environments and repeated loads; and (3) Increasing recognition that repeated loads change crack widths, deflections, and stiffness at service loads.

The earliest U.S. recommendations were the state-of-the-art report developed in 1974 by ACI Committee 215 [1]. That report utilized research findings prior to 1972. It implied that the fatigue resistance of a structure could be directly related to the fatigue resistance of its component materials and that interaction effects resulting from differing repeated loading responses for those materials were small. The 215 report provided little information on serviceability considerations or the effects of the loading environment. In the early 1970's the American Association of State Highway Officials became concerned that, with increasing use of grade 60 reinforcing bars in bridges and with automatic issuance of permits for truck overloads on payment of fees, reinforcement in bridges was being subjected to stresses known to cause fatigue fracture in such bars. They sponsored an extensive investigation of the fatigue strength of U.S. manufactured deformed reinforcing bars at the Portland Cement Association [2]. That work, together with some ancillary investigations [3], formed the main basis for the fatigue provisions of the 1977 ACI Committee 447 report [4], and the AASHTO Code for Bridges [5]. The philosophy underlying those specifications was similar to that in the ACI Committee 215 report. Fatigue resistance is considered adequate if certain stress limitations are satisfied at sections subjected to significant cyclic strains. The latest ACI recommendations concerning fatigue are those developed by Committee 357 for Offshore Structures [6]. Those recommendations are based on the same philosophy as the 215 recommendations. They also include shear provisions based on Committee 215 recommendations [7], and the proviso that if fatigue resistance is a serious problem a more complete analysis using cumulative damage considerations can be substituted for the stress limitation approach. Serviceability requirements are imposed for the control of cracking and deformations for extreme imposed loading and frequently occurring environmental conditions. Thus, increases in crack width, decreases in stiffness, and changes in deformation with repetitive wave loadings must be considered.

ACI Committee 215 has developed suggested design recommendations for fatigue but not published those recommendations pending incorporation of findings from recent convention sessions in San Juan and Dallas. Those recommendations are summarized in Appendix A. This paper discusses the basis for those recommendations and possible impacts on them of recent research findings.

2. FATIGUE CHARACTERISTICS OF COMPONENT MATERIALS

The Committee 215 recommendations prescribe threshold values for stress ranges in component materials with the intention that for greater values, the potential for fatigue damage should be evaluated by comprehensive approaches (see Appendix A).

2.1 Concrete

When plain concrete is subject to cyclic compressive loading varying between a maximum stress f_{max} and a minimum stress f_{min} specimens fail after a certain number of cycles N depending on, among other things, the values of the maximum and minimum stress. The failure of concrete under repeated loading results from progressive microcracking [8]. Progressive damage is indicated by increasing strains at f_{max} and f_{min}, decrease in pulse velocity, increase in acoustic emission and a progressive decrease in the secant modulus of elasticity [9-11]. The increase in internal microcracking under fatigue loading is substantially higher than that under monotonically increasing (static) loading [8]. The increase in strain at f_{max} under high cycle fatigue loading exceeds the long-term creep strain due to f_{max}. Since there are no plastic deformations to blunt microcracks, concrete has no endurance limit similar to that for mild steel. The fatigue strength of concrete decreases almost linearly with the log of the number of cycles to failure. That action is often expressed in terms of an S-N curve (Wohler diagram). The effect of f_{max} and f_{min} on N can be expressed as [12]:

$$\frac{f_{max}}{f_c} = 1 - 0.0685 \left(1 - \frac{f_{min}}{f_{max}}\right) \log_{10} N \qquad (1)$$

where f_c is the corresponding static strength.

Committee 215's recommendation is similar when Eq. (1) is expressed as a Modified Goodman diagram. For compressive loading, the recommendation is described by Eq. (A1). When f_{min} is zero, both Eqs. (1) and (A1) predict for 10^7 cycles f_{max} equal to 50 percent of static strength. When f_{min} and f_{max} are equal, the stress range becomes zero and $f_{max} = f_c$, which equals the long-term sustained strength taken as 0.75 f'_c for Eq. (A1). Provided there is no stress reversal, Eq. (1) applies equally well for compressive, tensile or flexural loading when f_c is the static strength in direct compression, direct tension, or flexural bending. Recent research has indicated that fatigue strength for tension-compression is less than that for tension-tension [13-14].

Eqs. (1) and (A1) were derived from specimens tested in normal laboratory environments and subjected to constant amplitude loading, applied at frequencies of about 5 to 10 cycles per second. Loading and environmental conditions are substantially different for concrete offshore structures, for Arctic structures and some transportation structures. Load variations are often random and specimens submerged in sea water. Many papers presented at the recent ACI symposiums dealt with the response for the conditions.

The hypothesis commonly used for determining the degree of damage due to randomly varying stresses is the Palmgren-Miner hypothesis:

$$\sum_{i-1}^{k} \frac{N_i}{N_{fi}} = 1 \qquad (2)$$

where N_i = number of constant amplitude cycles at stress level i, N_{fi} = number of cycles to failure at that stress level i, and k = number of stress levels.

Siems [15] found that hypothesis accurate and deviations to be due to inherent variations in compressive strength rather than falsity of the hypothesis. However, Holeman [16] found that differences between values predicted from Eq. (2) and those observed from experiments cannot be explained solely by the stochastic nature of compressive strength. In particular, the number of cycles to failure was dependent upon the loading sequence. For example, a decrease in amplitude reduced the fatigue life compared to a reversed order of load application.

Eq. (2) implies that damage caused by load repetition increases linearly with number of cycles. By contrast, the damage rate as indicated by strains, microcracking or pulse velocity is initially very high, then becomes constant (secondary stage of failure) before again increasing sharply near failure. Thus, in principal, Eq. (2) cannot be accurate. However if the second stage of failure occupies most of the fatigue life and the initial and final stages only a small part, then Eq. (2) can be an acceptable design simplification.

The fatigue strength of concrete submerged in ocean water differs from that for normal laboratory environments for at least three reasons: (1) The concrete is subjected to multiaxial stresses; (2) Water trapped in opening and closing cracks causes hydraulic fracturing; and (3) Water induced stress-corrosion. Waggard [18] reported a reduction in fatigue strength for specimens under hydrostatic pressure. By contrast, for concrete tested in air confining pressures can be beneficial to fatigue life. Submerged concrete at atmospheric pressure has a shorter fatigue life than air dried concrete and the smaller the frequency of cycling, the shorter the fatigue life [19]. This result is probably due to the solution stress-corrosion effect of pore water propagation on crack [8]. Microcracks in concrete propagate in the presence of water; the higher the stress, the more saturated the concrete, the higher the temperature or the longer the time, the more severe is crack propagation. Thus, for offshore structures, high amplitude, low frequency load cycles can be more critical than high frequency, small amplitude cycles.

2.2 Reinforcing Bars

For bars in beams tested in air, fracture is caused by a crack that initiates at a stress concentration point on the bar surface. The largest stress concentration is usually at the intersection of transverse lugs and longitudinal ribs. Cracks initiating at such points must propagate through the depth of the bar sufficiently to cause fracture. Thus, the fatigue life equals the life for crack initiation plus the life during the crack growth [19]. The fatigue strength of a reinforcing bar is only about one-half that of a coupon machined from the center of the same bar. The fatigue strength of the central coupon increases with bar grade. The strength of the deformed bar does not. The non-dependence on bar grade is caused by decarburization of the bar surface. Typically, the carbon content doubles in the first 3/100th of an inch from the bar surface. Except for stress range, most variables which designers can readily control such as bar size, type of beam, minimum stress, bar orientation, and grade of bar have little effect on fatigue strength. Thus, the threshold value specified in Eq. (A2) depends only on stress range. However variables related to manufacture, fabrication and exposure such as deformation geometry, bends, tack welding, surface treatment and environment have significant effects.

The lowest stress range for failure reported in the recent NCHRP Program [2] was 21.3 ksi at a minimum stress of 17.5 ksi tension for a No. 11, grade 60 bar. Based on statistical analyses of the data, it was recommended that for straight hot-rolled bars with no welds and no stress raisers more severe than deformations meeting ASTM A615, the stress range f_{rr} in ksi should not exceed:

$$f_{rr} = 21 - 0.33\, f_{min} + 8\, r/h \qquad (3)$$

where f_{min} is the minimum stress level, tension positive in ksi, and r/h is the base radius to height ratio of the transverse deformation. Where the r/h value is not known, 0.3 is recommended. Then for zero minimum stress f_{rr} equals 23.4 ksi. Equation (3) is the expression recommended for design in References [4] and [5]. The r/h term is included in Eq. (3) to encourage production of bars with improved fatigue resistance. The NCHRP program included tests on 353 deformed bars used as the main reinforcing element in concrete beams. The results are therefore directly applicable to design. Bars were from five U.S. manufacturers, of five sizes and three grades. The effective depth of the test beam was varied and minimum stress levels of 6 ksi compression, 6 ksi tension, and 18 ksi tension were used.

The effects of cyclic stressing on reinforcing bars are sufficiently well known that Eq. (3) is undoubtedly adequate for ordinary structures under ordinary circumstances. However, there is only sketchy information for galvanized and epoxy-coated bars or other alternatives likely where environmental extremes prevail. The importance of environmental effects has been shown by tests [20] on 41.3 ksi yield bars used as the main reinforcement in concrete beams tested in air, in sea water, and in a 3% NaCl solution. In air, those bars exhibited an endurance limit corresponding to a stress range of 32 ksi for $2x10^6$ cycles and greater. The stress range for failure predicted by Eq. (3) is 31.5 ksi for those bars. In sea water and in NaCl solution the fatigue strength decreased markedly. There was no endurance limit even at 10^7 cycles and stress ranges for failure dropped to 19.6 ksi and 16 ksi for sea water and NaCl solution, respectively. Fractographic examination of failure surfaces showed clearly the change in the fracture mechanism with environment. In air, fatigue cracks initiated at the intersection of transverse lug and longitudinal rib. In sea water and NaCl cracks often initiated at corrosion pits and sea water or NaCl increased the rate at which those cracks grew. Thus, the reduction in strength for sea water and NaCl was due to reductions in life for both crack initiation and propagation. Since cracks often initiated at corrosion pits, reductions in life for crack initiation in corrosive environments are likely to be time as well as frequency dependent.

Stress ranges predicted by Eqs. (A2) and (3) are appropriate for straight bars only. Fabrication procedures such as bending, tack welding, or mechanical splicing reduce drastically stress ranges for failure [1, 3]. Recently, Bennett [21] reported tests on beams with main reinforcement in the maximum moment region spliced by lapping, by lapping and cranking, by cold-forged swages, and by screw couplers. A beam with straight bars withstood $3x10^6$ cycles at a stress range of 18.9 ksi without failure, whereas a beam with lapped and cranked bars failed at the crank after only 10^5 cycles of loading at the same stress range. If the decrease in stress range for a given fatigue life is consistent with data for straight bars [2], the endurance limit for those cranked bars would be 9 ksi. For the bars with swaged splices, fatigue fractures occurred where bars entered sleeves and the stress range for failure at $2x10^6$ cycles was 21.7 ksi. A specimen subjected to a stress range of 18.9 ksi had still not failed after $4x10^6$ cycles. For bars spliced with screwed couplers, failures occurred in the coupler at a high stress range and where the bar entered the coupler for a lower stress range. In the former case, the stress range for failure was 18.9 ksi at $0.75x10^6$ cycles, while in the latter case the value was 14.5 ksi at $1.5x10^6$ cycles. Since both mechanical splices performed well in terms of strength, deflection, and crack width in static loading tests, splices must be carefully located in structures subject to repeated load and provision 3 of Appendix A applied where appropriate to splices.

In many countries outside North America, higher yield bars are made by cold twisting grade 40 bars. The endurance limit for such bars is considerably less than for similar untwisted bars [20]. However, bar geometry in those tests was altered by twisting so that the lug base radius for the twisted bar was significantly less than for the untwisted bar. The r/h values for the untwisted and twisted bars were 1.4 and 0.55, respectively. The corresponding f_{rr} values predicted by Eq. (3) are 31.5 and 24.7 ksi respectively. The measured result of 26 ksi for twisted bars was therefore consistent with the change in bar geometry. For twisted bars, the effects of sea water immersion were non-existent until $0.6x10^6$ cycles or greater. Then the fatigue strength for immersed bars, as compared to bars tested in air, decreased with increased cycling. The endurance limit of immersed bars was 19.6 ksi for $5x10^6$ cycles and greater, and equaled the limit for 10^7 cycles for hot-rolled bars immersed in sea water.

2.3 Prestressing Steel

Three basic types of prestressing tendons are used in the U.S.A.: wire, seven-wire strand, and bars. Wires and strands are made by drawing steels with carbon contents about double those for reinforcing bars. Bars are made from hot-rolled alloy steels. Only plain wires are used in the U.S.A. and their smooth surface results in stress ranges for failure comparable to those for hot-rolled deformed bars in spite of an increased carbon content. Prestressing steels do not seem to have an endurance limit and the values predicted by Eq. (A3) correspond to the likely fatigue life for $2x10^6$ cycles [22]. Eq. (A3) is intended primarily for pretensioned construction. In post-tensioned construction bending at the anchorage and anchorage details can cause stress concentrations that reduce the fatigue strength below that given by Eq. (A3). Unless there are data to the contrary, the fatigue strength of anchorages should not be taken as greater than half the fatigue strength of the steel.

3. FATIGUE CHARACTERISTICS OF STRUCTURAL SYSTEMS

In a structural system, fatigue distress may develop due to excessive flexural, shear or bond stresses, increases in crack widths and deflections, or decreases in stiffness. Any high stress range location may be critical. However, since concrete is a relatively notch-insensitive material, stress concentrations due to holes or changes in section need not be considered provided stress values are based on the net rather than the gross section.

3.1 Flexural Strength and Serviceability

The flexural fatigue strength can theoretically be controlled either by the concrete or steel properties. In practice, the latter always governs. Concrete stress ranges in reinforced concrete beams proportioned by ultimate strength methods are below the limits of Eq. (A1) if maximum steel stresses are limited to 23.4 ksi [23]. Further, for more than 200 partially prestressed or hollow core slabs, there were only three cases where the concrete stress exceeded 80% of the value of Eq. (A1) before the steel stress became critical [22]. A real structure is a composite of many members, each generally containing more than two tensile reinforcing elements. Fatigue fracture of one or more of those elements does not cause immediate failure of the structure [3]. Rather, deflections and crack widths increase and hence when those quantities exceed reasonable values, there is warning of the need to repair and strengthen the structure. Although codes require designers to consider deflection increases caused by long-term loadings, they generally ignore deflection and crack width increases caused by cyclic loading.

Increases in deflection and crack width of reinforced concrete beams subject to fatigue loading are caused by cyclic creep of the compressed concrete and a reduced stiffness of the tension-zone concrete due to fatigue cracking and deterioration of the bond between steel and concrete. Good agreement with test data for increases in deflection and crack width was obtained [24] when deflections were computed according to ACI Code 318-77 using an effective modulus concept to account for cyclic creep of concrete and an effective gross and cracked moment of inertia to account for reduced tensile stiffening of the concrete with cyclic loading. Reasonable agreement with crack width data was obtained when widths were calculated using a classical slip-theory approach that included the bond deterioration caused by fatigue loading.

Several empirical relationships have also been proposed to predict deflections and crack widths for reinforced concrete beams subjected to fatigue loading [25-27]. Deflections and crack widths can be predicted [25] by the expression:

$$\gamma = Ae^{Br} \tag{4}$$

where r = ratio between given number of cycles and number of cycles to failure; γ = value of deflection or maximum crack width under fatigue loadings; A = initial value of deflection or crack width at maximum load, (r = 0); e^B = deflection or crack width at end of fatigue life at maximum load (r = 1) relative to initial value; and B = 1.55 for deflection and 1.67 for maximum crack width. Alternatively, values can be predicted [27] from the expressions:

$$\Delta_n = 0.225\Delta_0 \log n$$

and (5)

$$w_n = w_0 \ (0.382 - 0.227 \log n) \log n$$

where Δ_0, w_0 are initial deflection and crack width at maximum load and Δ_n, w_n are corresponding deflection and crack width at maximum load for nth loading cycle.

For most prestressed concrete structures, fatigue considerations are not important unless the concrete cracks. However, once such cracking occurs due to over-load, accident, construction procedures or thermal strains, fatigue considerations become important, and of some concern, due to recent test results for full-size cracked pretensioned bridge girders [28]. In some of those tests the prestressing strands fractured after $3x10^6$ cycles that cause a calculated stress range in the strands between 142 and 151 ksi only. That range was 40% of the range predicted by Eq. (A3). In a cracked prestressed concrete beam the stress range in the steel increases with cycling due to accumulation of residual strains in the concrete on the compression side of the beam and an increase in crack widths on the tension side. In the test beams, the measured stress range exceeded 20 ksi at failure. Probably the reduced strength was partially due to the use of pitted strands and crack formers. Nevertheless, until additional data are available, it is recommended that steel stress ranges in cracked prestressed beams, evaluated using gross section properties, be limited according to Eq. (A4).

Performance of reinforced concrete in flexure in marine environments is another area where additional data are highly desirable. Both high and low cycle response are important since failure is undesirable for either long-term environmental loadings likely during the service life or a limited number of overloads greater than the design load. The greatest threat is from low-cycle high amplitude repeated loading, an accident, or thermal condition, that creates cracking left unrepaired and followed by numerous lesser amplitude cycles. Whether such cracking makes corrosion of the reinforcement possible and a reduction in fatigue life likely is also a matter of debate. Tests on rectangular beams loaded at slow frequencies in simulated marine environments and in air are reported in Reference [29]. Most tests were uni-directional, but some involved reversed bending. The uni-directional bending specimens tested in marine environments experienced progressive blocking of cracks on their tension side due to accumulation of salts. That blocking reduced the stress range in the bar, increased the mean stress, and increased the fatigue life of the beam compared to that for a similar specimen tested in air. Beams tested at higher frequencies did not experience crack blocking and, as expected from results reported in Reference [19], had fatigue lives less than those for specimens tested in air. In reversed bending tests on doubly reinforced beams, crack blocking occurred, but that blocking prestressed the beams locally at the flexural cracks. Fatigue lives were less than for specimens tested in air. Blocking for uni-directional loading will be sensitive to the chemical composition of the concrete and pozzolans may sharply reduce the potential for crack blocking effects.

3.2 Bond Strength

The bond fatigue strength is strongly dependent on the geometry of a member and its loading. If diagonal tension cracks do not occur in the anchorage zone for the reinforcement, then the bond strength for 10^6 cycles is about 60 percent of the static strength and bond fatigue is unlikely to control the fatigue response. If diagonal tension cracks occur, then the bond strength can drop to 40 percent of the static strength. Then shear fatigue rather than bond fatigue controls the fatigue response [30].

3.3 Shear Strength

Committee 357 has recommended that "where maximum shear exceeds the allowable shear on the concrete alone, and where the cyclic range is more than half the maximum allowable shear in the concrete alone, then all shear should be taken by the stirrups." That recommendation is based on the findings of Reference [30]. Inclined cracking is a prerequisite for a shear fatigue failure. Such cracks can form under multiple repetitive loads at stresses 50% of those for static loading. After inclined cracking stirrups strains increase rapidly until nearly all the shear is carried by the stirrups. If the reinforcement is bent in the cracked zone, its cyclic stress range should be limited according to provision 3 of Appendix A.

Recent Japanese research confirms the wisdom of those recommendations [31, 32]. A systematic study was made of changes in stirrup strain with crack development and cycling. Considerable redistribution of stresses among stirrups with cycling was observed. The average strain in the stirrups intersected by inclined cracks increased at almost a constant rate with the log of the number of loading cycles. The effective contribution of the concrete to the shear strength decreased proportionately. Expressions were developed for predicting those changes. Fatigue fractures of stirrups occurred at the bends at stress ranges consistent with those reported in Reference [30]. Stirrup failures occurred in beams with maximum applied shears as little as 44% of the static capacity. Thus, beams that fail in flexure under static loading due to stirrup yield fail in shear under repeated loadings due to stirrup fracture.

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APPENDIX A

Provisions for Fatigue of Concrete Suggested by ACI Committee 215

Fatigue shall be considered by rational evaluation when the stress range in concrete members under a large number of repeated service loads exceeds the following:

1. Concrete in compression under maximum loading:

$$f_{cr} = 0.5\, f'_c - \frac{2}{3} f_{min} \qquad \text{(A1)}$$

2. Deformed reinforcement in tension or a combination of tension and compression:

$$f_{rr} = 20 \text{ ksi} \qquad \text{(A2)}$$

3. The value of f_{rr} shall be reduced by one-half in the region of bends or of locations where auxiliary reinforcement has been tack welded to main reinforcement.

4. Prestressing tendons in tension

4.1 Where the nominal tensile stress in the precompressed tensile zone does not exceed $6\sqrt{f'_c}$ and the member is uncracked:

$$f_{tr} = 0.10\, f_{pu} \qquad \text{(A3)}$$

4.2 Where the nominal tensile stress in the precompressed tensile zone exceeds $6\sqrt{f'_c}$ or the member is cracked:

$$f_{tr} = 0.04\, f_{pu} \qquad \text{(A4)}$$

Notation: f_{cr} = stress range in concrete under repeated service loadings; i.e., difference between maximum and minimum compressive stress in psi.

f_{rr} = stress range in deformed reinforcement under repeated service loadings; i.e., difference between maximum and minimum stress in psi.

f_{tr} = stress range in prestressing tendons under repeated service loadings; i.e., difference between maximum and minimum stress in psi.

f_{min} = minimum compressive stress in psi (compressive stress positive).

New Concepts for Concrete Fatigue Design Procedures in Japan

Nouveaux concepts pour le dimensionnement à la fatigue des structures en béton, au Japon

Neue Konzepte der Ermüdungsbemessung von Stahlbeton in Japan

YOSHIO KAKUTA
Assoc. Professor
Hokkaido University
Sapporo, Japan

HAJIME OKAMURA
Assoc. Professor
University of Tokyo
Tokyo, Japan

MICHIYUKI KOHNO
Executive Director
Nihon Kotsu Gijutsu KK
Tokyo, Japan

SUMMARY
The "Tentative Recommendations for the Limit State Design of Concrete Structures" were published by the Concrete Committee of the Japan Society of Civil Engineers in April 1981. The present report describes the fatigue design procedures introduced in the Recommendations. Fatigue strength of concrete in compression, reinforcing bars in tension and reinforced concrete in shear are dealt with.

RESUME
Les Recommandations provisoires pour le calcul aux états-limites des structures en béton furent publiées par la commission "béton" de la Société japonaise des ingénieurs civils en avril 1981. Le présent rapport décrit les méthodes de calcul à l'état-limite de fatigue que la commission a recommandées. On traite de la résistance à la fatigue du béton en compression, des barres d'armature sous tension et du béton armé sous un effort tranchant.

ZUSAMMENFASSUNG
Die vorläufigen Empfehlungen zur Bemessung von Betonbauwerken auf Grenzzustände wurden vom Betonkomitee der Japanischen Gesellschaft der Bauingenieure im April 1981 veröffentlicht. Der vorliegende Beitrag stellt das Bemessungsverfahren auf Ermüdung gemäss den Empfehlungen vor. Dabei werden die Ermüdungsfestigkeiten von Beton unter Druckbeanspruchung, Betonstahl unter Zugbeanspruchung, und Stahlbeton unter Schubbeanspruchung behandelt.

1. INTRODUCTION

In the 1967 Standards, the Japan Society of Civil Engineers (JSCE) introduced the fatigue provisions for design of reinforced concrete structures. Through the experiences of applying the provisions to the Shinkansen railroad structures, fatigue design procedures had been developed, and new procedures were introduced in the Tentative Recommendations for the Limit States Design of Concrete Structures in 1981 [1]. This report describes the fatigue design procedures introduced in the Recommendations and their background.

2. CONCEPTS OF FATIGUE DESIGN PROCEDURES

The concepts adopted in the Recommendations for the fatigue design procedures are summarized as follows:

(1) Fatigue limit states should be considered in the design of structures when the effects of variable loads are dominant.

(2) Design fatigue strength, which is obtained by dividing the characteristic strength by the appropriate partial safety factor for strength, should be larger than applied design variable stress.

(3) Characteristic fatigue strength shall be calculated from permanent stress and equivalent cycles, N, of the applied design variable stress. The equivalent cycles may be evaluated on the assumption of a linear cumulative damage theory, such as Miner's.

(4) Characteristic fatigue strength of concrete, f'rck, may be calculated as follows:

$$f'_{rck} = (\ 0.9k\ f'_{ck} - \sigma'_{cp}\ )\ (\ 1 - \log N/15\ ) \qquad (1)$$

where f'ck denotes the characteristic static strength of concrete, σ'_{cp} permanent compressive stress, and k(=0.85) coefficient considering the difference of the concrete strengths between cylinder specimens and the structural members. Principally, fatigue strength of reinforcing bars shall be determined on the basis of test results. The following values may be used for deformed bars with a diameter not larger than 32 mm:

$$\begin{aligned} f_{rsk} &= (160\ \mathrm{MPa} - \sigma_{sp}/3)\ 10^{-0.2(\log N-6)} && \text{for } \log N < 6 \\ &= (160\ \mathrm{MPa} - \sigma_{sp}/3)\ 10^{-0.1(\log N-6)} && \text{for } \log N > 6 \end{aligned} \qquad (2)$$

where σ_{sp} is permanent tensile stress. However, fatigue strength of bars with bends or welding connections must not be taken greater than half of the values calculated from Eq.(2), unless determined by test results.

(5) Applied design stresses due to flexure may be calculated by the elastic theory of cracked section. However, the fatigue limit state for concrete in compression may be examined only for the stress at the location of compressive resultant. Applied design variable stress and permanent stress in shear reinforcement can be calculated as follows:

$$\begin{aligned} \sigma_{wrk} &= \{1.15(V_{md} - 0.5\ V_{cd})s\}/\{A_w\ d(\sin\alpha + \cos\alpha)\}(V_{rd}/V_{md}) \qquad (3) \\ \sigma_{wrp} &= \sigma_{wrk}\ (V_{pd}/V_{rd}) \end{aligned}$$

where Vmd is the design maximum shear force, Vcd design ultimate shear force resisted by concrete, Vpd applied design permanent shear force, Vrd applied design variable shear force, Aw area of shear reinforcement within a distance s, d effective depth and α angle between shear reinforcement and longitudinal axis of the member.

(6) Fatigue limit states for reinforced concrete beams may generally be examined only for longitudinal tensile reinforcement and shear reinforcement. Fatigue limit states for reinforced concrete slabs may generally be examined only for tensile reinforcement. The examination of fatigue limit state for reinforced concrete columns may generally be omitted.

3. BACKGROUND OF THE CONCEPTS

3.1 Methods for Checking of Safety

Limit states are generally placed in two categories, the ultimate and serviceability limit states. Fatigue limit states are considered as a kind of ultimate limit states since fatigue may result in a collapse of a part or whole structure.

The characteristic fatigue strength for a certain number of stress cycles corresponding to the specified life of a structure can be determined on the basis of the same concepts as the characteristic static strength which substantially corresponds to the strength for one stress cycle. The similar concepts may be applied to the partial safety factor for the strength of materials. Namely, the values of γm for fatigue limit states can be taken to be same as for the ultimate limit states. On the other hand the loadings associated with the fatigue limit states are essentially of the same nature as those with serviceability limit states. Therefore the values of γf for the fatigue limit states may be taken as much as for serviceability limit states, that is, in general $\gamma f = 1.0$.

Criterion for fatigue failure of a material is generally expressed by a function of the permanent or the minimum cyclic stress, the range of stress or the maximum cyclic stress, and the number of stress cycles. The Recommendations define fatigue strength as the range of stress calculated from a fatigue criterion function for given permanent stress and a given number of cycles. Then the checking of safety against fatigue failure can be achieved to ensure that the design fatigue strength, frd, is larger than applied design variable stress, σrd.

$$frd > \sigma rd \qquad (4)$$

where $frd > frk/\gamma m \quad , \quad \sigma rd = \sigma rk \qquad (5)$

frk denotes the characteristic fatigue strength and σrk variable stress due to the characteristic loadings.

3.2 Characteristic Fatigue Strengths

The characteristic fatigue strengths adopted in the Recommendations were determined on the basis of the investigations domestic as well as overseas'. The limitation of space, however, confined to enumerate only Japanese investigations, but a little overseas' ones.

(1) Fatigue Strength of Concrete

The statistical analysis of test data led to the following equation for probable fatigue life of concrete,

$$\log N = 17\ [1 - Sr/(1-Sp)] \qquad (6)$$

in which Sr represents the range of stress level, defined as the ratio of variable stress to ultimate static strength, and Sp permanent stress level. Fig.1 shows a part of test data used in the analysis with the line obtained from Eq.(6). Fatigue strength calculated from Eq.(6) has an approximately 50 percent failure probability, and Eq.(1) was derived to provide characteristic fatigue strength of concrete with about 5 percent failure probability.

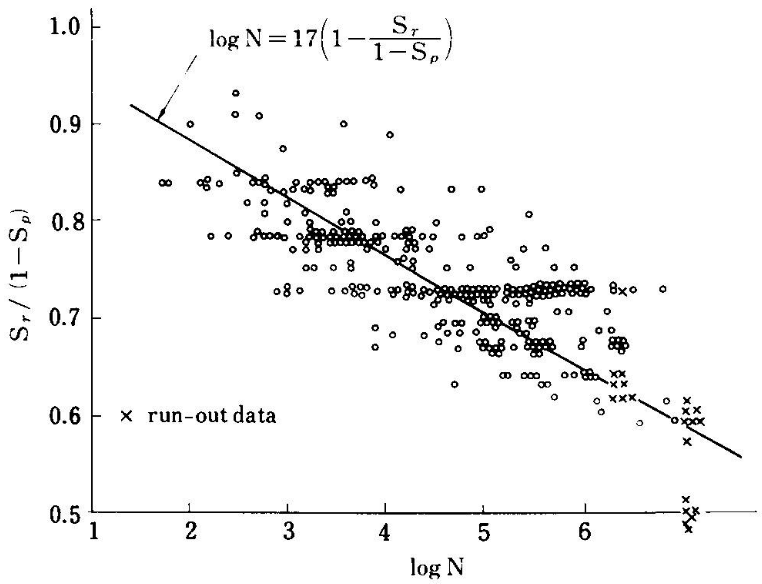
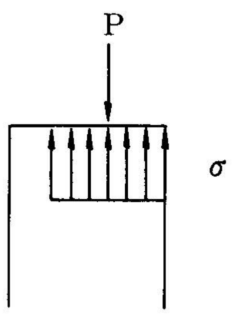


Fig.1 Fatigue strength of concrete in compression [4]-[8]



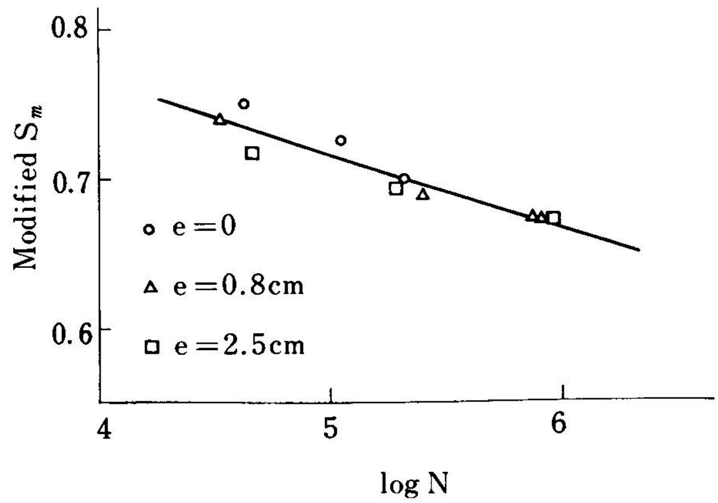


Fig.2 Stress calculation by rectangular stress distribution

Fig.3 Ople and Hulsbos' data with modified ordinate by [11]

It has been found that stress gradient has a significant effect on the fatigue strength of concrete in compression [8]-[10], in which the stress level is defined as the ratio of the extreme fiber stress to static strength in axial compression. On the other hand Matsushita and Makizumi [11] have pointed out that it is rather rational to define the stress level as the ratio of applied load to the ultimate load and proposed a method for calculating the stress of concrete by using a rectangular stress distribution, as seen in Fig.2, so that the effect of stress gradient diminishes as shown in Fig.3. The article in 2.(5) is based on their proposal.

The examination by numerical examples of the design of reinforced concrete members has shown that the checking of safety against fatigue failure of concrete can generally be omitted for the structures designed with $\gamma mc = 1.5$ and $\gamma f = 1.5$ for the ultimate limit states. However, it must be noted that the logarithm of fatigue life of lightweight aggregate concrete is about 70 percent of that of normal concrete, as seen in Fig.4 [5][9][10], and that logarithm of fatigue life of concrete in the water is only about two thirds or less of that in the air [15][16].

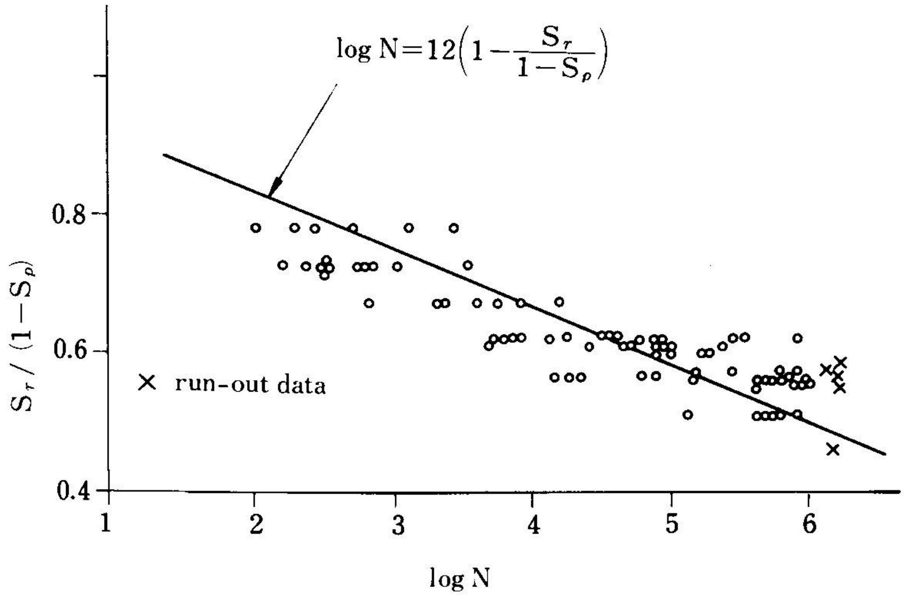


Fig.4 Fatigue strength of lightweight aggregate concrete in compression [5][9]

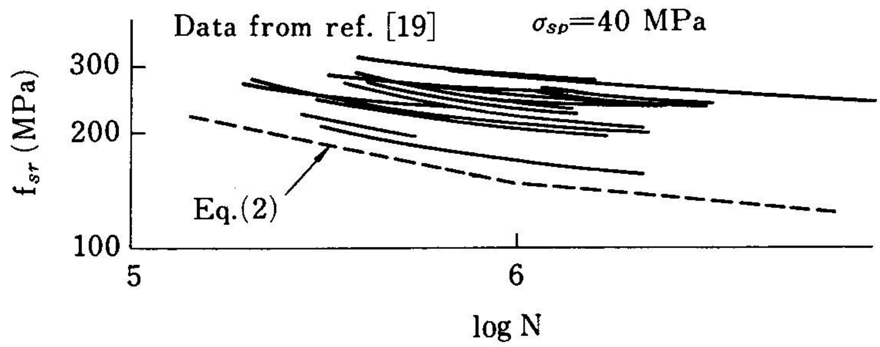
It has been found that the fatigue strengths of concrete in tension or flexure are about the same as that in compression when expressed by a percentage of the corresponding static strength [3][6]. Therefore, if necessary, Eq.(1) may be applied to determine characteristic fatigue strengths of concrete in tension or flexure.

Tests on concrete with compressive strength of 80 to 100 MPa [13][14] have shown that the fatigue strength of high strength concrete expressed by a percentage of its static strength is not less than that of normal concrete.

(2) Fatigue Strength of Reinforcing Bars

It is known that the fatigue strength of reinforcing bars depends on many factors, such as geometry of surface deformations, bar diameter, yield and tensile strength, bending, and welding [3][17][18]. However, the test results regarding the effect of these factors reported quantitatively differ considerably each other. Accordingly, it must be difficult to establish fatigue strength equation which can be applied widely. The Recommendations, therefore, state that principally, the characteristic fatigue strength of reinforcing bars, frsk, shall be determined on the test results.

Many investigations have shown that frs - N curves exhibit transition of a steeper slope to a flatter one in the vicinity of one million cycles. This may imply that reinforcing bars have an endurance limit, however, the evidence is still insufficient because of the lack of data in the region of a sufficient number of cycles. As far as the tests conducted in Japan are concerned, the average slope of log frs - log N curves of the deformed bars is about -0.2 in the finite life region as seen in Fig.5, and Eq.(2) was derived to cover all the data on the safe side.



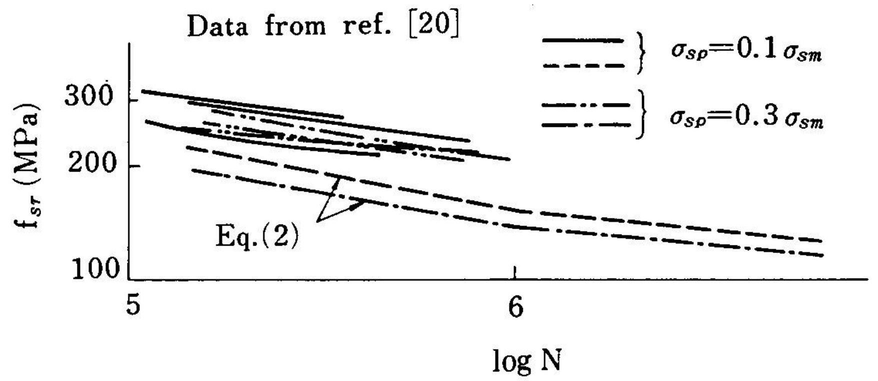


Fig.5 Fatigue strengths of reinforcing bars in tension

It is noted that most of the deformed bars made in Japan today have considerably higher fatigue strength than the value obtained from Eq.(2) owing to the efforts which have been made to improve the geometry of the deformed bars, particularly on the base radii of deformations [19]-[26]. It is, therefore, profitable to determine the characteristic fatigue strength of reinforcing bars by tests in most cases.

It has been found that the fatigue strength of bent bars, such as stirrups and bent-up bars, is considerably lower than that of straight bars [18][27]-[29].

The tests with bars bent around a pin radius conforming to the Japanese Code Requirements have shown that the fatigue strength of bent bars is 1/2 to 2/3 of that of the straight bars [28][29]. It has also been recognized that welding may considerably reduce the fatigue strength of reinforcing bars [30]-[32]. The tests [32] have shown that the fatigue strength of bars with stirrups attached by welding is about half of those attached by wire ties.

(3) Fatigue Strength of Reinforced Concrete in Shear

The investigations [33]-[36] have handled the fatigue shear strength of reinforced concrete beams without web reinforcement. Most of the data reported in these references have the fatigue lives longer than those predicted by Eq.(1), which was originally derived for the fatigue strength of concrete in compression. Furthermore, most of the data in the reference [37], in which the fatigue strength of reinforced concrete slabs in punching shear is dealt with, have longer lives predicted by Eq.(1). Therefore, if necessary, Eq.(1) may be used to determine the characteristic fatigue strength in shear of reinforced concrete beams or slabs without shear reinforcement, unless more rational procedures are available.

It is important to establish rational fatigue design procedures for shear reinforcement because the fatigue strength of bent bars is very low. The investigation [38] has proposed that the applied stress range of stirrups may be calculated by truss theory and has compared it with the fatigue strength of bent bars. A more rational method has been proposed by the recent investigations [39][40], in which the behavior of stirrups in strain is examined in detail. Eq.(3) is based on these investigations.

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Design Recommendations for Offshore Concrete Structures

Recommandations pour le dimensionnement des structures offshore en béton

Bemessungsempfehlungen für Offshore-Bauten aus Beton

KNUT WAAGAARD
Ph.D.
Det Norske Veritas
Oslo, Norway

SUMMARY
The paper presents a short summary of recommendations for fatigue strength evaluations of offshore concrete structures. Offshore concrete structures are exposed to an environment which is different from the environmental exposure of land based structures. These special features are discussed in relation to the design recommendations. As the environmental loads are random in nature, the paper will discuss how a design recommendation can incorporate random loading.

RESUME
Cette publication présente un condensé de recommandations en vue d'évaluer la résistance à la fatigue des structures offshore en béton. Du fait de leur environment, les structures offshore sont soumises à des charges qui les différencient fondamentalement des structures onshore. Ces particularités sont détaillées en relation avec les recommandations de dimensionnement. Compte tenu que les charges dues à l'environnement sont aléatoires par nature, on s'attache à étudier comment des recommandations de dimensionnement peuvent tenir compte du caractère aléatoire du chargement.

ZUSAMMENFASSUNG
Der Beitrag präsentiert eine kurze Zusammenfassung der empfohlenen Bestimmung der Ermüdungsfestigkeit für Offshore-Betonkonstruktionen. Offshore-Bauten sind anderen Umgebungsbedingungen ausgesetzt als entsprechende Bauten an Land. Diese Besonderheiten werden im Zusammenhang mit den Bemessungsempfehlungen erörtert. Da die äusseren Lasten und Kräfte Zufallscharakter aufweisen, wird dessen Berücksichtigung in den Empfehlungen erläutert.

1. INTRODUCTION

Offshore concrete structures have been constructed and are in operation in several parts of the world. Following the first offshore concrete oil platform, which was installed nearly eight years ago, several offshore concrete structures have been constructed in several countries.

The design of the first offshore concrete structure with regard to fatigue strength was based upon an evaluation of the available knowledge regarding fatigue strength of concrete in the marine environment. The available knowledge was to some extent represented by design practice in different national codes and the current research presented by the work of ACI Committee 215 [2].

At that particular time, it was widely accepted among experts in this field that fatigue failure of reinforced/prestressed concrete structures most likely would occur in the reinforcement/tendons. The owners, designers and certifying authorities did, however, accept the risk that reversible of stresses could cause pumping [3] in crack, (see Figure 1) thus reducing the fatigue life of an offshore structure relative to a conventional land structure. The design philosophy was to limit the likelyhood of the occurrence of cracks in offshore structures exposed to fatigue loading.

The environmental loads on an offshore structure are completely random in nature with respect to frequency, magnitude and order of loading. In order to handle the above nature of the load in an analytical investigation, the general accepted method has been to devide the stress histogram into stress blocks (see Figure 2) and applying the Miner's hypothesis in its origianl form or a variant of this method.

Based upon above early evaluations of the fatugue design of offshore concrete structues, some research activities were started in different countries.

At Veritas [3] work was started on defining the fatigue strength of submerged concrete members, the area of investigation was the influence of reversible cyclic loading on the compressive fatigue strength of submerged concrete. Submerged concrete members exposed to reversible cyclic loading will crack and pumping of water in and out of the crack may occur. The conclusion [3] in the early work was that a reduction in the fatigue strength was observed when the concrete specimens were tested under hydrostatic pressure and reversible cyclic loads. The work described in [3] has been continued as a sponsored project.

At TNO in Holland, some interest was also generated on the compressive fatigue strength of concrete [4] and [6].

The main parameters in the first study [4] were:

- storage time in water
- saturated vs unsaturated concrete
- effect of frequency on saturated concrete

The conclusions from the TNO tests [4] are shortly summarized as follows.

- submerged concrete has a shorter fatigue life than air dried concrete
- the longer the storage time in water, the shorter is the fatigue lie (effect of saturation)
- the frequency affect the fatigue life, the shorter the frequency the shorter is the fatigue life

In its second study [6], TNO studied the influence of cumulative damage of concrete in compression with the use of Miner's rule. Van Leeuwen and Siemes [6] conclude that the Miner number proved to have a logarithmic - normal distribution with a mean value which is less than one. This conclusion was similar to the results obtained in Sweden by Tepfers et al [5] and by Holmen [7] in Norway.

Offshore concrete structures are located in a very corrosive environment and it is natural that corrosion fatigue of the reinforcement has been important research topics in the later years. The studies have been carried out in Britain [9], [10], [11] and in France [12]. The general conclusion from these works are:

- The fatigue life of rebars is only slightly lowered due to corrosion fatigue.
- Cracks which are constantly kept open will heal when submerged in sea-water even under dynamic loading.

The experimental data has been obtained for the test condition which is considered most severe for corrosion fatigue, namely tension-tension cycling. In submerged parts of offshore concrete structures which is exposed to cyclic loading, it is unlikely that above stress condition will be allowed. These structures will remain predominately in compression and effect of tension-compression and mean stress levels, which are compressive, will have significant interest as test parameter. Normally tensile stresses and cracks will not be accepted for offshore concrete structures for members exposed to fatigue loading.

Ben Gerwick [1] has additionally reviewed the design of offshore concrete structures with respect to fatigue strength. His paper will make useful reading in respect to general design criteria.

SUMMARY OF VERITAS REQUIREMENTS

The general requirements for the design against fatigue failure of offshore concrete structures are described in Chapter 7.7 of the Veritas Rules for the Design, Construction and Inspection of Offshore Structures [13]. The requirements are here expressed in general terms. More detailed recommendations on how to satisfy these general requirements are given in Appendix D8 to above rules. It should be noted that above recommendations are non-mandatory. The engineer is free to use other methods and procedure than those recommended, provided an equivalent standard of quality and safety is obtained.

The Veritas Rules require the characteristic S-Log N curve to be determined from the 5th percentile of the test results. The S values should additionally be divided by the appropriate material coefficient, $_{m}$. The material coefficient is to be agreed upon with the Society. The Rules require that cumulative damage to the structure caused by different fatigue loading is to be included in the analysis.

The structural aspect is considered to be of great importance for fatigue evaluations and the following points are stressed in the Rules.

- Geometric layout of structural elements and reinforcement should be such as to minimize the possibility of fatigue failure. Ductility should be assured by confinement of the concrete by appropriate reinforcement.

Submerged concrete members that are essential for the integrity of the structure and are subjected to loadings that may cause fatigue failure are to be designed without membrane tension for any load combinations. Edge stresses due to bending is to be limited so that no cracking occurs. Where creep effects may cause transfer of compressive stress from the concrete to the reinforcement such effects are to be accounted for in the determination of the concrete stresses.

RECOMMENDED PRACTICE

In the recommended practice in Appendix D8, it is accepted that offshore concrete structures are exposed to more dynamic and complex loading than most other types of structures This makes it difficult to extrapolate earlier experience on land based structures. With respect to the special influence of the marine environment, the recommendations have been based on the pilot study [3]. The criteria for the reinforcement have been based on the work by Helgason [8].

Reinforced Concrete Exposed to Axial and Flexural Dynamic Loads

For submerged concrete members exposed to axial and flexural load, the following combined Goodman and Wöhler curves are specified:

$$\log_{10} N = 10.0 \; \frac{1.0 - \dfrac{S_{max}}{\alpha \dfrac{f_k}{\gamma_m}}}{1.0 - \dfrac{S_{min}}{\alpha \dfrac{f_k}{\gamma_m}}} \qquad (1)$$

where

S_{max} maximum average outer fibre stress in stress block i, calculated on the basis of linear elastic theory assuming cracked section

S_{min} minimum average stress in the same outer fibre calculated on the basis of linear elastic theory assuming cracked section

f_k characteristic compressive strength measured on concrete cylinders

γ_m material factor = 1.25

α takes account of the flexural gradient across the section [14]

Since the evaluation of fatigue strength is monotonous, it is of importance to derive at a method of making a quick assessment on the need to carry out futher fatigue calculations. Veritas recommends such a method.

Should a detailed fatigue check be necessary, then the cumulative fatigue life may be investigated according to a modified Miner's hypothesis.

$$\sum_{i=1}^{m} \frac{n_i}{N_i} < 0.2 \qquad (2)$$

where

m = number of stress blocks (minimum 8)
n_i = number of stress cycles in stress block, i
N_i = number of cycles to fatigue failure for average stress in stress block, i.

The number of applied cycles, n_i, within stress block, i, is obtained from an investigation of the sea states, wind direction, and static and dynamic response of the structure expected within the design life, which is normally not to be taken less than twenty years.

The number of cycles to fatigue failure at constant amplitude, N_i, within stress block, i, may be obtained from test results or from equation 1.

For a fatigue analysis, it is necessary to obtain stress history diagrams as function of log n (see Figure 2). The stress history gives information on maximum and minimum stress in an element or member as a function of the logarithm to the number of applied cycles. The load histogram is divided into stress blocks, normally at least eight blocks.

For the reinforcement in the concrete, the number of cycles, N, causing fatigue failure of straight bars at a given stress range and minimum stress level, may be taken as:

$$\log_{10} N = 6.5 - 2.3 \frac{S_r}{\dfrac{f_{sy}}{\gamma_m}} - 0.002\, S_{min} \qquad (3)$$

where

S_{min} and S_r describe the minimum stress and the stress range.

The endurance limit, f_r, is taken as

$$f_r = \frac{165}{\gamma_m} - 0.33\, S_{min} \qquad (4)$$

If the stress range in the reinforcement at 10000 cycles is less than the endurance limit as defined above, no further checks on the fatigue strength is required.

If the stress range in the reinforcement exceeds the endurance limit, then detailed investigations are necessary. It will in this case be necessary to elongate the Wöhler curve as defined above beyond the endurance limit (see Figure 4).

For bent bars with a bend of diameter less that 25d and greater than 8d, and for welded reinforcement, the Wöhler curve defined above is modified to take account of the reduced fatigue life.

Veritas makes no recommendations for the incorporation of cumulative damage in the reinforcement. It is, however, reasonable to apply the Miner's hypothesis in its original form with a Miner sum equal to 1.0.

Reinforcement Concrete Members Exposed to Transverse Shear Loading

For reinforced and prestressed concrete members exposed to transverse shear loading of variable magnitude, the following design recommendations are made for designing against shear failure in the concrete.

The proposed Wöhler curve is similar to that used for concrete in compression and bending. In stead of formulating the Wöhler curve as a function of stress, the total shear capacity is used, which also includes the contribution from the longitudinal reinforcement and ties.

For concrete members with positive $V_{f_{max}}/V_{f_{min}}$ the V-N diagram is referred to as:

$$\log_{10}N = 10.0 \frac{1.0 - \frac{V_{f_{max}}}{V_r}}{1.0 - \frac{V_{f_{min}}}{V_r}} \tag{5}$$

where

$V_{f_{max}}$ maximum average shear force in stress block, i

$V_{f_{min}}$ minimum average shear force in stress block, i

V_r design shear resistance

The total shear resistance is not to be taken greater than

$$V_{rmax} = 0.25 \cdot f_{cr} \cdot b \cdot d \tag{6}$$

where

$$f_{cr} = \frac{f_{ck}}{\gamma_m}$$

For members which are exposed to completely reversable transverse shear stress i.e. $V_{f_{max}}/V_{f_{min}}$ is negativ, the capacity part of the concrete, V_{cr}, should be ignored in the calculation of V_r.

The cumulative damage of the concrete should be investigated using the Miner's cumulative method with a summation of 0.2.

The fatigue strength of the reinforcement requires to be checked independantly. The Wöhler curves and endurance limit as described earlier should be used in the investigation. In calculation of the reinforcement stresses, realistic models in estimating the reinforcing stress should be used.

Fatigue Strength in Bond

In lack of more detailed information, it is recommended to double the anchorage length normally required for static design, if the number of load repetitions exceeds 10000 cycles and the bond stress range (S_{br}) at 10000 cycles exceeds the bond strength (f_{br}) at 2.10^6 cycles.

Fatigue Strength of Tendons

The Veritas recommendations presumes that fatigue strength data will be provided by the manufacturer as the fatigue characteristics will be dependant upon the steel qualities and other details of productions and anchorage of tendons.

The fatigue data should contain data on the total assembly of the tendons including the anchorage.

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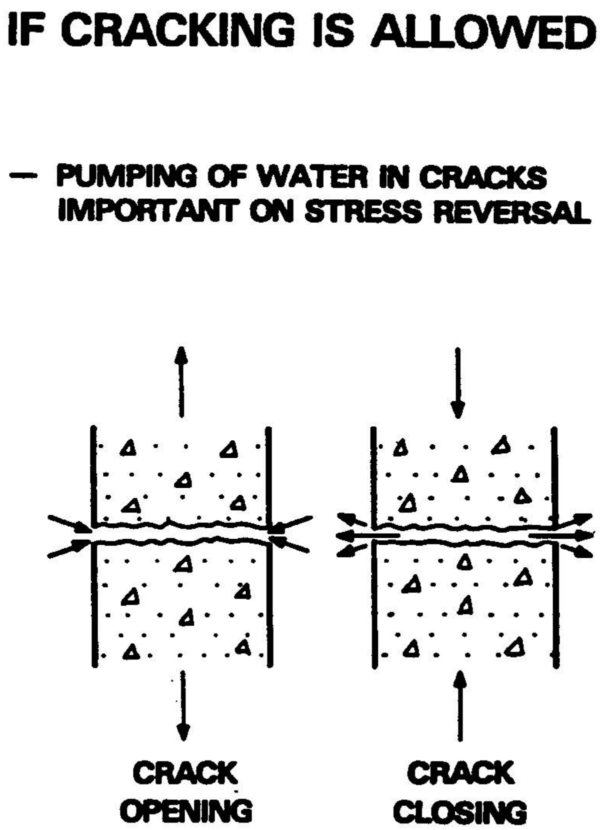

Fig. 1 Pumping of Water in a Crack on Stress Reversal

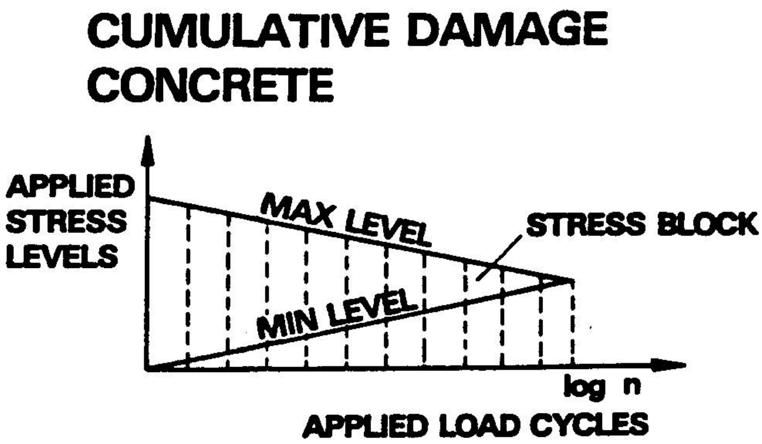

Fig. 2 Stress Histogram for use when Analysing Cumulative Damage of Concrete

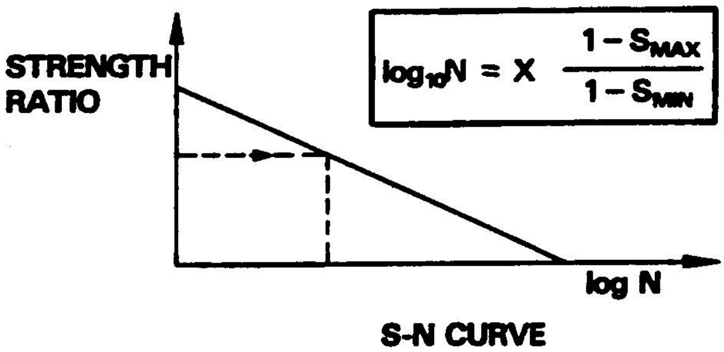

Fig. 3 Example of Wöhler Curve for Concrete

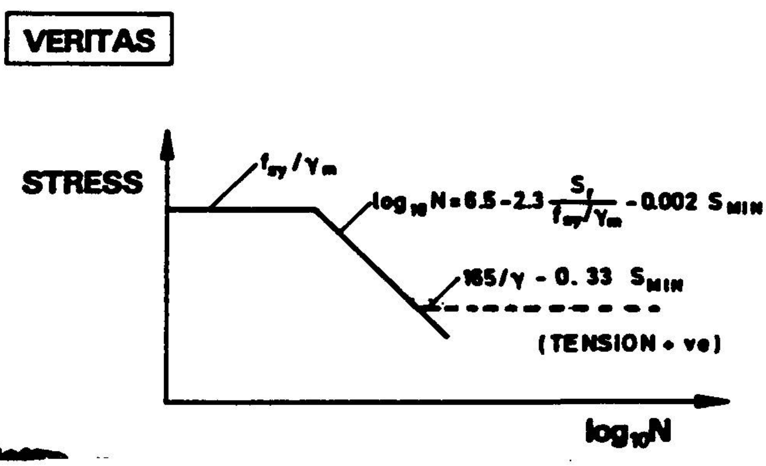

Fig. 4 Wöhler Curve for Reinforcement

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Fatigue as a Design Limit State for Bridges

Fatigue en tant qu'état-limite de dimensionnement pour les ponts

Ermüdung als Bemessungsgrenzwert für Brücken

PAUL GRUNDY
Associate Professor
Monash University
Melbourne, Australia

SUMMARY
Current methods of design for fatigue life are part deterministic for load effects and part probabilistic for resistance. An estimate is made of risk of fatigue failure, which is found to be much higher than for other ultimate limit states. This difference needs to be reconciled through reference to inspection and maintenance, and the prevention of collapse by redundant load paths.

RESUME
Les méthodes existantes de dimensionnement à la fatigue sont d'une part déterministes en ce qui concerne les charges et d'autre part probabilistes pour ce qui est de la résistance. On fait une estimation du risque de rupture par fatigue, lequel est trouvé beaucoup plus élevé que pour d'autres états-limites ultimes. Cette différence doit être prise en considération en ce qui concerne les inspections et la maintenance, ainsi que la prévention des ruptures sous le passage de charges répétées.

ZUSAMMENFASSUNG
Gegenwärtige Verfahren für den Dauerfestigkeitsnachweis sind deterministisch für Lasteffekte und basieren auf wahrscheinlichkeitstheoretischen Überlegungen für die Festigkeit. Das Risiko eines Ermüdungsbruches wird abgeschätzt. Es stellt sich heraus, dass es wesentlich grösser ist als dasjenige anderer Traglastbemessungen. Dieser Tatsache muss durch Überprüfung und Wartung Rechnung getragen werden. Vorsorge gegen Versagen sollte ausserdem durch die Möglichkeit einer Kräfteumlagerung getroffen werden.

1. INTRODUCTION

This paper is an attempt to assess the implicit risk in designing structures for adequate fatigue life in accordance with modern specifications for structural steel design, [1,2,5,18] with the intention of reconciling fatigue design philosophy with limit state design philosophy in general [4,12]. An anomaly exists between design for fatigue and the design for other ultimate limit states, with the risk for the former far greater than that for the latter.

Fatigue design is based upon actual traffic for loads, and fatigue life curves for 95% [1,2] or 97.7% [5] survival at design life. The design concept is shown in Fig. 1 in which the estimated number of cycles, n, must not exceed the design limit, N, for a representative stress range, S_R. In practice there is a probability distribution function (PDF) associated with each parameter. Current design philosophy admits the PDF for N, but assigns a deterministic value to n.

These failure rates of greater than 0.023 are to be compared with nominal rates less than 10^{-3} for other collapse limit states. Such high fatigue failure rates are not acceptable without qualification. It is necessary to review the load and resistance models used to obtain a better estimate of fatigue reliability. The role of redundant load paths and inspection programs in controlling risk must then be taken into account. These matters are considered in the following sections.

2. ESTIMATION OF LOAD EFFECTS

2.1 Design Vehicle or Train

Cumulative damage is accounted for by the Palmgren-Miner Rule [16], allowing for the reduction or abolition of the fatigue limit which occurs under variable amplitude loading with some amplitudes above the fatigue limit [13, 2]. Artificial stress amplitudes producing equivalent fatigue damage can be used because of the linear log S - log N relationship employed for fatigue resistance [see Eq. 1 in section 4]. If a stress amplitude other than S_{RRMS} or S_{RMMS} (MMS = Miner's mean or root mean cube) is used, then the actual number of stress cycles has to be modified to produce the equivalent fatigue damage [10,15,22,23].

A wide range of artificial stress amplitudes are used. BS5400 [5] uses a characteristic vehicle for the estimate of S_R, this vehicle weight being less than the maximum design vehicle for bridge capacity. The design value of n bears a close relationship to the actual value occurring. Since n depends on influence length or span, the value specified in codes is supported by a background analysis of the influence of axle spacing and load spectrum on cumulative damage, represented as a correction to n.

In the AASHTO and AREA specifications [1,2], the maximum design vehicle for bridge capacity is used. This necessarily produces a larger S_R than S_{RRMS} or S_{RMMS}, so that n must be reduced below the actual number of cycles to produce the equivalent fatigue damage. Even more extreme is the UIS proposed λ_T for railway bridges [19]. In this the whole train is reduced in its effect to a single stress cycle S_R (= λ_T $S_{R,UIS}$), producing the same damage as the actual spectrum of stress cycles.

The use of a characteristic vehicle helps to separate load effects from resistance. It locates the design S_R on the S-N curve at a point corresponding to actual stresses. Changes in the design S-N curve which future experience and research might bring can be accommodated without adjustment of the load effect, S_R. Secondly, fatigue damage assessment remains independent of the design

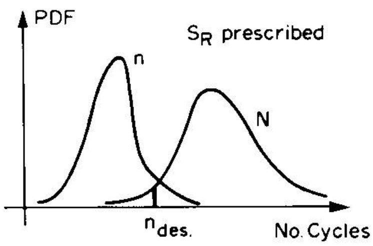

Fig. 1 Load effect, n, and Resistance, N.

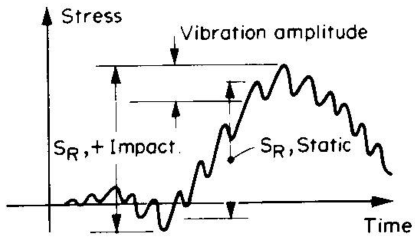

Fig. 2 Vibration and Impact

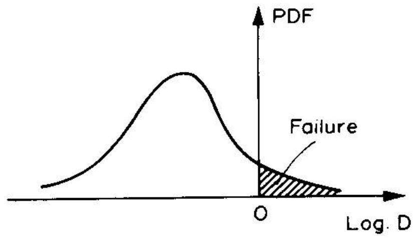

Fig. 3 Damage Function

vehicle used for ultimate load capacity, which can be changed to meet changing legislation on vehicle limits. Fatigue damage is not directly related to such changes.

Essentially, the load effect is the estimated *mean* value of stress amplitudes and frequency of occurrence associated with a given traffic density. This enables the calculation of actual fatigue damage. It is in no way equivalent to the upper 95 percentile characteristic load of floor live loads, for example.

2.2 Traffic Density

One mitigating factor leading to an overestimate of n is the use of categories to define traffic. Whether for cranes or bridges, the effect of using categories, with quantum steps in n between classes of use (motorway vs. arterial road, heavy vs. light crane duty, etc.) is to reduce the mean vs. design traffic density. The effect is similar to that of having discrete member sizes, standard sections, available for selection of design member resistance. The implications of load categorization will be included in the general statement later.

2.3 Correlation of computed with actual load effects

The simplifying models used for structural analysis are sometimes inadequate for calculating realistic values of S_R under specified loads. Many instances have been cited where measured stresses have been less than predicted by theory [7,13,17]. Where this occurs through the participation in structural action of elements normally ignored, such as cladding, topping, timber decks, ballast, sleepers and rails, then the result is wholly beneficial. Byers suggests a mean ratio of 0.93 of actual to computed stress range, with a coefficient of variation of 0.15 [7] for railway girders of longer span. Moses suggests lower ratios for shorter spans [17].

Sometimes the reduction in actual stress can be attributed to load sharing which has been ignored or underestimated by the designer. Cross girders, bracing or diaphragms might then experience stresses greater than calculated. Fatigue failures associated with these details in the past can frequently be attributed in part to a neglect of load sharing in the analysis. There is a case for allowing for the difference between calculated and actual values of S_R. Current codes based upon the S_R concept do not take advantage of this factor.

2.4 Impact Factor

The use of the maximum impact factor required for static strength on all loads in the calculation of fatigue load effect naturally leads to overestimates of damage. Average impact factors, which are much less than maximum values, should be used. To achieve the maximum a serious imperfection in the riding surface is required. A common dynamic response of a bridge to a passing vehicle is a vibration arising from the interaction of vehicle suspension, track roughness and bridge deflection. This vibration is superimposed on the basic static response, amplifying the static S_R by the amplitude of the vibration, and adding many cycles of stress of the amplitude of the vibration (Fig. 2). Usually, the amplitude of the vibration is small compared with the basic static S_R, so that the contribution of the vibration to fatigue damage is not great. The probability distribution of stress amplitudes is, however, changed from the Raleigh distribution assumed in the AREA specification, [10] to some form of exponential distribution, which measurements confirm [3].

Only in BS5400 with the standard fatigue vehicle for highway bridges is the question of impact factor circumvented in damage calculations. Statistical data on impact factor should be used in formulating the load to be used in assessing fatigue damage. Byers estimated the additional load on railway bridges from impact to be 14% with a coefficient of variation of 0.61 [6]. For cranes the picture is more obscure. Oscillations associated with acceleration and deceleration of moving parts can be very significant, and sensitive to damping, so that quantitative estimates of impact factors and cumulative damage for cranes can be very elusive.

3. ESTIMATION OF FATIGUE RESISTANCE

3.1 Statistical Properties of Fatigue Life

To obtain the lower confidence survival curves used in fatigue design, the laboratory data must be evaluated. A problem here lies in reconciling test results from different sources. Each series may have a low coefficient of variation, V_N, on log N but differing mean values, leading to a higher V_N when the results are taken together. Fisher et al. [8] find $V_N = 0.101$ for their own tests for fatigue life at welded cover plate ends, but the scatter with other test results taken into account is much larger [8,14,20]. Diversity in fabricating and testing procedures leads to this systematic difference in results between laboratories, and it is proper that fatigue life characteristics should be based upon several sources in order to include this diversity in the statistical characteristics. There is a serious possibility that the data base does not include adverse environmental effects of actual structures in service. Tests do show differences between field and laboratory [11].

Only BS5400 publishes the statistical assumptions on which the design curves are based. V_N varies from 0.179 to 0.251, values which are much higher than for some individual test series. The design curves of the American and British Codes are fairly consistent on the same details. More detailed comparison is not possible on the published data.

3.2 Identification of Detail

It is important to place a fatigue sensitive detail in the correct fatigue category. Placing the detail in the adjacent category to the correct one can lead to a large overestimate or underestimate of life. The maximum error can be about 60% reduction from estimated life to true life using AASHTO and 42% reduction using BS5400. The difference arises from the fact that the American specifications use four fatigue categories for the same details described in seven categories in the British Code. The error applies to the constant K in the expression for fatigue life (Eq. 1). In all specifications the increments in log K between categories are unequal so that the consequence of incorrectly identifying a fatigue detail are statistically variable.

Some allowance should be made for the designer making an error in classifying detail. This might be described as a professional error [12, 21] in the estimate of K.

3.3 Size Effect

A specific factor in transferring laboratory data to field application lies in the usually larger size of the design structure compared with the test pieces used for the data base. Fisher has proposed a Category E' lower than E, where $K_{E'} = 0.44\ K_E$, for the same cover plate details as E where plate thicknesses exceed 19 mm [11]. This is a large change in fatigue life, solely due to size effect. A similar question must be asked about the size effect for other welded details, but more research is required to answer it.

In the ensuing analysis it is assumed that size effect is considered in the design so that there is no shift in the mean value of K due to it. However, some uncertainty of modeling must be attached to this assumption.

3.4 Design Life

There is no agreement to be found on design life, which is 120 years in BS5400, 80 years in the AREA specification, and unstated in the AASHTO specification, which instead specifies the number of stress cycles to be endured. Whether design life is a genuine target for survival or merely a strategem for achieving some adequate life less than stated has never been elucidated. The concept is fundamental to fatigue design, which is based on a probabilistically non-stationary process, unlike most other limit states, but it is beyond the scope of this paper to attach probabilistic significance to it.

4. ESTIMATE OF RELIABILITY OF PRACTICAL FATIGUE DESIGN

4.1 Damage Equation

The usual fatigue life relationship

$$N = K_2\ S_r^{-m} \qquad (1)$$

where K_2 is a constant yielding an appropriate confidence limit for survival (97.7% for two standard deviations), can be transformed to a damage expression

$$D = K^{-1}\ n\ S_r^{m} \qquad (2)$$

where n is the actual number of cycles applied. When the damage, D, exceeds unity, failure occurs. All terms of Eq. 2 are random functions with appropriate probability distributions.

Current design philosophy obtains what is superficially an expected value of n and S_r, and these are used deterministically to define a load effect, S . The scatter in fatigue resistance, R , is accommodated by modifying K_o to K_2 - a form of partial resistance factor [21]. To bring fatigue design into line with other limit state criteria requires an evaluation of the statistical significance of all the factors mentioned (and no doubt others overlooked) to establish the reliability on a first order basis [4].

Using lognormal distributions, which fit at least some of the data quite well, log D becomes a direct measure of reliability (Fig. 3), with all values of log D greater than zero representing failure.

4.2 Estimation of Reliability of Current Design Procedure

The reciprocal of D is a measure of safety or reliability. It is merely necessary to tabulate the influence of the various parameters in terms of mean values and coefficients of variation as follows:

	Item	Parameter	Mean	V(log)
1.	Design Vehicle/Train Specification	S_{Rd}/S_R	1.0	0.030
2	Traffic density	N_d/N	0.75	0.076
3	Actual vs. computed S_R	S_{Rd}/S_R	0.93	0.150
4.	Impact factor	S_{Rd}/S_R	0.88	0.031
5.	Fatigue life of detail	N_d/N	0.398	0.200
6.	Fatigue category identification	K_o/K_d	1.0	0.100
7.	Size effect	K_o/K_d	1.0	0.100

These figures, arbitrary at times, are derived as follows:

Item 1: Allows for misrepresentation of the load effect by the design vehicle or train. It could include errors due to deficiencies in Miner's rule and the method of counting stress cycles.

Item 2: Assumes design traffic density is 95% confidence limit with mean value in the middle of the traffic range to the next lower traffic range - consistent with BS5400 Highway Traffic designations.

Item 3: Uses Byers' figures for railway bridges [7].

Item 4: Uses Byers' figures of 1.14 impact factor [7] compared with typically 1.30 used in design [BS5400], and Byers value of V_S.

Item 5: These values are chosen to link the determination of D with the Code design point, N, of Mean minus two standard deviations of log N. The value of V_N = 0.2 is an average figure for a range of fatigue categories, and log 0.398 = - 2 x 0.2 for consistency.

Item 6: The value of V = 0.1 is a compromise between (a) an assumed 80% probability of selecting the correct fatigue category with an average factor of 0.631 to the next category in BS5400 and (b) an assumed 85% probability of correct selection with an average factor of 0.501 in the AASHTO specification.

Item 7: Lack of information prevents an estimate of mean ratio other than 1.00 being made, and V_K corresponds to a category identification error.

From the tabulated values, assuming m = 3,

$$\bar{D} = 0.75 \times 0.93^3 \times 0.88^3 \times 0.398 = 0.164$$

$$V_D = [3(0.03^2 + 0.15^2 + 0.031^2) + 0.076^2 + 0.2^2 + 0.1^2]^{1/2} = 0.373$$

$$\beta = \frac{-\log \bar{D}}{V_D} = \frac{0.786}{0.373} = 2.11$$

For comparison the present semi-probabilistic practice would only consider item 5 of the table, with $\bar{D}$ = 0.398, V_D = 0.200, β = 2.0.

The above result could have been couched in the standard *R - S* formulation, with the first four items modifying *S* and the last three modifying *R* , with essentially the same result.

4.3 Discussion

A β-index value of 2.11 is low compared with values accepted for other forms of collapse, where β ranges above 3.1 and is typically 3.5. β = 2.11 representa a nominal probability of failure of 0.0221, compared with 0.000 233 for β = 3.5. Such a low β-value is unacceptable by conventional standards. Account must be taken of three factors so far ignored, if it is to be justified. These are:

1. The significance of inspection and maintenance in preventing collapse, and the cost-benefit implications.
2. The time-dependent nature of the risk of failure, such that an economically useful life is to be expected before collapse or repair.
3. The significance of redundant load-path design in preventing collapse in spite of individual fatigue failures.

It can be shown for a design life exceeding forty years with $\beta \simeq 2$ that even if the cost of failure includes replacement of the structure the marginal cost of modifying the structural details to improve the fatigue category one level must be very low for it to be economically justified. This assumes that there is no risk to human life in the fatigue failure. For this to be possible either inspection sufficient to detect all major fatigue cracks in time must be guaranteed, or the structure must have redundant load paths.

All specifications disregard the role of inspection and maintenance in the formulation of fatigue design rules. In the Ontario Highway Bridge Design Specification [18] redundant load paths are mandatory, thereby rendering the current rules with $\beta \simeq 2$ acceptable, but not necessarily optimum. Only in the AASHTO specification is the risk of fatigue failure in non-redundant load path structures recognised by an approximate reduction in S_R by 40%. Based on the tabulated parameters above, this leads to an estimate of $\beta = 3.89$, a figure quite acceptable by limit state design standards.

The above generalised reliability estimate can be invalid in particular circumstances. For example, using BS5400 to design a girder where maximum traffic is clearly anticipated, e.g. in the slow lane of a motorway, using the design vehicle for fatigue and a rigorous analysis for stresses could lead to the mean correction factors in the table for items 2, 3, and 4 being converted to unity. This would lead to $\bar{D} = 0.398$, $V_D = 0.373$, $\beta = 1.073$; $p_f = 0.142$. Such a high risk is likely to be unacceptable, although it could be justified on economic grounds if failure of the bridge as a whole could be prevented in the event of an individual fatigue failure.

5. CONCLUSIONS

5.1 Present practice in fatigue design which treats stress amplitudes and the number of load cycles deterministically falls short of an adequate limit state format.

5.2 There is lack of consistency in treating factors affecting load effects, such as correlation of measured with theoretical stress amplitudes, impact factor and traffic density. This prevents fatigue design of consistent reliability for different structures. Ideally, the fatigue resistance characteristics of structural steel is the same for all steel structures, and independent of the load effects.

5.3 There is need to evaluate a professional factor allowing for designers' errors in applying the design rules.

5.4 This study indicates an effective safety index, β, not much more than 2.0 in current practice. This represents an unacceptably low reliability unless inspection and maintenance are considered, and/or redundant load path design is employed to prevent catastrophic collapse.

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Ermüdungsfestigkeitsnachweis in den neuen Stahlbauvorschriften der DDR

Fatigue Strength Calculation to the new Steelwork Standard of the German Dem. Rep.

Vérification de la résistance à la fatigue selon la nouvelle norme de construction métallique de la RDA

WOLFGANG GRASSE
Dr. sc. techn.
VEB Metalleichtbaukombinat
Leipzig, DDR

PETER BERGER
Dr. -Ing.
VEB Metalleichtbaukombinat
Leipzig, DDR

ZUSAMMENFASSUNG
Der Nachweis der Ermüdungsfestigkeit in der neuen Stahlbauvorschrift TGL 13 500 gestattet die differenzierte Berücksichtigung unterschiedlicher Spannungsspielzahlen und Formen der Spannungskollektive. Die Überlegungen zur Festlegung der zulässigen Ermüdungsspannungen werden dargestellt.

SUMMARY
The calculation of fatigue strength according to the new specification for steel structures, TGL 13 500, permits the consideration of different numbers of stress cycles and shapes of stress spectra. Some considerations involved in fixing the allowable fatigue stresses are presented.

RESUME
La vérification de la résistance à la fatigue selon la nouvelle norme de construction métallique TGL 13 500 permet de tenir compte de manière différenciée du nombre de cycles de contraintes et de l'allure du spectre des contraintes. Quelques réflexions en vue de la fixation des contraintes admissibles de fatigue sont présentées.

1. BISHERIGE SITUATION UND ZIEL DER NEUBEARBEITUNG DES STANDARDS

Für den Nachweis ermüdungsbeanspruchter Stahlkonstruktionen ist in der DDR die TGL 13 500 [1] verbindlich. Nach der derzeit noch gültigen Ausgabe von 1972 ist der Ermüdungsfestigkeitsnachweis bei Lastspielzahlen zwischen $6 \cdot 10^4$ und $5 \cdot 10^5$ als Zeitfestigkeitsnachweis und bei Lastspielzahlen über $5 \cdot 10^5$ als Dauerfestigkeitsnachweis zu führen. Die zulässigen Spannungen hierfür werden aus kerbfall- und baustahlspezifischen Zeit- bzw. Dauerfestigkeitslinien in Abhängigkeit vom Spannungsverhältnis $\varkappa = \min\sigma / \max\sigma$ entnommen. Diese Werte beruhen auf umfangreichen Ermüdungsversuchen an Kleinproben, wobei gegen die einer Überlebenswahrscheinlichkeit von 90 % entsprechenden Wöhlerlinien mit einem $\varkappa$-abhängigen Faktor abgesichert wurde. Der Sicherheitsfaktor verändert sich mit $\varkappa$ und liegt zwischen 1,1 und 1,3 [2]. Der Ermüdungsfestigkeitsnachweis erfolgt durch Vergleich der Maximalspannung mit der zulässigen Ermüdungsspannung.

Der größte Nachteil der dargestellten Berechnungsmethode besteht darin, daß es nicht möglich ist, die in der Regel vorhandene Veränderlichkeit der Spannungsamplituden im Beanspruchungsprozeß zu berücksichtigen. Der Ansatz der Maximalspannung führt in den Fällen, in denen sie bezogen auf die Gesamt-Lastspielzahl selten auftritt, zu unwirtschaftlichen Bemessungen. Die Beseitigung dieses Mangels und die Möglichkeit einer differenzierteren Berücksichtigung der tatsächlichen Spannungsspielzahl waren die Hauptziele der Neubearbeitung des Ermüdungsfestigkeitsnachweises in der TGL 13 500.

2. ERTRAGBARE BEANSPRUCHUNGEN BEI EINSTUFENBEANSPRUCHUNG

Für die ertragbaren Beanspruchungen bei konstanten Spannungsamplituden liegen vergleichsweise viele an Kleinproben gewonnene Daten vor [3], [4]. Etwa ab 1970 wurden Versuchsergebnisse von geschweißten Bauteilen und auch von nicht geschweißten Großproben bekannt [5], [6], [7], die teilweise erheblich unter den Werten für Kleinproben lagen. Die Ursachen hierfür werden in den höheren Eigenspannungen und im Maßstabseinfluß gesehen, der einen Kerb bei einem großen Bauteil im Verhältnis zu den Gesamtabmessungen schärfer erscheinen läßt.

Durch die mit der neuen Vorschrift beabsichtigte genauere Erfassung der Belastung hinsichtlich Lastspielzahl und Völligkeitsgrad des Spannungskollektivs und den damit im allgemeinen beträchtlichen Abbau von Sicherheitsreserven erschien es geboten, auch die Erkenntnisse über die niedrigere Tragfähigkeit von Bauteilen zu berücksichtigen. Um auch kleine Elemente mit der neuen Norm wirtschaftlich bemessen zu können und in Übereinstimmung mit der Berechnungsvorschrift für geschweißte Maschinenbauteile zu bleiben, wurden sowohl zulässige Spannungen für Bauteile (als Regelfall) als auch solche für Kleinteile aufgenommen.

Die neue Vorschrift TGL 13 500 [8] sieht 10 Kerbfälle vor. Nach den Versuchsdaten wurden die bei $\varkappa = -1$ gültigen Abminderungsfaktoren

$$\alpha = \frac{\text{ertr. } \sigma_{D,\text{Bauteil}}}{\text{ertr. } \sigma_{D,\text{Kleinprobe}}} \tag{1}$$

entsprechend Tabelle 1 festgelegt. Hierbei ist ertr. σ_D die 2 . 10^6 mal ertragene Spannung.

Tabelle 1 Abminderungsfaktoren α

Kerbfall	0 bis 4	5	6	7	8	9
α	0,70	0,75	0,80	0,85	0,90	0,95

Man erkennt, daß die Unterschiede zwischen Bauteilen und Kleinteilen mit wachsender Kerbschärfe der Verbindung kleiner werden.

Tabelle 2 enthält die ertragbaren Spannungen bei $x = -1$ (Wechselbeanspruchung), Einstufenbelastung und 2 . 10^6 Spannungsspielen für Kleinteile. Die ertragbaren Spannungen werden im Streuband der Versuchsergebnisse so festgelegt, daß ihnen eine Überlebenswahrscheinlichkeit von 90 % entspricht.

Tabelle 2 Grundwerte ertr. $\sigma_{D,-1}$ in N/mm^2

Kerbfall	ertr. $\sigma_{D,-1}$ S 38/24	S 45/30	S 52/36	S 60/45
0	144	154	163	174
1	131	136	141	146
2	113			
3	94			
4	75			
5	58			
6	45			
7	35			
8	28			
9	21			

In Bild 1 sind Ergebnisse von Bauteil- und Kleinprobenversuchen dargestellt, die in den Kerbfall 6 entsprechend der neuen TGL 13 500 einzuordnen sind. Weiter sind dort die neuen zulässigen Spannungen eingezeichnet.

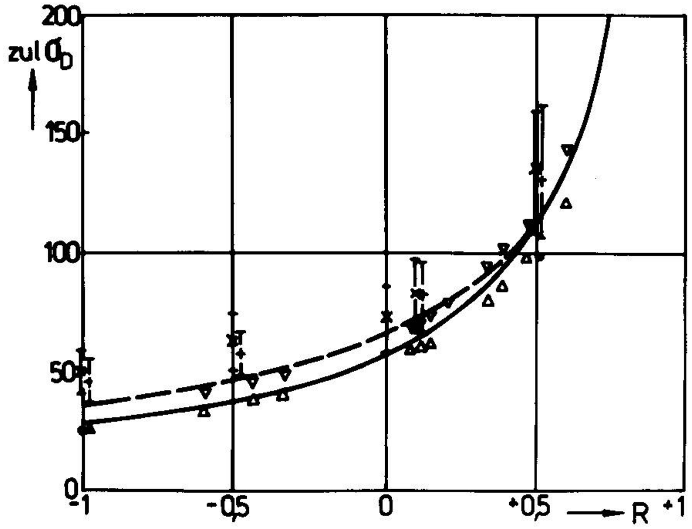

Bild 1 Ergebnisse von Bauteil- und Kleinprobenversuchen nach [5], [6] und [7], zulässige Spannungen für Kerbfall 6 nach [8]

– – – – zulσ_D für Kleinteile

——— zulσ_D für Bauteile

● ▲ △ ▽ Bauteile

+ × Kleinproben

3. ZULÄSSIGE SPANNUNGEN FÜR EINSTUFENBEANSPRUCHUNG

In den Stahlbauvorschriften der DDR wurden die zulässigen Ermüdungsspannungen für einen bestimmten Kerbfall und einen speziellen Baustahl als zulässige Maximalspannung in Abhängigkeit vom Spannungsverhältnis $\varkappa$ angegeben. Die Versuche an Bauteilen zeigten, daß - offenbar bedingt durch hohe Eigenspannungen - die ertragbare Differenz $\Delta\sigma$ = max σ - min σ unabhängig von $\varkappa$ ist. Dieses sogenannte $\Delta\sigma$-Konzept bedeutet eine Vereinfachungsmöglichkeit für den Nachweis der Ermüdungsfestigkeit. Um jedoch den Anschluß an die bisher gültigen Stahlbauvorschriften und an die Vorschriften zur Bemessung geschweißter Maschinenbauteile zu wahren und die Berechnung von Kleinteilen innerhalb der TGL 13 500 nicht nach einem anderen Konzept vornehmen zu müssen, wurde der Nachweis mit max σ und $\varkappa$ beibehalten, wobei die zulässigen Werte für Bauteile voll dem $\Delta\sigma$ -Konzept entsprechen, denn es gilt stets

$$\max\sigma = \frac{\Delta\sigma}{1 - \varkappa} \qquad (2)$$

Die zulässigen Spannungen ergeben sich aus den ertragbaren Werten entsprechend Abschnitt 2 nach Division durch einen Sicherheitsfaktor ν_D, der ν_D = 1,25 beträgt. Bei $\varkappa$ = -1 weisen die so festgelegten zulässigen Spannungen bei Kleinteilen Überlebenswahrscheinlichkeiten von mindestens 99,3 % auf. Die zulässigen Spannungen für Bauteile wurden durch Multiplikation der Werte für Kleinteile mit den α -Werten (Tabelle 1) so festgelegt, daß sie den unteren 2s-Werten der Versuchsergebnisse entsprechen und damit bei Normalverteilung Überlebenswahrscheinlichkeiten von 97,7 % aufweisen.

Bild 2 zeigt die Wöhlerlinien der zulässigen Spannungen von Bauteilen für alle 10 Kerbfälle, Baustahl St 38 und $\varkappa$ = -1. Stähle höherer Festigkeit haben nur in den sehr günstigen Kerbfällen 0 und 1 höhere zulässige Ermüdungsspannungen.

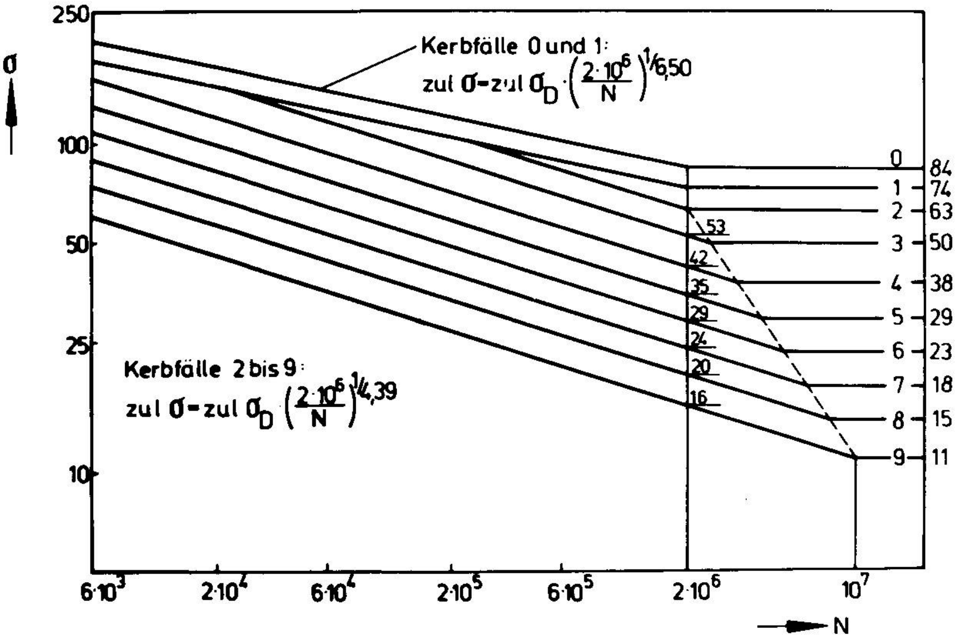

Bild 2 Wöhlerlinien der zulässigen Spannungen (Bauteile)

4. BERÜCKSICHTIGUNG VON BELIEBIGEN SPANNUNGSSPIELZAHLEN UND MEHRSTUFENBEANSPRUCHUNGEN

Entsprechend den in Bild 2 angegebenen Beziehungen für die Wöhlerlinien erhält man bei Spielzahlen unter 2 . 10^6 höhere, über 2 . 10^6 niedrigere zulässige Ermüdungsspannungen. Bei der Festlegung der zulässigen Werte wurde nicht nur das Verhältnis γ_N der ertragbaren Spannung bei einer beliebigen Spannungsspielzahl N zum Wert bei 2 . 10^6 Spannungsspielen berücksichtigt, sondern außerdem ein Sicherheitsfaktor ν_N. Dieser beträgt $\nu_N = 1{,}0$ bei N = 2 . 10^6 und $\nu_N = 1{,}15$ bei N = 2 . 10^5 und dient zum Ausgleich der bei kleinen Spannungsspielzahlen größeren Wahrscheinlichkeit einer Fehleinschätzung des Umfangs des Spannungskollektivs. ν_N wird zwischen o. g. Spielzahlen geradlinig interpoliert. Damit ergibt sich die zulässige Spannung für eine beliebige Spannungsspielzahl N bei Einstufenbeanspruchung zu

$$\text{zul}\,\sigma_x = \frac{\gamma_N}{\nu_N} \cdot \text{zul}\,\sigma_{D,x} \qquad (3)$$

Die Verwendung des Sicherheitsfaktors ν_N bewirkt eine Drehung der Wöhlerlinie der ertragbaren Spannungen um den Punkt σ_D; N = 2 . 10^6, d. h. der Wöhlerlinienexponent der Kerbfälle 2 bis 9 steigt von 3,5 auf 4,39 (s. Bild 2).

Es ist zu beachten, daß N die Zahl der Spannungsspiele ist, die oft beträchtlich höher liegt als die Zahl der Lastspiele.
Der relative Anteil kleiner und großer Spannungsausschläge am gesamten Spannungskollektiv wird durch den Kollektivbeiwert k beschrieben. Durch Klassieren eines regellosen Beanspruchungsprozesses kann man die Häufigkeitsverteilung der Spannungsmaxima und -minima ermitteln. Durch Darstellung der Spannungsspielzahlen im logarithmischen Maßstab und Messen der Spannungsausschläge vom Mittelwert zwischen Maximal- und Minimalspannung des Kollektivs (Symmetrierung) erhält man eine Darstellung, die den Normkollektiven entsprechend Bild 3 rechnerisch oder visuell zugeordnet werden kann. Das Normkollektiv mit k = 0 genügt einer diskreten halben Binomialverteilung. Diese Kollektivform kommt – zumindest näherungsweise – bei vielen praktischen Beanspruchungsprozessen vor und wurde umfangreichen Mehrstufen-Ermüdungsversuchen zugrunde gelegt. Die Kollektivformen mit den Beiwerten k = 1/3 und 2/3 folgen der gleichen Verteilung und ergeben sich durch proportionale Einschaltung zwischen k = 0 und das Vollastkollektiv mit k = 1.

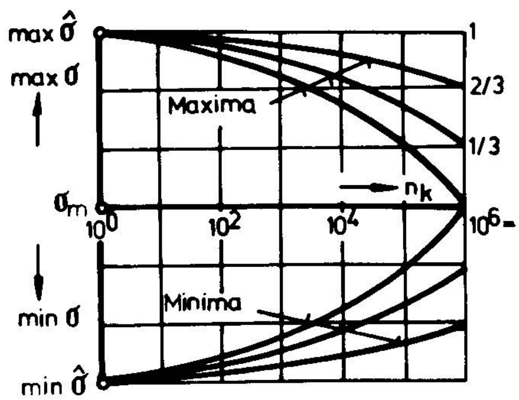

Bild 3 Normkollektive

Die neue Vorschrift TGL 13 500 gestattet sowohl eine Berechnung der zulässigen Betriebsspannung zul σ_{Be} auf der Grundlage der gemessenen oder berechneten Kollektivumfänge N und Beiwerte k als auch ihre vereinfachte Ermittlung über Beanspruchungsgruppen. Tabelle 3 gibt die Zuordnung der Beanspruchungsgruppen B 1 bis B 7 zu den Werten N und k an, während in Tabelle 4 die Betriebsfestigkeitsfaktoren γ in Abhängigkeit von Kerbfall und Beanspruchungsgruppe enthalten sind. Entsprechend Gleichung (4) ergibt sich damit

$$\text{zul}\ \sigma_{Be} = \gamma \cdot \text{zul}\ \sigma_{D,x} \tag{4}$$

wobei zul $\sigma_{D,x}$ als Tabellenwert vorliegt und für das jeweilige x, den Kollektivbeiwert k = 1 sowie N = 2 . 10^6 gilt.

Tabelle 3 Beanspruchungsgruppen

Spannungsspielzahl N	Kollektivbeiwert k 0	1/3	2/3	1
6 . 10^3 N = 2 . 10^4	B 0	B 0	B 1	B 2
2 . 10^4 N = 6 . 10^4	B 0	B 1	B 2	B 3
6 . 10^4 N = 2 . 10^5	B 1	B 2	B 3	B 4
2 . 10^5 N = 6 . 10^5	B 2	B 3	B 4	B 5
6 . 10^5 N = 2 . 10^6	B 3	B 4	B 5	B 6
2 . 10^6 N = 6 . 10^6	B 4	B 5	B 6	B 7
6 . 10^6 N = 2 . 10^7	B 5	B 6	B 7	B 7
2 . 10^7 N = 6 . 10^7	B 6	B 7	B 7	B 7
N 6 . 10^7	B 7	B 7	B 7	B 7

Tabelle 4 Betriebsfestigkeitsfaktoren γ

Beanspruchungsgruppe	Kerbfall 0 und 1	2	3	4	5	6	7	8	9
B 0	2,84	4,83							
B 1	2,39	3,71							
B 2	2,01	2,86							
B 3	1,69	2,20							
B 4	1,42	1,69							
B 5	1,19	1,30							
B 6	1,00	1,00							
B 7	1,00	1,00	0,96	0,92	0,86	0,82	0,77	0,73	0,69

Der Betriebsfestigkeitsfaktor γ in Gl. (4) ist definiert durch

$$\gamma = \frac{\gamma_N}{\nu_N} \cdot \frac{\gamma_k}{\nu_k} \ . \tag{5}$$

Die Werte γ_N und ν_N beziehen sich auf die Spannungsspielzahl und wurden bei Gl. (3) erläutert, γ_k wird auf der Grundlage der Schadensakkumulationshypothese von CORTEN/DOLAN berechnet und bringt die Erhöhung des ertragbaren Maximalwertes des Kollektivs bei "weichen" Kollektiven (kleine Werte k) zum Ausdruck. Um der bei abnehmender Kollektivvölligkeit steigenden Gefahr einer Fehleinschätzung vorzubeugen und den ungünstigen Randomeinfluß abzudecken, wird mit einem Sicherheitsfaktor ν_k abgesichert, der von $\nu_k = 1$ bei $k = 1$ auf $\nu_k = 1{,}6$ bei $k = 0$ wächst.

Es sei noch bemerkt, daß der Ermüdungsfestigkeitsnachweis wegen des geringen Schädigungseinflusses nicht mit der absolut größten Lastspannung zu führen ist, sondern mit dem Wert, der pro 1 Million Spannungsspiele einmal auftritt. Demzufolge ist nachzuweisen, daß die mit einer Wahrscheinlichkeit von 10^{-6} auftretende Maximalspannung kleiner ist als der Wert zul σ_{Be} nach Gl. (4).

5. EINSCHÄTZUNG DES NEUEN NACHWEISES DER ERMÜDUNGSFESTIGKEIT

Es ist offensichtlich, daß der neue Nachweis eine wesentlich flexiblere und damit wirklichkeitsnähere Bemessung ermüdungsbeanspruchter Stahlbauteile gestattet. Bei den sehr häufig auftretenden Kollektiven im Bereich von $k = 0$ und $1/3$ sind bei nicht übermäßig hohen Spannungsspielzahlen (etwa unter $6 \cdot 10^6$) selbst unter Beachtung des ungünstigen Bauteileinflusses leichtere Dimensionierungen möglich als bisher. Sind dagegen die Spannungskollektive nachgewiesenermaßen völlig und haben einen großen Umfang, so ergeben sich besonders im Bereich der Wechselbeanspruchung ($\varkappa = -1$) materialintensivere Bemessungen als bisher.

Die TGL 13 500 ist eine Grundvorschrift im Vereinheitlichten Vorschriftenwerk für den Stahlbau in der DDR und gibt Tragfähigkeitswerte an, die für den gesamten Stahlbau gelten. Sie ist im Zusammenhang mit den Vorschriften für die einzelnen Fachgebiete, z. B. für den Stahlbrückenbau, Kranbau und Stahlhochbau anzuwenden, die u. a. die Lastannahmen vorschreiben und damit ebenfalls das Bemessungsergebnis beeinflussen. Es ist Aufgabe dieser Fachgebiete des Stahlbaus, die maßgebenden Beanspruchungskollektive zu ermitteln, um durch Zuordnung zu den Normkollektiven der TGL 13 500 oder eine genauere Ermittlung der zulässigen Betriebsspannung von den Möglichkeiten der neuen Vorschrift Gebrauch zu machen.

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Basis of Fatigue Design for Welded Joints

Principes des règles de calcul des assemblages soudés

Grundlagen der Ermüdungsbemessung geschweisster Verbindungen

T.R. GURNEY
Head
Struct. and Comp. Dep., the Welding Inst.
Abington, Cambridge, England

SUMMARY
The paper considers the general form of the S-N curves which should appear in fatigue design rules for welded joints. It is agreed that, for as-welded joints, they should be based on stress range using all available test results from the literature. The curves should be linear on a Log S v. Log N basis, with a slope m = 3, probably with a single band to a shallower (but fictitious) slope at 10 million cycles for cumulative damage calculations. For stress relieved joints the allowable stress range could be increased for partially compressive loading.

RESUME
L'article traite de la forme générale des courbes S-N que l'on doit prendre en considération dans les règles de calcul à la fatigue des assemblages soudés. Il est admis que, pour les assemblages soudés sans traitement ultérieur, les règles doivent être basées sur la différence des contraintes en exploitant tous les résultats d'essais de la littérature. Les courbes doivent être linéaires dans une échelle Log S - Log N avec une pente m = 3, probablement avec un domaine singulier de pente plus faible (mais fictive) au-delà de 10 millions de cycles pour le calcul de dommages cumulés. La différence de contraintes admissible peut être augmentée dans le cas d'assemblages soudés ayant subi un traitement thermique et soumis à des charges partielles de compression.

ZUSAMMENFASSUNG
Der Beitrag behandelt in allgemeiner Form die S-N Kurven, die für die Ermüdungsbemessung geschweisster Verbindungen berücksichtigt werden sollten. Die Bemessungsregeln für Verbindungen ohne Nachbehandlung sollten auf der Spannungsdifferenz fundieren unter Berücksichtigung aller erhältlicher Versuchsresultate aus der Literatur. Die Wöhlerlinie sollte im doppeltlogarithmischen Massstab eine Gerade mit der Neigung m = 3 darstellen. Für Schadenakkumulationsberechnungen kann ab 10 Millionen Lastwechseln eine flachere (fiktive) Neigung angenommen werden. Im Fall von spannungsarm geglühten Schweissverbindungen kann der zulässige Spannungsbereich bei teilweiser Druckbeanspruchung erhöht werden.

1. INTRODUCTION

Fatigue design rules for welded joints have existed in several countries for many years. The majority of them, at least for high cycle fatigue, have tended to relate particularly to bridges, but in the absence of any alternative they have in many cases been used for the design of other types of structure as well. They are therefore of considerable significance.

It so happens that in several countries the fatigue design Standards have been updated comparatively recently, and it is known that others are currently in course of preparation. Also several attempts are now being made on an International basis (e.g. by I.S.O., IIW and ECCS) to produce other fatigue design rules. At first sight one would expect all these rules to be very similar, since the basic data available to the writers of the rules must have been similar, most of it having been published. In fact, however, there are still significant differences between rules prepared in different countries. It is hoped, therefore, that it will prove useful to review some of the variables which should be covered by fatigue design rules.

2. DEFINITION OF BASIC S-N CURVES

As far as welded joints are concerned it is not yet possible to predict, by means of fracture mechanics, the S-N curves for all joint geometries. In theory that is a possibility but in practice it would require knowledge of the stress intensity factor for each type of joint, together with details of the expected pre-existing defects and of the weld shape in the vicinity of those defects. Such information does not, in general, exist. It is therefore inevitable that the basic S-N curves appearing in fatigue design rules must be based on experimental laboratory results, the great majority of which relate to constant amplitude loading.

The first stage in writing fatigue design rules must therefore be the analysis of available experimental data, but it is important only to include relevant data. Thus it is wise to exclude:-

(a) Data obtained a long time ago, since in many cases their accuracy is suspect and in any case welding methods have improved over the years. In the analysis carried out in relation to the British rules [1] any results obtained prior to 1950 were automatically excluded;

(b) Data obtained other than under tensile loading. This restriction relates to the influence of residual stresses and will be considered in more detail later.

On the other hand it has been shown in numerous investigations [2] that the high cycle fatigue strength of welded joints in structural steels is independent of the static strength of the parent material so that results for all such steels can be included.

It is also important to recognize at this stage that fatigue test results inevitably show considerable scatter in life for a given stress, as a result both of variations in overall geometry for a particular type of joint (e.g. attachment size, weld size, etc.) and also because of small variations in weld profile and toe defect geometry. By way of example tests at The Welding Institute on nominally identical non-load-carrying fillet welds made by twenty welders showed a scatter of life of approximately 5:1, while even specimens made by the same welder produced a scatter of about 3.6:1. Clearly, therefore, in order to allow for the variations which will inevitably occur in practice, it is essential to use data from as many sources as possible and not to base design rules on data generated by, for example, a single laboratory.

Most of the data currently available relate to the endurance range 10^5 to 2×10^6 cycles and there now seems to be general agreement that, in that region at least, the curve is a straight line when plotted on the basis of Log S v. Log N. That is also consistent with a fracture mechanics approach [3] assuming that the whole of the life is taken up in crack propagation - i.e. that fatigue cracks develop from small pre-existing defects or 'cracks'. On that basis it is easy to show that the slope, m, of the S-N curve as defined by

$$S^m.N = \text{constant}$$

should be equal to the exponent in the Paris crack propagation equation

$$\frac{da}{dN} = C.(\Delta K)^m$$

In effect this has been confirmed since an analysis of crack propagation data for weldable structural steels [4] has shown that the value of m is typically in the range 2.4 to 3.6 with a mean value of 3.0. Equally an analysis of fatigue test data for welded joints containing high tensile residual stresses and subjected to tensile loading also showed that the average slope was typically about 3.0, at least for the lower strength joints (e.g. joints with longitudinal fillet welds) which are normally critical from the design point of view.

In some instances, notably in Germany, it has been proposed [5] that the slope of S-N curves should be assumed to be considerably flatter with m typically equal to 3.75. Indeed it has been shown that, given a wide enough scatter band increasing in width as stress decreases, several sets of fatigue data can be made to fit such a slope. However in assessing the significance of those results it has to be remembered that S-N curves with shallower than normal slopes can arise for several reasons, among which are the following:-

(a) If the joints are tested in bending (of the plate) rather than axially. In this respect tests on beams are obviously different, and therefore satisfactory, since the stress in the flanges will be essentially axial;

(b) If the mode of failure does not involve cracks originating at pre-existing toe defects, so that it is necessary both to initiate and propagate a crack. This is true of most of the higher strength joints, such as continuous longitudinal welds (e.g. web to flange joints) and joints which have been improved by, for example, dressing the weld toe. Since the initiation period is longer at lower stresses the S-N curve is rotated to a shallower slope;

(c) If the joints have low residual stresses, due either to stress relief or to the fact that the specimens were too small to hold residual stresses. The latter is particularly true of joints with transverse welds and is a very common fault of specimen design. The whole problem of residual stresses is considered in more detail below.

In most structures the stresses are essentially axial, the design is not governed by joints with high fatigue strength and the joints are likely to contain high residual stresses. Consequently one would not expect S-N curves with shallow slopes to be relevant.

For ease of computation there is some benefit both in having m as an integer value and in having as many S-N curves as possible parallel to each other. Thus, in the light of all these considerations it would seem realistic to base the analysis of the experimental data, at least for all joints failing from the weld toe, on an assumed straight line relationship between Log S and Log N with a slope m = 3.0.

3. INFLUENCE OF RESIDUAL STRESSES

It has been noted above that the slope of the S-N curve can be considerably influenced by the presence of residual stresses, and they also influence the choice of stress parameter to be used in design rules.

The majority of welded structures are not stress relieved so that, in them at least, it is realistic to assume that high tensile residual stresses of yield stress magnitude will exist in some places. In general the only sensible assumption which can be made is that such stresses may exist at any point where a fatigue crack could initiate. In simplified terms, assuming the residual stress to be equal to yield stress tension, the actual stress cycle to which the material adjacent to the weld will be subjected under applied cyclic loading will vary from yield stress tension downwards, regardless of the nominal stress cycle. For example, if the nominal stress cycle is $+\sigma_1$ to $-\sigma_2$, giving a total range equal to $(\sigma_1 + \sigma_2)$, the actual stress cycle will vary from $+\sigma_y$ to $\{\sigma_y - (\sigma_1 + \sigma_2)\}$. In other words the fatigue behaviour of a welded joint in a real structure can be expressed in terms of stress range alone, and there is no need to consider the stress ratio $R(= S_{min}/S_{max})$.

It can therefore be argued that cycles which are (nominally) partially or even wholly compressive should be just as damaging as cycles which are fully tensile. To some extent this has been confirmed experimentally. For example Fig. 1 shows some results for specimens with longitudinal non-load-carrying fillet welds tested at R = 0 under compressive loading. All the results lie within the scatter band for tensile loading although within the upper half of it. For comparison some specimens of the same batch were also tested under tensile loading and the results are also shown in Fig. 1; clearly they can be considered as identical to the earlier tensile test results. These, and other similar, data tend to confirm the validity of basing design stresses on stress range alone. However, recent results obtained under compression-to-compression loading suggest that this approach may be unduly conservative for that situation, but further check tests are required before a final conclusion can be reached.

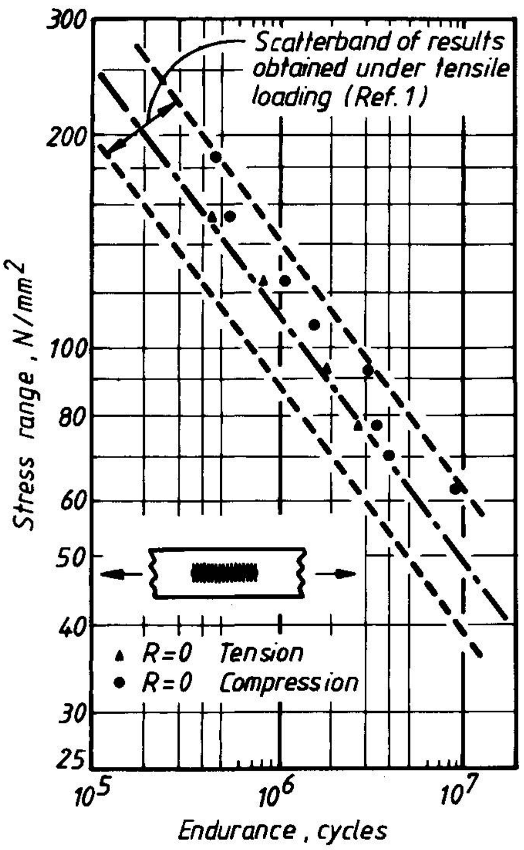

Fig. 1 Comparison of test results for tensile and compressive loading

In contrast to welded structures many welded specimens do not contain high tensile residual stresses because they are too small to provide the necessary restraint; typically such specimens are usually only about 100-150mm wide x 12mm thick, or even less. It is for this reason that it was suggested previously that only results obtained under tensile loading should be used in deriving design rules.

Nevertheless it is reasonable to assume that, in the as-welded condition, specimens with the welds parallel to the loading direction ('longitudinal' welds) are likely to contain high residual stresses, since the heated width during welding will be localized so that residual stresses are able to form. In contrast, with transverse welds a relatively large proportion of the cross section will be heated at one time which will tend to inhibit the formation of residual stresses.

The difference between these two situations has been investigated by carrying out tests under pulsating tension (R = O) loading on specimens with longitudinal non-load-carrying fillet welds, one series being stress relieved (and thereby simulating the situation existing in small specimens with transverse welds) and one being as-welded. These results are shown in Fig. 2, from which it is clear that the as-welded specimens gave the steeper S-N curve, although the difference in fatigue strength is quite small. This is due to the as-welded specimens cycling at a higher mean strain than the stress relieved specimens when both are subjected to the same nominal stress range.

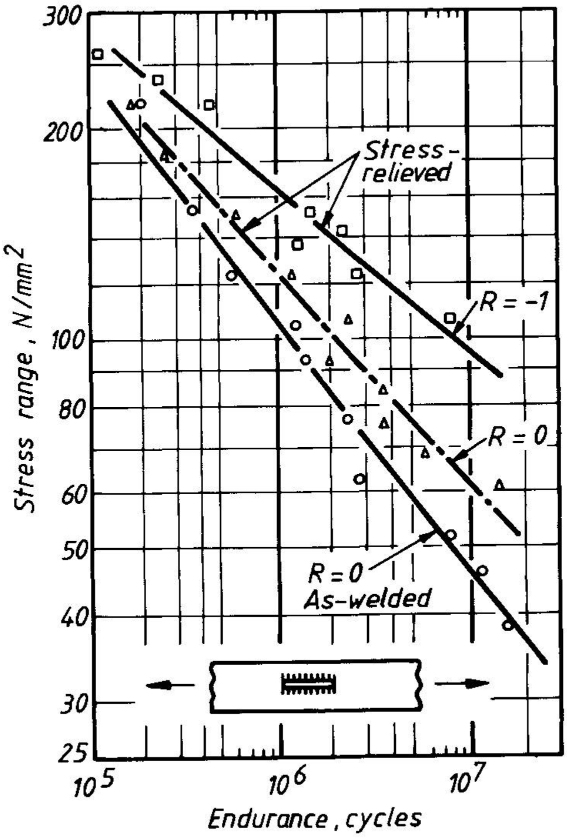

Fig. 2 Influence of stress ratio and stress relief on fatigue strength

Figure 2 also shows some results for stress relieved specimens tested at R = -1. Clearly these show a large increase in stress range and tend to confirm that if a joint contains low residual stresses and is subjected to partly compressive loading fatigue strength is increased. Thus it is obviously unsafe to use such results to define design rules for more severe conditions (i.e. for tensile loading and high residual stresses).

It may therefore be concluded that S-N curves for as-welded joints should be based on stress range and be independent of stress ratio, although there may well be a case for increasing design stresses for joints subjected only to compression-to-compression loading. For stress relieved joints it would certainly be safe to increase the design stress range for joints subjected to partially compressive loading. In this context it has been proposed that the revised British rules for the design of offshore structures should specify that such joints may be designed for a stress range equal to the tensile component of stress plus only sixty per cent of the compressive component.

4. THE S-N CURVE AT SHORT AND LONG ENDURANCES

Given that it is realistic to assume that the S-N curve is linear in the intermediate life region, there is still a need to define its form for design purposes in the low and high life regions.

As far as the short life situation is concerned there is no real problem, since it is known that the Log S v. Log N line can certainly be extrapolated up to a stress at least equal to yield. However it is also true that a design will normally be limited to an upper limit stress which is less than yield stress. Hence in the low life region it seems reasonable merely to extrapolate the design S-N curves back to yield stress.

In the high life region the situation is more complicated. If tests are carried out under constant amplitude loading a fatigue limit will ultimately be found below which failures do not occur. There is some evidence that this limit will occur at different lives for different classes of joint; in effect the lower the joint class the longer the life and the lower the fatigue limit stress. However, for simplicity of design, there is some benefit in defining that the fatigue limit will correspond to a particular life regardless of class. In the British rules 10^7 cycles was the selected life, based on a fracture mechanics analysis, but there are theoretical indications that this may be too short for some types of joint, particularly those in thick plates.

However in most instances the position of the constant amplitude fatigue limit is irrelevant since, in service, a joint will usually be subjected to a stress spectrum with some stresses above, and some below, the fatigue limit. In this situation a conventional cumulative damage calculation by, for example, Miner's rule is fallacious because, if applied literally, the stresses below the fatigue limit will be assumed to do no damage. In fact the higher stresses in the spectrum will propagate a crack and as the crack grows the lower stresses will progressively become effective in helping it to grow.

This problem has been investigated by fracture mechanics and it has been shown, for a wide variety of stress spectra, that a reasonable design solution is to lower the 'cut-off' to the stress corresponding to 2×10^7 [6]. An alternative solution, originally proposed by Haibach, which was subsequently adopted in the new British design rules, is to introduce a fictitious bend in the S-N curve to a shallower slope ($m = 5$). The advantage of the latter approach is that it avoids the anomaly which can occur when using the Miner summation method in conjunction with an S-N curve with a sharp cut-off. That produces a step change in the value of $\Sigma \frac{n}{N}$ depending on whether a particular stress level is just above or just below the cut-off stress.

It will be seen, therefore, that in design rules it is really necessary to have two 'cut-off' stresses (see Fig. 3), one corresponding to the constant amplitude fatigue limit (say 10^7 cycles) and one being relevant to cumulative damage calculations. The former may be applicable both under constant amplitude loading or if the expected number of cycles is very large, in which case it may be necessary to keep <u>all</u> stress ranges below the limit. The latter may either consist of a fictitious cut-off at (say) 2×10^7 cycles, which has the merit of simplicity but can lead to difficulties of interpretation for stresses close to the cut-off stress, or may take the form of a bent S-N curve; this avoids problems of interpretation but is more difficult to justify theoretically. Nevertheless for design purposes it is probably the better solution.

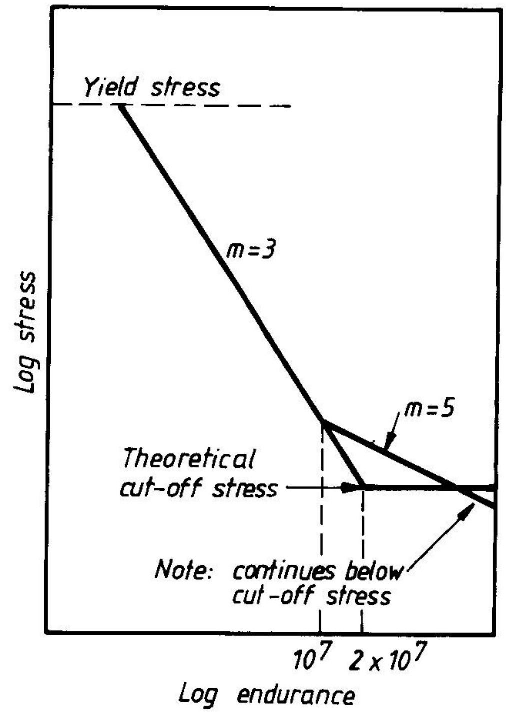

Fig. 3 Possible forms of S-N curve at long life

An interesting implication of this approach is that, except for very special applications or purely for research purposes, there is no point in continuing fatigue tests to long lives. As far as defining the design stress for a particular joint type is concerned all that is required is the S-N curve in the intermediate life (10^5 - 2×10^6 cycles) region. This can greatly reduce the cost of fatigue testing since testing machines are expensive. Looked at from a different angle, supposing that one does carry out tests to longer lives and obtains some unbroken specimens, what can one do with those results; surely they are more or less useless.

5. JOINT CLASSIFICATION

Having defined the general form of the S-N curves the main problem which remains is to fix their positions for each type of joint. As far as is known, in all the Standards produced to date, the curves have been arranged to fit the test results usually by amalgamating several types of joint into a simple 'class' and allocating a curve to the class. Recently it has been proposed that the process should be reversed and that there should be a standard set of parallel S-N curves, arranged in approximately geometrical progression of stress. The joint classification would then consist of fitting the test results to the set of curves and selecting the one which seemed most appropriate.

This certainly seems to be the better procedure, provided that the gradation of curves is reasonable. If one were to assume a spacing between curves of about ten per cent, and a need to cover a range of fatigue strengths from about 40 to 200 N/mm^2 at 2×10^6 cycles, approximately fifteen curves would be needed. This is more than in most National Standards at the moment, but having a fine gradation considerably eases the problem of compromise between different Standards by reducing the severity of the changes that may be necessary.

An important point which arises, however, is to decide whether the curves represent the mean of the data or the design stress and, if the latter, what 'factor of safety' should be introduced. This is complicated by the fact that some joints give far more scatter than others. For example Fig. 4 shows a comparison between the scatter bands (mean ± 2 standard deviations) for transverse and longitudinal non-load-carrying fillet welds [1]. It will be seen that although the mean strength of the longitudinal welds is lower than that of the transverse, the lower limit stresses are reversed in order because of the lower scatter of the longitudinal welds. In view of this problem it is believed that the curves in a design standard should be the design curves and not the mean curves, and that they should be assumed to represent the mean - two standard deviations stress. That should be suitable for most types of structure.

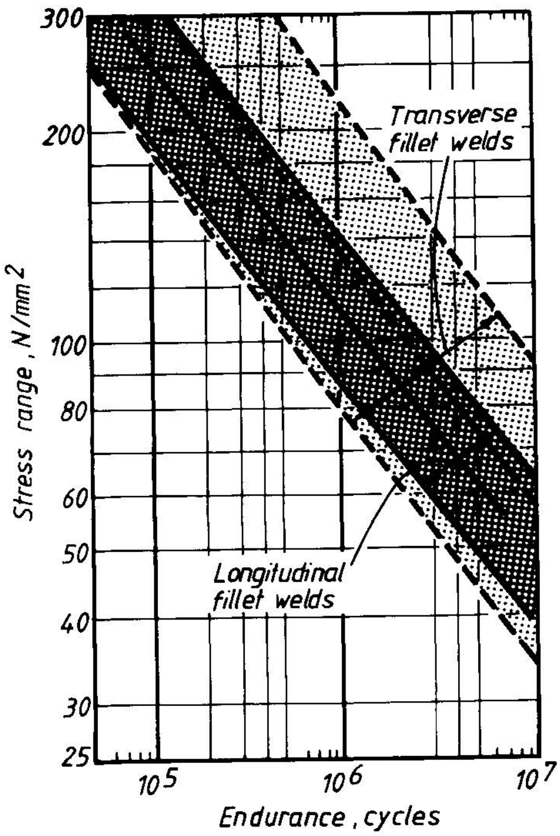

Fig. 4 Comparison of scatter for two types of fillet welded joint

6. CONCLUDING REMARKS

In summary, therefore, the following suggestions can be made regarding the form of fatigue design rules for welded joints:-

(1) For as-welded joints the design stresses should be based on stress range, regardless of stress ratio. For joints subjected to fully compressive loading there may be a case to relax that requirement but further research is still required on that problem. For stress relieved joints an increase in stress range can be permitted if the stress cycle is partly compressive; it is suggested that this would involve only taking into account sixty per cent of the compressive half cycle.

(2) It is important to base design stresses on as wide a range of available results as possible and not to base them on results obtained in a single laboratory or even a single country if that can be avoided.

(3) To simplify the designer's work and to ensure consistency with the fracture mechanics approach it seems reasonable to make all S-N curves (at least those for joints involving failure from the weld toe) parallel with a slope $m = 3.0$.

(4) Although it can be shown by fracture mechanics methods that reasonable design stresses are produced, for variable amplitude loading situations, by introducing a 'fatigue limit' at 2×10^7 cycles, it is probably more practical to bend the curve to a shallower slope (see Fig. 3). This avoids difficulties of interpretation for stresses in the region of the 'fatigue limit'.

However, regardless of the fact that fatigue design rules must inevitably cover several different aspects of the problem, there is no doubt that the most important part is the joint classification system and the associated S-N curves. In effect this represents an enforced educational exercise for the designer because it will inevitably tend to encourage him to avoid using joints with high stress concentrations and low fatigue strengths. By that means alone designs can be greatly improved. In other words good fatigue design must start with GOOD DETAILS.

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Bemessungskonzept der UIC für Eisenbahnbrücken

UIC Concepts for Steel Railway Bridges

Concept de dimensionnement de l'UIC pour les ponts-rails métalliques

H. SIEBKE
Prof. Dr. -Ing. Ministerialrat
Deutsche Bundesbahn
Frankfurt a.M., BRD

ZUSAMMENFASSUNG
Die internationale Harmonisierung bautechnischer Regelwerke wird am Beispiel des UIC-Merkblattes 778-1 —Empfehlungen zur Berücksichtigung der Ermüdung bei der Bemessung stählerner Eisenbahnbrücken — dargestellt. Dabei wird das Grundsätzliche der angewandten Methode an der Entwicklung und den Voraussetzungen aufgezeigt. Die neuzeitlichen Ergebnisse der Betriebsfestigkeitsforschung werden berücksichtigt und die abmessungsrelevanten Einflüsse der Beanspruchungen herausgestellt.

SUMMARY
UIC leaflet 778-1 "Recommendations on Fatigue Factors to be Considered in the Design of Steel Railway Bridges" is presented as an example of the international harmonization of construction regulations. The principle of the method proposed is discussed with respect to the basis of the development and the necessary assumptions. Recent results of research in the field of resistance in service are taken into consideration and the influence of the dimensions on the stresses emphasized.

RESUME
L'harmonisation internationale des règlements de construction est présentée par l'exemple du feuillet UIC 778-1 — Recommandations relatives aux facteurs de fatigue à considérer pour le dimensionnement des ponts-rails métalliques. Le principe de la méthode proposée est exposé sur la base du développement et des hypothèses nécessaires. Les récents résultats de la recherche dans le domaine de la résistance de service sont pris en considération et les influences des dimensions sur les sollicitations sont soulignées.

1. INTERNATIONALE ZUSAMMENARBEIT

Harmonisierung der Regelwerke für die Bemessung von Bauwerken ist eine Voraussetzung, um auch in diesem Bereich die Handelsbeziehungen über nationale Grenzen hinweg auszuweiten. Es ist aber auch eine Möglichkeit, um weltweit gemachte Erfahrungen im nationalen Bereich zu nutzen. Läßt man wirtschaftliche Aspekte außer acht, bleibt die traditionelle Gepflogenheit, wissenschaftliche und technische Erkenntnisse über die nationalen Grenzen hinweg unter Fachleuten auszutauschen. Alle drei Betrachtungsweisen haben bei der Ausarbeitung des UIC-Merkblattes 778 "Empfehlungen zur Berücksichtigung der Ermüdung bei der Bemessung stählerner Eisenbahnbrücken", über welches berichtet werden soll, eine Rolle gespielt.

UIC ist die 60 Jahre alte Vereinigung der Eisenbahnverwaltungen, die dem grenzüberschreitenden Verkehr auf dem Schienenweg Rechnung trägt. Durch Vereinheitlichung bei Gleisen, Wagen und Lokomotiven sowie Absprachen und Vereinbarungen bei Fahrplänen und Tarifen sind die nationalen Eisenbahnnetze zusammengewachsen. Vor rd. 30 Jahren führte ein sporadischer Erfahrungsaustausch der für die Eisenbahnbrücken verantwortlichen Ingenieure zu einer ständigen Zusammenarbeit im Rahmen der UIC. Dieser am Anfang unverbindliche Austausch von Ansichten machte bald deutlich, wie sich durch das Infragestellen der eigenen, von der nationalen Tradition geprägten Meinung, im internationalen Kreis neue Erkenntnisse und wirtschaftliche Erfolge erzielen lassen. Wie auch in anderen Bereichen zu beobachten ist, wird zukünftig auch im Bauwesen entscheidender Fortschritt nur in internationaler Zusammenarbeit zu erreichen sein. Dies liegt vor allem an der Notwendigkeit, zur Beantwortung der immer komplexer werdenden Fragestellungen sowohl umfangreichere Versuchsreihen als auch aufwendigere Theorien zu benötigen. Ein derartiger Aufwand ist nur in Kooperation zu bewältigen. Die weltweit zu beobachtenden Anstrengungen bestätigen diese These. Ob solche Bemühungen von Erfolg gekrönt sein werden, wird weitgehend davon abhängen, ob es gelingt, den Prozeß einer Harmonisierung als solchen mit seinen arteigenen Gesetzmäßigkeiten zu erkennen und in den Griff zu bekommen.

Im Unterausschuß Brücken der UIC hat sich ein nachweisbarer Harmonisierungserfolg eingestellt. Es könnte daher hilfreich sein, am Beispiel des Merkblattes 778 nicht nur den endgültigen Inhalt, sondern auch den Weg, der dazu geführt hat, darzustellen.

2. HARMONISIERUNG DER EISENBAHNLASTEN

Die UIC-Frage 7/J/2a wurde eingerichtet, um die Bemessungsmethoden für stählerne Eisenbahnbrücken zu studieren. Als Faktum ergab sich nach einer Fragebogenaktion eine unübersichtliche Vielfalt von Bemessungsregeln, die im eigenartigen Widerspruch zu dem gleichartigen Aussehen der meisten Brücken stand. Besonders deutlich waren die unterschiedlichen Auffassungen hinsichtlich der anzunehmenden Verkehrslasten und des anzusetzenden Stoßfaktors, der eine aus den dynamischen Ablauf der Belastung herrührende Vergrößerung der statisch wirkenden Lasten berücksichtigen soll (Bild 1).

Abhilfe sollte ein international durchzuführendes Meßprogramm schaffen. Man hoffte durch eine statistische Auswertung der Meßergebnisse die nötigen Lasterhöhungswerte erkennen zu können, erzielte dann zwar zahlreiche Detailerkenntnisse, aber auch die Einsicht: Ohne Theorie, d. h. ohne "Vorurteil", welches Ergebnis eigentlich erwartet wird, sind Messungen wenig aussagekräftig. In Zusammenarbeit mit dem ORE (Forschungs- und Versuchsamt des Internationalen Eisenbahnverbandes mit Sitz in Utrecht) wurde dann eine Theorie für den Schwingbeiwert aufgestellt, in Modellversuchen überprüft und

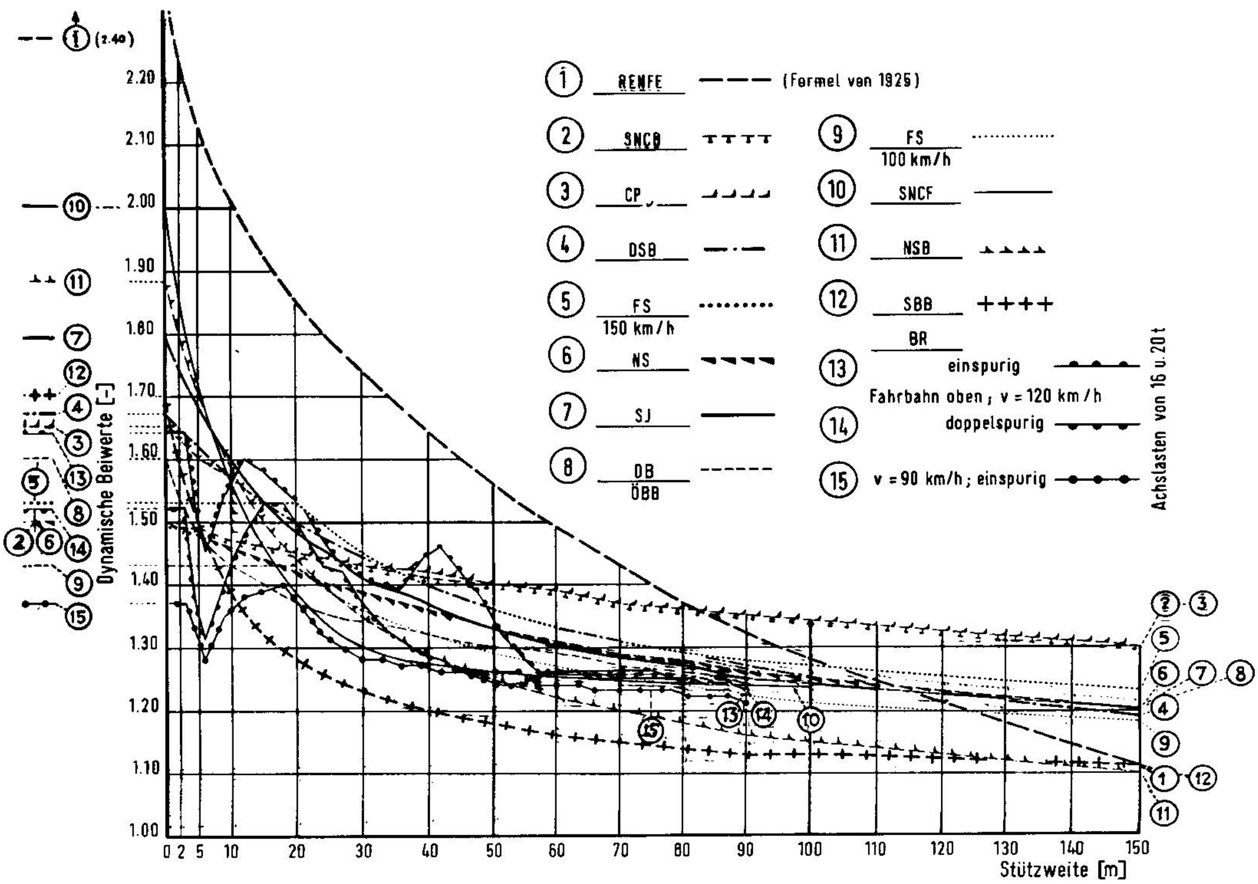


Bild 1: Dynamische Beiwerte nach seinerzeit gültigen Formeln der Eisenbahnverwaltungen (Eisenbahnbrücken aus Stahl, ohne Schotterbett, Schienen auf Schwellen ohne Schienenstöße).

auf die Meßwerte an Eisenbahnbrücken übertragen. Mit Hilfe moderner Datenverarbeitung gelang es, diese Theorie zu verfeinern und besonderen Gegebenheiten wirklichkeitsnah anzupassen. Um sie für die Praxis handhabbar zu machen, war diese Theorie vom UIC-Unterausschuß "Brücken" zu generalisieren. Hierzu war die Spanne wahrscheinlicher Eigenfrequenzen und Dämpfungen sowie auch eine Serie von Idealtypen für Betriebslastenzüge festzulegen und gleichzeitig das Belastungsbild für die statische Bemessungsbeanspruchung zu definieren, da beide Angaben als Einheit zu sehen sind.

Um unnötige Sicherheitsmargen abzubauen hatte man sich zum Ziel gesetzt, die Wirkung schnellfahrender, aber relativ leichter Personenzüge und die Wirkung langsamer, aber schwerer Güterzüge gleichzeitig abzudecken. Außerdem sollte das Belastungsbild einfach sein, damit die Auswirkungen konstruktiver Variationen im Verlauf einer Entwurfsbearbeitung überschaubar bleiben. Mit diesen Forderungen entfernte man sich weit von bisherigen Vorstellungen: Das Belastungsbild UIC 71 (Bild 2) ist kein idealisierter Eisenbahnzug und der Schwingbeiwert Φ ist kein dynamischer Effekt (Bild 3) des Belastungsbildes UIC 71! Es deckt in einer statischen Berechnung die wahrscheinlichen Beanspruchungen, die in Brücken aus der Überfahrt von Zügen entstehen können, ab. Das Belastungsbild ist klassifizierbar, d. h. durch Angabe eines logarithmisch gestuften Faktors können Lastannahmen für schwerere oder leichtere Eisenbahnverkehre getroffen werden oder umgekehrt, es können bestehende Lastannahmen oder bestehende Eisenbahnverkehre daran gemessen werden.

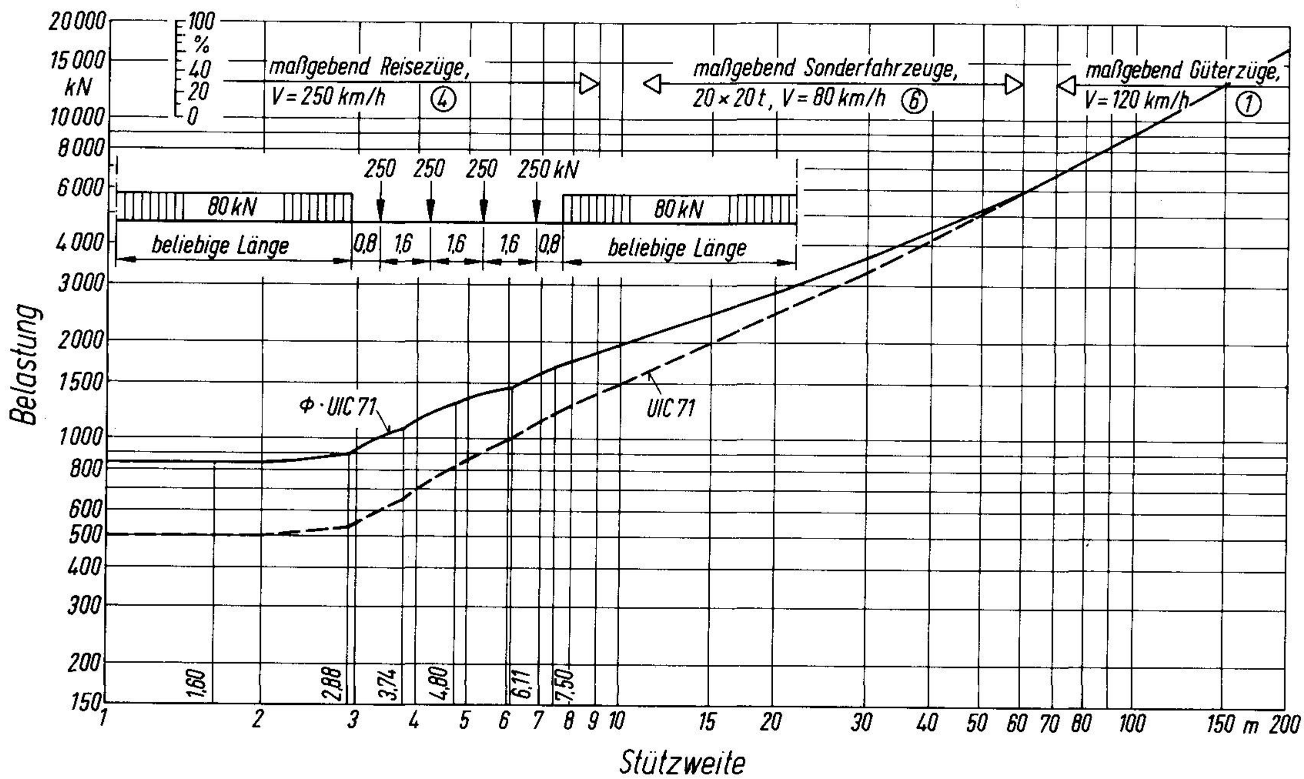


Bild 2: Belastungsbild UIC 71 und Biegemomente in Balken auf 2 Stützen in Abhängigkeit von der Stützweite.

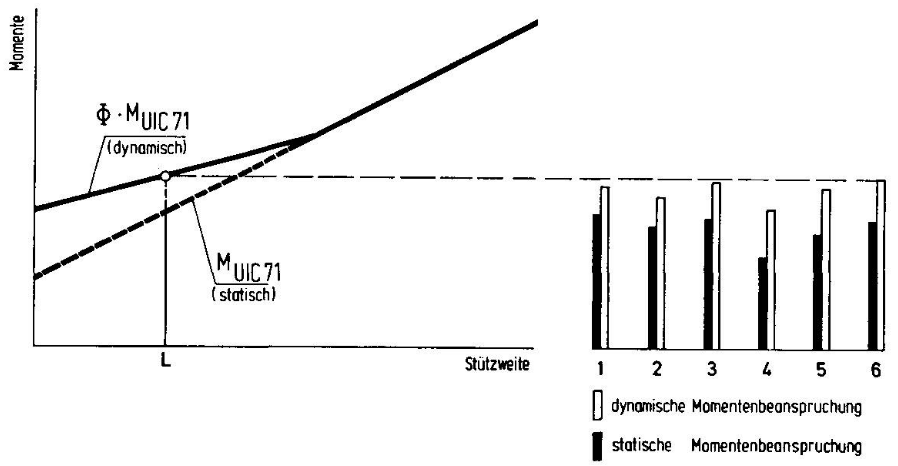


Bild 3: Verfahren zur Ermittlung der dynamischen Momentenbeanspruchung.

Dieses Vorgehen muß verglichen werden mit dem Abgehen von den an Rathäusern fixierten Klaftermaßen zum metrischen Maßsystem: Bei der Ausarbeitung des Belastungsbildes UIC 71 ist ein methodisch neuer Weg beschritten worden. Während sonst häufig der kleinste gemeinsame Nenner zu internationalen Vereinbarungen gemacht wird, von dem nur wenig harmonisierende Kraft ausgehen kann, wurde hier ein Maß definiert, in dem sich jede Verwaltung messen und mit dessen Maßzahlen Erfahrungen ausgetauscht und Entwicklungen gesteuert werden können.

3. HARMONISIERUNG DER ERMÜDUNGSBEMESSUNG

Der für die Harmonisierung der Eisenbahnlasten aufgezeigte Weg wurde für die Betriebsfestigkeitsbemessung erfolgreich fortgesetzt. Die analysierte Situation bei den Mitgliedsverwaltungen ergab

- völlig unterschiedliche Vorstellungen hinsichtlich der Ermüdung,

- nur wenige Beanspruchungsmessungen in Eisenbahnbrücken,

- offensichtliche Überschreitungen der Betriebsfestigkeit an bestimmten Stellen stählerner Eisenbahnbrücken,

- zahlreiche Versuchsergebnisse an verschiedenen Probeformen mit unterschiedlichem Auswertemuster und

- unterschiedliche Ausgangspositionen für Ermüdungstheorien.

Dieser Tatbestand war nicht geeignet, schnell zu einer allgemein anerkannten Harmonisierung zu führen. Ohne auf die tatsächliche Entwicklung einzugehen, soll jedoch das Wesentliche hervorgehoben werden. Die verwirrende Zahl oft widersprüchlicher Auffassungen und Erkenntnisse müßte gesiebt werden an ihrem Einfluß auf die Abmessungen im Eisenbahnbrückenbau und an der Möglichkeit zutreffender Prognosen. Nicht die größere Richtigkeit einer Theorie gegenüber einer anderen sollte für die Festlegung des Bemessungsverfahrens bestimmen, sondern die Fragen:

- Ergeben sich konstruktiv herstellbare Abmessungsdifferenzierungen und
- kann der Entwerfende über zutreffende Werte der Eingangsgrößen verfügen?

Mit dieser Zielsetzung wurde nicht versucht die international vorhandenen Ergebnisse von Betriebsfestigkeitsversuchen auf möglichst genaue, den Problemen entsprechende Wöhlerlinien festzulegen, sondern umgekehrt, es wurde versucht, ein Raster von Wöhlerlinien aufzustellen, in welches alle vorhandenen und alle zukünftigen Versuchsergebnisse so eingepaßt werden können, daß mit dieser Zuordnung die nötigen Abmessungen von Eisenbahnbrücken sinnvoll abgestuft werden können. Die Abstufung im logarithmischen Maßstab vorzunehmen ist naheliegend; gewählt wurde für den Spannungsmaßstab die Normzahlreihe R_{20}*). Während für den Bereich der Betriebsfestigkeit zahlreiche Arbeiten vorlagen, die zu harmonisieren waren, bestand auf der Seite der Lastannahmen große Leere und es mußten mehrere eigene Formulierungsversuche unternommen werden. Eine Studiengruppe des ORE hat hier sehr grundlegende Arbeit geleistet. Schlüsselerkenntnis für jeden entwerfenden Ingenieur muß es sein, daß die Beanspruchungsgröße nur in Abhängigkeit von dem Betriebsfestigkeitscharakter der zu bemessenden Stelle anzugeben ist und

*) Dezimal-geometrische Folge mit 20 Werten in einer Dekade:
$R_{20} = q \sqrt[20]{10}$

der von der Neigung und dem Abknickpunkt der Wöhlerlinie bestimmt wird (Bild 4). Um eine einfache Bemessungsregel zu haben ist eine für alle

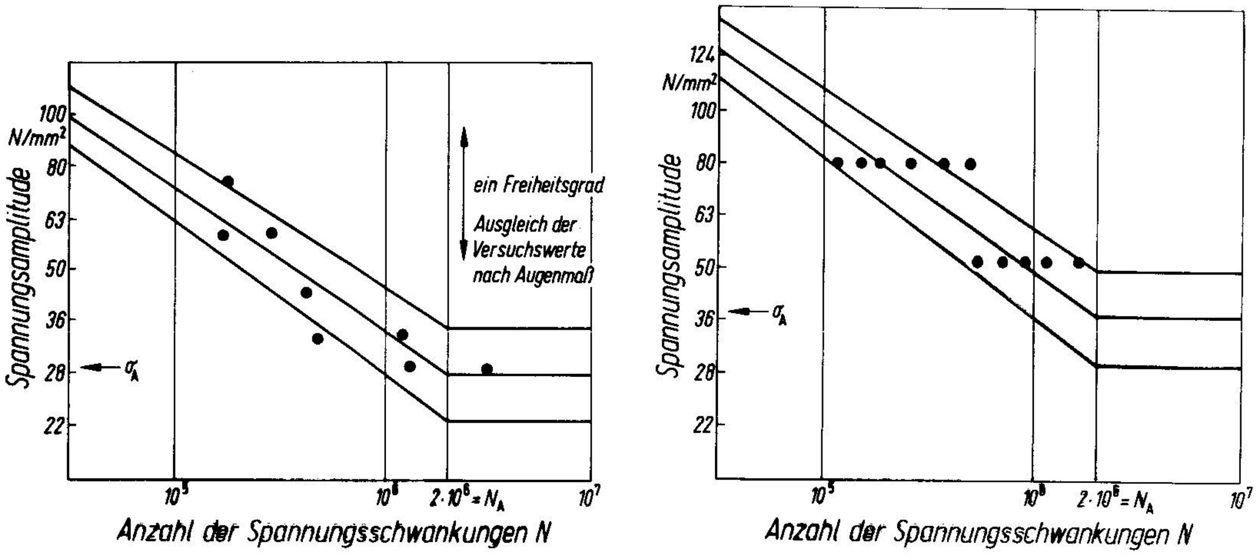


Bild 4: Auswertung von Versuchsergebnissen mit Hilfe der normierten Wöhlerlinie für Schweißverbindungen aus Baustahl.

Schweißverbindungen gleiche Neigung der Wöhlerlinien im doppeltlogarithmischen Netz und ein Grenzpunkt in Höhe einer gleichen Lastwechselzahl sinnvoll. Müssen für gelochtes oder ungestoßenes Material andere Werte angenommen werden, sind hierfür die Lastannahmen neu zu entwickeln. Das UIC-Merkblatt berücksichtigt zur Zeit nur Schweißverbindungen. Somit ist eine

einheitliche Lastannahme möglich. Der Verzicht auf eine Berücksichtigung der Stahlsorte als Eingangswert leitet sich für die zur Zeit gehandelten Baustähle als zwingend aus den Versuchsergebnissen ab.

Da für die Bemessungsregeln Wöhlerlinien aus Einstufenversuchen gewählt werden ist eine Methode anzugeben, wie die bei Zugüberfahrten schwankenden Beanspruchungshöhen den Kennwerten dieser Wöhlerlinien gegenübergestellt werden sollen. Da das Erschöpfen der Ermüdungsfestigkeit sich aus der Aufsummierung zahlreicher Einzelschädigungen ergibt, ist ein Verfahren festzulegen, nach dem die Größe der Einzelschädigungen festgelegt und aufsummiert wird. Man hat sich für die "rain-flow"-Zählmethode und die "Miner"-Regel entschieden. Dabei hat man auf die Berücksichtigung eines Mittelspannungseinflusses verzichtet. Dieser Einfluß ist umstritten und wird in Abhängigkeit vom Eigenspannungszustand gesehen. Seine konsequente Berücksichtigung würde eine modifizierte Zählmethode und eine iterative, von der gewählten Bemessungsgröße abhängige Beanspruchungsgröße erfordern. Die dadurch vielleicht zu erzielende größere Teilgenauigkeit an dieser Stelle im Bemessungsablauf liefert aber wegen der Schwankungsbreite der verfügbaren Eingangswerte und der grundsätzlichen Schwankungsbreite der Lebensdauervorhersage hinsichtlich der Abmessungen keine Aussage von Gewicht. Um nun die bemessungsrelevanten Einflüsse herauszufinden, wurde als Kriterium die Normzahlreihe R_{20} festgelegt. Einflüsse, deren Auswirkungen in dieser Reihe nicht unterscheidbar sind, bleiben unberücksichtigt.

Um die Summierung der schädigenden Lasteinflüsse für das Bemessungsverfahren zu normieren wurden eine Reihe von Verkehren hinsichtlich der Zugzahlen je Jahr, der Zuggattungen (Personen-, Güter-, Ganz-, Leerzüge), der beförderten Massen und der Achslasten international untersucht. Da es sich um eine Integration zahlreicher Einzeleinflüsse handelt, zeigte sich eine relative Unabhängigkeit von nationalen Besonderheiten. Um bei der Einzelbemessung die Einflußparameter mit möglichst geringer Fehlerbreite zu berücksichtigen, ist ein Ausgangswert zu suchen, der sich im Zentrum der möglichen Bandbreite befindet. Für die nach heutiger Erkenntnis zu berücksichtigenden Einflüsse ergeben sich danach folgende Werte:

- Streckenleistung 120 Züge je Tag und Gleis mit 22 Millionen Leistungstonnen je Tag und Gleis,
- Stützweitenbereich 7,1 m bis 10 m und
- Eingleisiges Bauwerk

Die sich mit diesen Werten in 50 Jahren aufsummierende Schädigung ergibt im doppelt-logarithmischen Netz eine Schädigungslinie parallel zur Betriebsfestigkeitslinie, die durch ein Wertepaar "Spannungswert der Doppelamplitude" und "Lastwechselzahl" zu kennzeichnen ist. Es erweist sich dabei als zweckmäßig, die Größe des Spannungswechsels in λ_T als Relativwert zum Beanspruchungswert aus der statischen Verkehrsbelastung (Belastungsbild UIC 71 mit Schwingbeiwert Φ) anzugeben. Nimmt man hier als zentralen Ausgangswert $\lambda_T = 0,5$ an (dieser Wert wurde bei Extremwertmessungen als überwiegend schädigende Betriebsbeanspruchung erkannt), ergibt sich als zugehörige Lastwechselzahl $N = 2 \times 10^6$. Mit anderen Worten: Die Schädigungssumme eines "Normalverkehrs in einem Zeitraum von 50 Jahren" entspricht bei einem eingleisigen 7,1 m bis 10 m weit gespannten Überbau einem einstufigen Lastwechselkollektiv mit $N = 2 \times 10^6$ Lastwechseln und einer Doppelspannungsamplitude von $\lambda_T = 0,5$.

Der bei einer Bemessung unter Berücksichtigung der Betriebsfestigkeit einzuhaltende Sicherheitsabstand wurde zu $\nu = 2,5$ gewählt. Hierbei stand der Gedanke Pate, daß für ein Bauwerk mit einer geplanten Nutzungszeit von 100 Jahren erst in der zweiten Hälfte dieser Zeit die operative Versagenswahrscheinlichkeit aus Ermüdung diejenige aus dem Versagen einer Grenzbeanspruchung überschreiten soll.

Mit diesen Festlegungen schrumpft die Betriebsfestigkeitsbemessung auf eine Entscheidungsfindung in einem Normzahlnetz zusammen. Das Auffinden der zulässigen Spannungsdoppelamplitude aus dem Belastungsbild UIC 71 geschieht mit Hilfe eines Abszissenwertes m, der sich aus den Anteilen aller Parametereinflüsse zusammensetzt.

$$m = m_o + \Delta m_l + \Delta m_a + \Delta m_b + \ldots + \Delta m_i$$

Dem Ausgangswert m_o, der sich auf die schon angesprochenen Zentralwerte in der Bandbreite der Einflußparameter bezieht, ist der Wert - 2 zugeordnet. Der Einfluß der Stützweite Δm_l, der Streckenbelastung Δm_t und der Mehrgleisigkeit Δm_a bzw. Δm_b werden einem einfachen Tafelwerk entnommen (Bild 5). Der sich ergebende ganzzahlige m-Wert legt in dem schon angesprochenen Normzahlennetz (Bild 6) die zulässigen Schweißverbindungstypen in Abhängigkeit von der aus der statischen Berechnung bekannten Doppelspannungsamplitude aus der Beanspruchung entsprechend Belastungsbild UIC 71 fest.

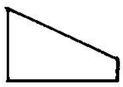
$\Delta\ m_l$ - Stützweiteneinfluss des betrachteten Bauteils.

Stützweite L [m]	< 3,6	3,6 ÷ 4,5	4,5÷5,6	5,6 ÷7,1	7,1 ÷ 10,0	10,0 ÷14,0	14 ÷ 28	> 28
$\Delta\ m_l$	.4	.3	.2	.1	± 0	+ 1	+ 2	+ 3

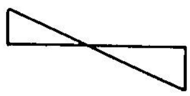
$\Delta\ m_t$ - Einfluss der Streckenbelastung

Streckenbelastung [Mio. Lt/Jahr]	< 12	12 ÷ 18	18÷28	28 ÷45	< 45
$\Delta\ m_t$	+ 2	+ 1	± 0	− 1	− 2

$\Delta\ m_a$ - Mehrgleisiger Überbau, addierende Wirkung

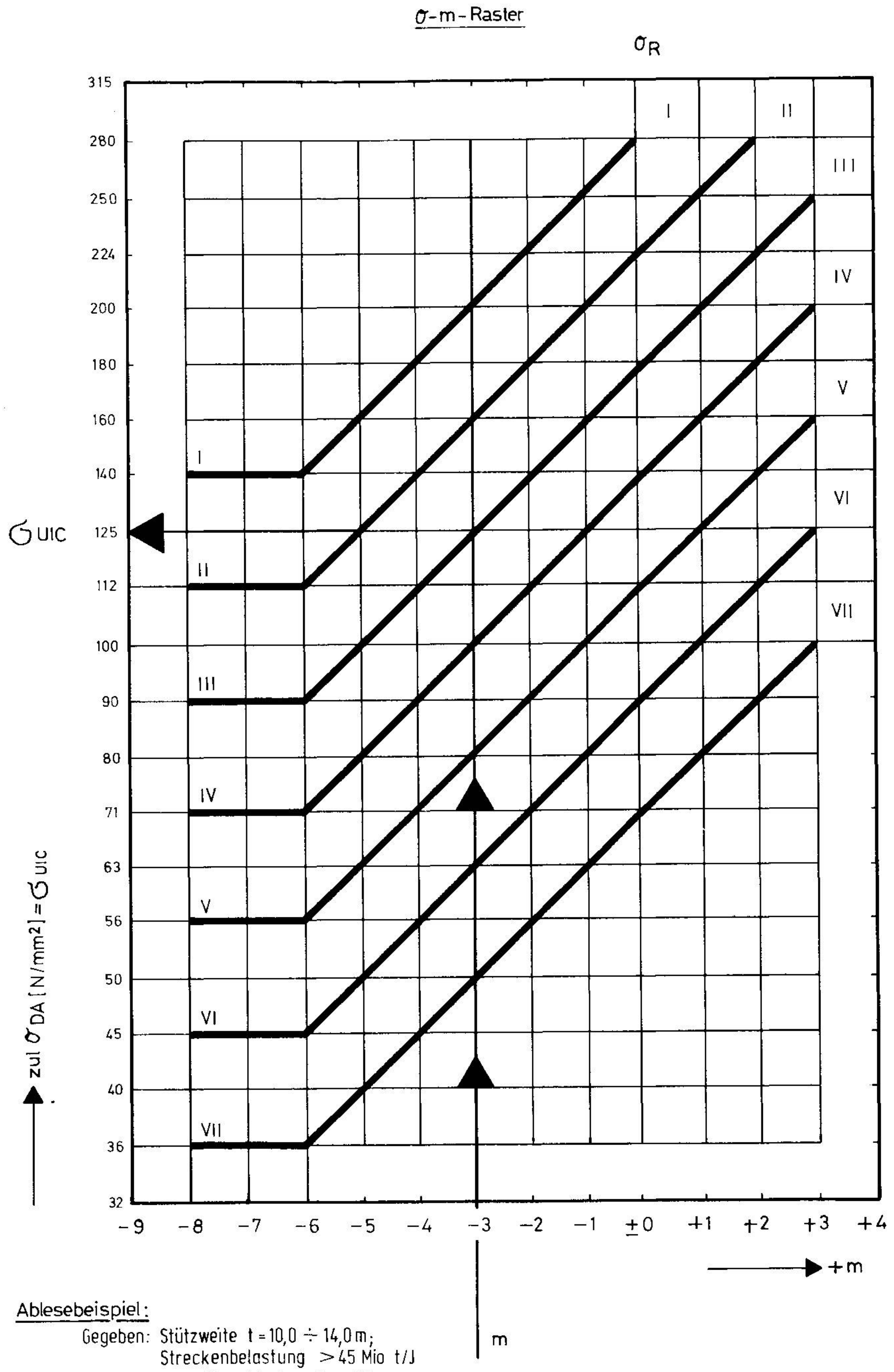
P1  P2 $a = \frac{P_1}{P_1 + P_2}$	0,5 ÷ 0,6	0,6 ÷ 0,75	0,75÷ 0,9	0,9 ÷ 1,0
$\Delta\ m_a$	+ 3	+ 2	+ 1	± 0

$\Delta\ m_b$ - Mehrgleisiger Überbau, substrahierende Wirkung

P 1  P 2 $b = P_2 / P_1$	0 ÷ 0,71	0,71÷ 1,0
$\Delta\ m_b$	± 0	− 1

b = Einwirkung des zweiten Gleises p_2 im Verhältnis zur Einwirkung des ersten Gleises p_1, wenn die beiden Einwirkungen entgegengesetzte Vorzeichen haben.

Bild 5: Werte Δm_l, Δm_t, Δm_a, Δm_b.



<u>Ablesebeispiel:</u>

Gegeben: Stützweite $l = 10{,}0 \div 14{,}0$ m;
Streckenbelastung > 45 Mio t/J
Schweißverbindungstyp III

Damit wird $m = m_o + \Delta m_L + \Delta m_t = -2 + 1 - 2 = -3$;

Daraus folgt $\sigma_{UIC} = 125$ N/mm²

Bild 6: Normzahlennetz

4. EIN ZUKUNFTSWEISENDES BEMESSUNGSVERFAHREN

Das hier aufgezeigte Vorgehen des UIC-Merkblattes 778 kann als zukunftsweisend bezeichnet werden; das Verfahren

- berücksichtigt durch die Normzahlrasterung nur wirklich bemessungsrelevante Einflüsse,
- läßt Raum für weitere Einflüsse, die sich als bemessungsrelevant erweisen,
- ermöglicht die Berücksichtigung abweichender Auffassungen (in einer Tabelle ist z. B. dargestellt, wie sich andere Sicherheitsabstände auswirken),
- ist ausbaufähig, um etwa andere Neigungen der Wöhlerlinie als $k = -3{,}75$ zu berücksichtigen und
- ist außerordentlich praxisgerecht, denn es läßt Unsicherheiten in den Annahmen der Einflußparameter wirkungsgleich beurteilen. Es läßt mit einem Blick erkennen, ob eine feinere Berechnung Wirkung haben kann und wie groß etwa die Parameterverschiebung sein müßte, um die vorgesehene Konstruktion als ausreichend anzusehen.

Alle diese Überlegungen erfordern von Ingenieuren eine vom üblichen Vorgehen abweichende, entscheidungsfreudige Einstellung zu seinen Bemessungsproblemen. Mit einem Paar aus ganzen Zahlen über die Eignung einer Konstruktion zu entscheiden, ist schon etwas Neues, Großartiges. Die Deutsche Bundesbahn (DB), die an der Aufstellung des UIC-Merkblattes 778 federführend mitwirkte, wird dieses Bemessungsverfahren im Rahmen der Fortentwicklung der "Vorschrift für Eisenbahnbrücken und sonstige Ingenieurbauwerke (VEI)" - DS 804 der DB -, zu geeigneter Zeit im Rahmen ihrer Zuständigkeit einführen. Dies bedarf noch einer gewissen Abstimmung im nationalen Bereich, Aufklärung für die eigenen Ingenieure, Prüfingenieure sowie ausführenden Firmen und einer Umgewöhnung vom traditionellen Anschreiben zulässiger Spannungen zu zulässigen Verbindungstypen hin.

Ein erster Schritt in diese Richtung wurde mit der Herausgabe einer Übergangsvorschrift als Vorausgabe der DS 804 getan. Diese ersetzt die jahrzentelang gültige Berechnungsvorschrift BE - DV 804 der DB und die "Vorschrift für geschweißte Eisenbahnbrücken" DV 848 der DB, enthält allerdings noch das κ-Verfahren

Der Wirklichkeit ist man mit diesen Modifikationen jedoch nicht näher gekommen. Dies ergibt sich allein aus der Streubreite, mit der Ermüdungsversagen überhaupt vorhergesagt werden kann. Es muß hinsichtlich der Dauerhaftigkeit eine log-normale Verteilung mit einem Variationskoeffizienten $v_x = 0{,}8$ angenommen werden. Dies bedeutet, daß ein Bauwerk, dem eine mittlere Nutzungszeit von 10 Jahren vorhergesagt wird, mit gleicher Wahrscheinlichkeit nach 2,5 Jahren zu Bruch gehen oder noch 40 Jahre halten kann. Diese Aussage ist in Entscheidungsprozessen außerordentlich weich. Damit wird die Notwendigkeit zu einem zurückhaltenden Umgang mit den Zahlenwerten in Betriebsfestigkeitsbemessungsregeln deutlich. Konstruktive Regeln und stetige Überwachung müssen hinzukommen, um Bauwerksicherheit zu gewährleisten. Konstruktive Regeln lassen sich am leichtesten gewinnen, wenn Konstruktionen mit einem gleichen Maßstab gemessen werden können, Maßstäbe sind Konvention. Das UIC-Merkblatt ist eine Konvention mit dem Anspruch Maßstab zu sein.

Fatigue Assessment According to Eurocode 3 (Steel Structures)

Vérification à la fatigue selon l'Eurocode 3 (constructions métalliques)

Betriebsfestigkeitsnachweis für Stahlbauten nach Eurocode 3

G. SEDLACEK
Professor
Rheinisch-Westfälische Technische Hochschule
Aachen, BRD

SUMMARY
The fatigue assessment of steel structures according to the limit state equation of Eurocode 3 is defined in terms of service life (number of cycles). The safety factors are derived from the "Level II" method of constant sensitivity factors.

RESUME
Les états limites considérés dans l'Eurocode 3 pour la vérification de la sécurité à la fatigue des constructions métalliques sont définis sur l'axe des durées de vie. Les facteurs de sécurité à utiliser pour la vérification sont déterminés selon la procédure issue des théories de "Level II" et faisant intervenir des facteurs de sensibilités constants.

ZUSAMMENFASSUNG
Für den Betriebssicherheitsnachweis für Stahlbauten nach Eurocode 3 werden der Grenzzustand in der Achse der Lebensdauer definiert und nach dem aus der Level II-Methode abgeleiteten Verfahren der konstanten Wichtungsfaktoren die Sicherheitsfaktoren für den Nachweis abgeleitet.

1. GENERAL

The calculative safety assessment for the fatigue behaviour of dynamically loaded steel structures should be carried out in a way to attain as often as possible a target reliability expressed by the safety index β_T, not falling below a minimum value min β , if possible and also not exceeding a maximum value max β for economic reasons.

The justification for a proposal for the determination of the safety elements is developed in the following using the procedure of global sensitivity factors which is derived from the Level II method /3/.

2. ASSESSMENT FOR A DETERMINED $\Delta\sigma$-LEVEL AND A DETERMINED NUMBER OF CYCLES n

Fig. 1 demonstrates a scatter-band of test results from fatigue tests with a component with a critical detail for different levels of damaging stress ranges $\Delta\sigma$, which are calculated as nominal stresses for measured test loads.

As a resistance model the S-N-lines for defined survival probabilities can be illustrated in double logarithmic scale as for instance fig. 2.

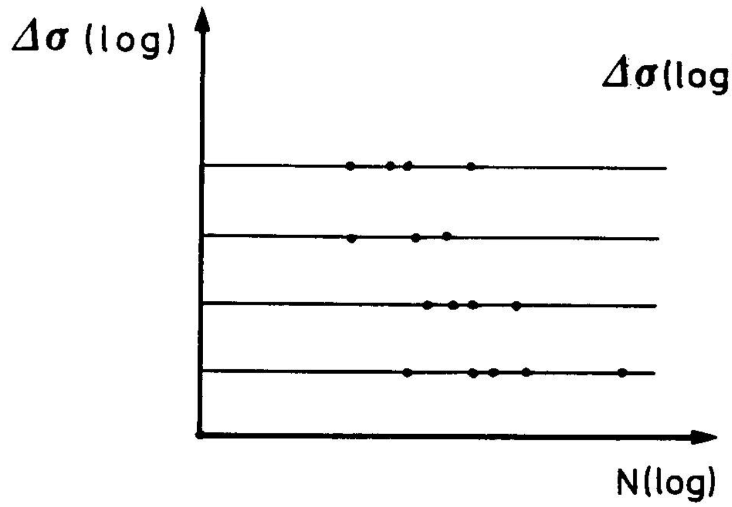


Fig. 1: Scatter-band of fatique test results

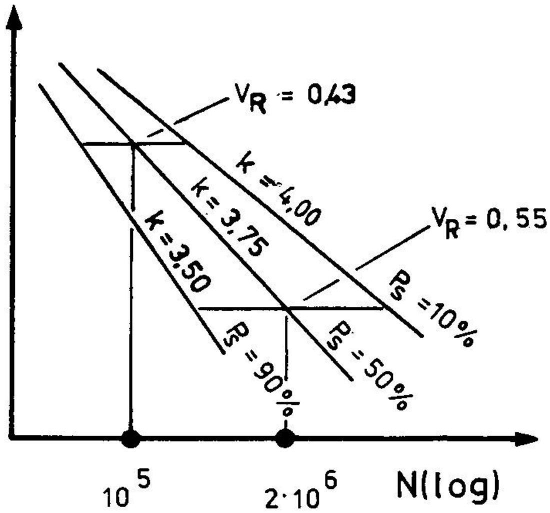


Fig. 2: S-N-lines for small test specimens

The safety assessment should be carried out for a structural component designed in the same way as the test elements.

The limit state equation for fatigue for the stress range level $\Delta\sigma_i$ is then

$$g\ (X_i) = N_i - n_i = 0 \tag{1}$$

see fig. 3.

Here a log-normal distribution with the variation coefficient V_R - for instance for small components according to fig. 4 - is assumed for the basic variable for the "resistance" N_i, and for the "action" n_i the sum of the cycles of all measuring time intervals is extrapolated over a designed service life and defined as,fig. 5

$$n_i = \sum_{T_L} n_t \quad . \tag{2}$$

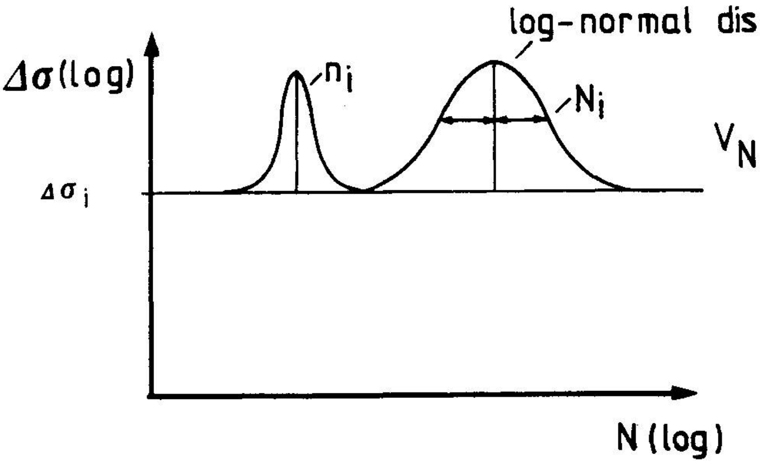


Fig. 3: Distributions of N and n

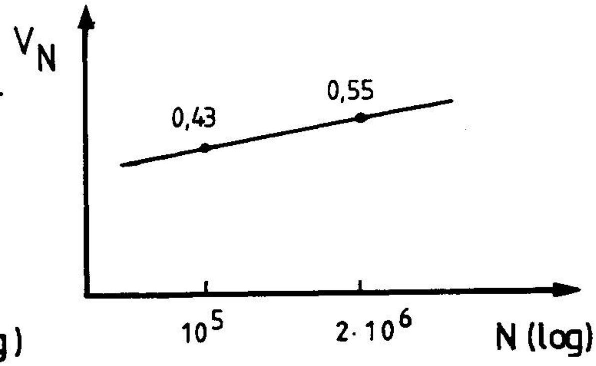


Fig. 4: Variation coefficient (N-log normal)

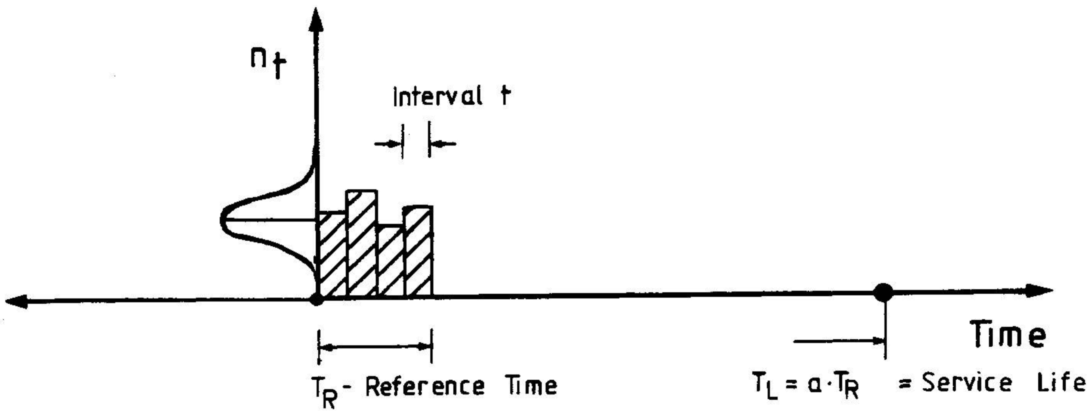


Fig. 5: Number of stress range cycles

If a symmetrical density distribution of the numbers of cycles is assumed over the measuring time intervals within service life, then the variation coefficient for n_i is zero.

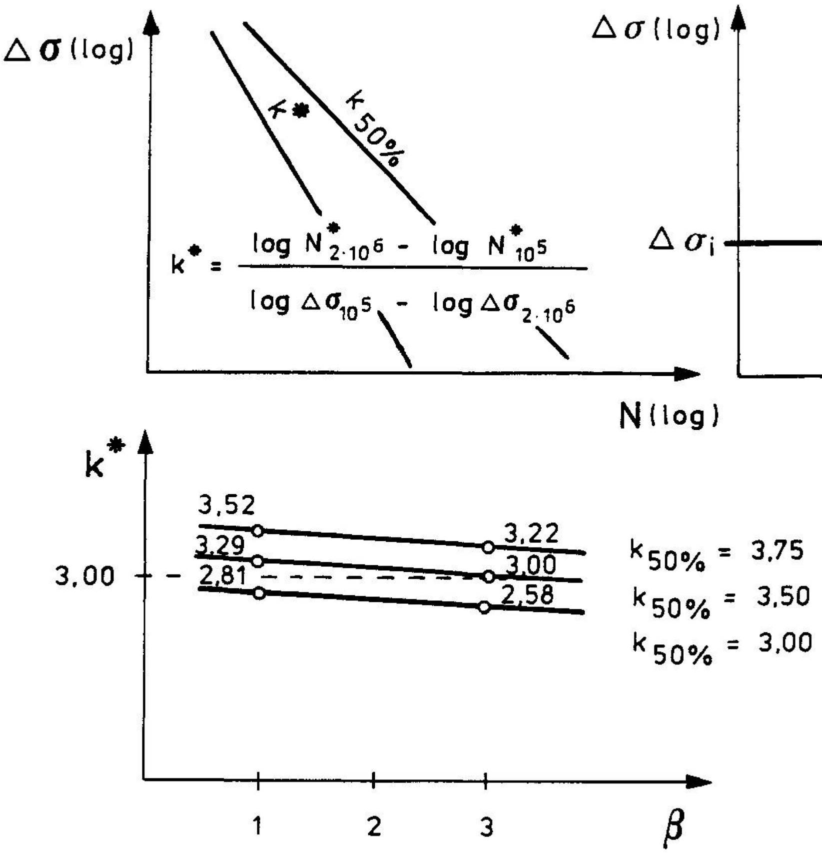
The design values N_i^* and n_i^* can be expressed according to /4/

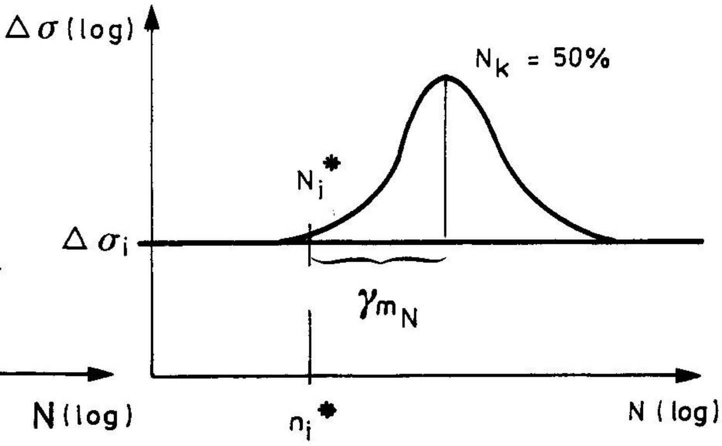
$$\left.\begin{aligned} N_i^* &= m_N \exp\left(-\alpha_R \beta V_R - \tfrac{1}{2} V_R^2\right) && \text{with } \alpha_R = 1{,}0 \\ n_i^* &= m_n \left(1 + \alpha_S \beta V_S\right) = m_n && \text{due to } \alpha_S = 0 \end{aligned}\right\} \tag{3}$$

The design values N_i^* for various stress levels $\Delta\sigma_i$ will then lie on the design S-N-curve with the slope k* which is influenced by β, the assumed variation coefficient and by the slope of the 50 %-S-N-line, see fig. 6.

For target values β_T of the order of 2 the slope is k* = 3,0. This value is also obtained in tests with full scale structural components for which the variation coefficients are smaller and the scatter-bands are more parallel as compared with the test results obtained with small test-specimens.

The safety verification for a deterministically given stress range can now be carried out in the time scale with γ_{mN}, with reference to the characteristic values N_k (for example $N_k = N_{50\ \%}$) according to fig. 7.



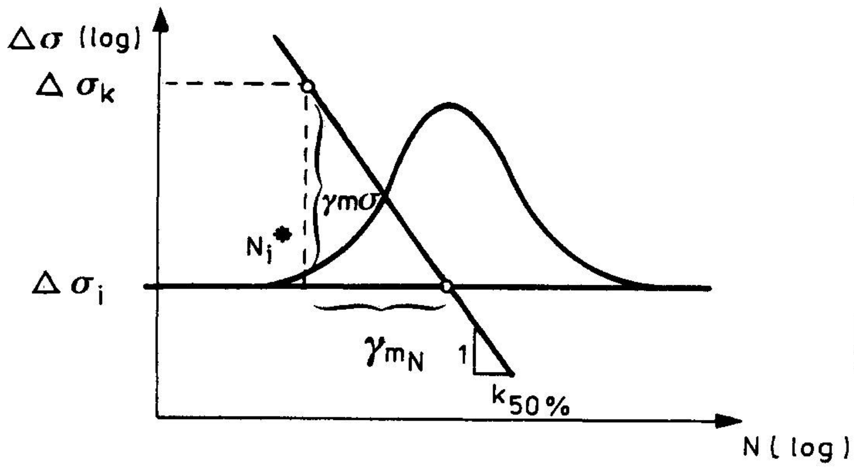


$$\gamma_{mN} = \frac{N_k}{N_i^*}$$

Fig. 6: Design values for the slope k*.

Fig. 7: Safety factor in N-scale.

An equivalent verification with respect to the stress range scale is performed with the slope k of the characteristic S-N-line according to fig. 8.



$$\gamma_{m\sigma} = \sqrt[k]{\gamma_{mN}}$$

$$\frac{\log N_k - \log N_i^*}{\log \Delta\sigma_k - \log \Delta\sigma_i} = \frac{\log \gamma_{mN}}{\log \gamma_{m\sigma}} = k$$

Fig. 8: Safety factor in $\Delta\sigma$-scale.

3. ASSESSMENT FOR SPECTRA WITH VARIOUS DETERMINED $\Delta\sigma_i$-LEVELS AND VARIOUS DETERMINED NUMBERS OF CYCLES n_i

If according to fig. 9 the stress ranges $\Delta\sigma_i$ are acting at different levels, for example $\Delta\sigma_i$ with n_i and $\Delta\sigma_A$ with n_A, the equivalent-damage stress range cycles n_e at the reference level $\Delta\sigma_A$ can be expressed using Miner's rule, fig. 10.

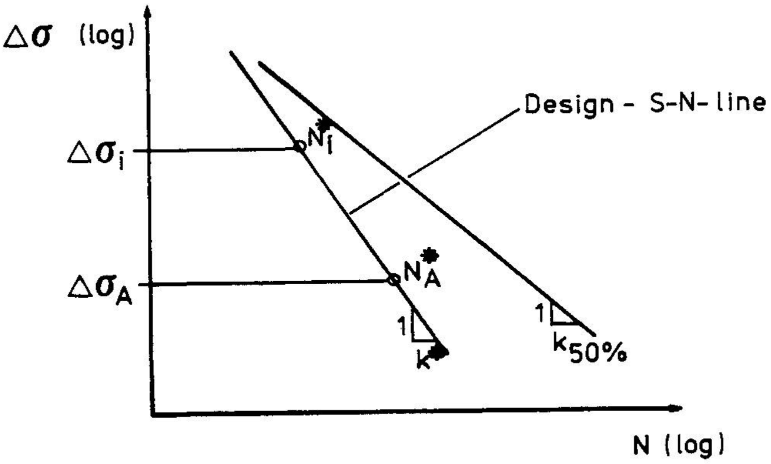


Fig. 9: Design-S-N-line.

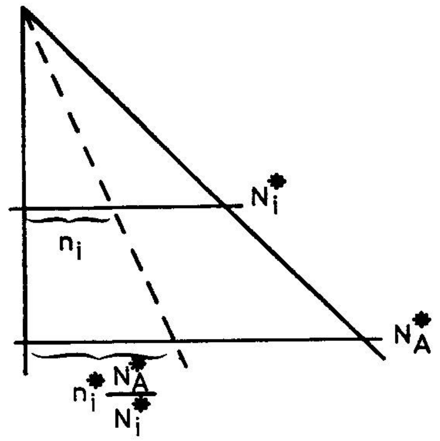


Fig. 10: Equivalent-damage stress cycles on different levels of stress range.

$$n_e^* = \sum n_i^* \frac{N_A^*}{N_i^*} \tag{4}$$

Considering the equation of the S-N*-line

$$\Delta\sigma_i^{k^*} \cdot N_i^* = \Delta\sigma_A^{k^*} \cdot N_A^* \tag{5}$$

it follows from (4)

$$n_e^* = \sum n_i^* \left(\frac{\Delta\sigma_i}{\Delta\sigma_A}\right)^{k^*} \tag{6}$$

The design-equation is then

$$g(X_i^*) = N_A^* - n_e^* = 0 \tag{7}$$

or using characteristic values and safety factors, fig. 11

$$\frac{N_{Ak}}{\gamma_{mNA}} - n_e = 0 \tag{8}$$

By defining the reference level $\Delta\sigma_A$ as the level of the equivalent-damage stress range $\Delta\sigma_e$ for the number of load cycles $\sum n_i$, fig. 12.

$$n_e^* = \sum n_i^* \left(\frac{\Delta\sigma_i}{\Delta\sigma_e}\right)^k = \sum n_i \tag{9}$$

the equivalent-damage stress range.

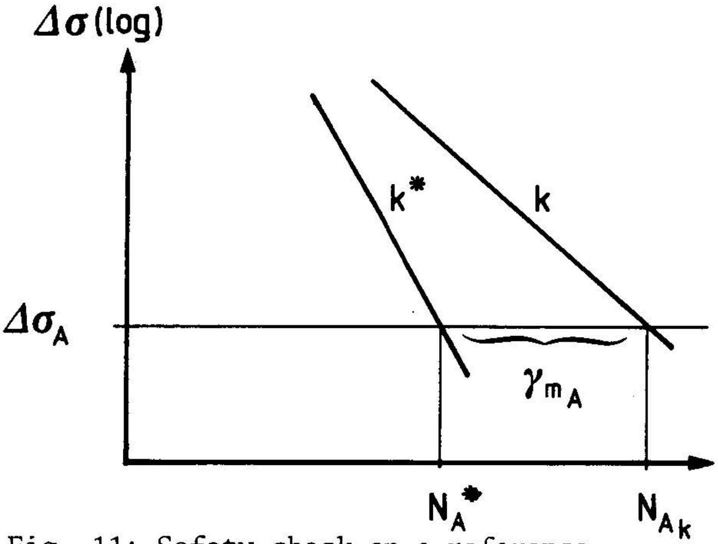


Fig. 11: Safety check on a reference stress range level $\Delta\sigma_A$.

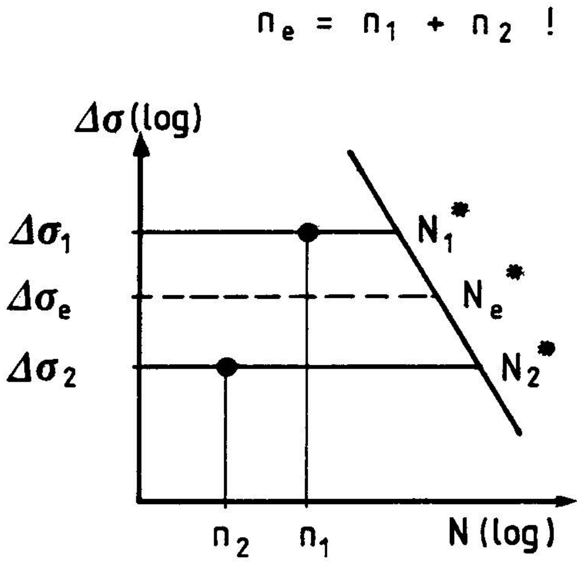


Fig. 12: Safety check on the equivalent-damage stress range level $\Delta\sigma_e$.

$$\Delta\sigma_e = \left(\frac{\sum n_i \Delta\sigma_i^{k^*}}{\sum n_i}\right)^{\frac{1}{k^*}} \tag{10}$$

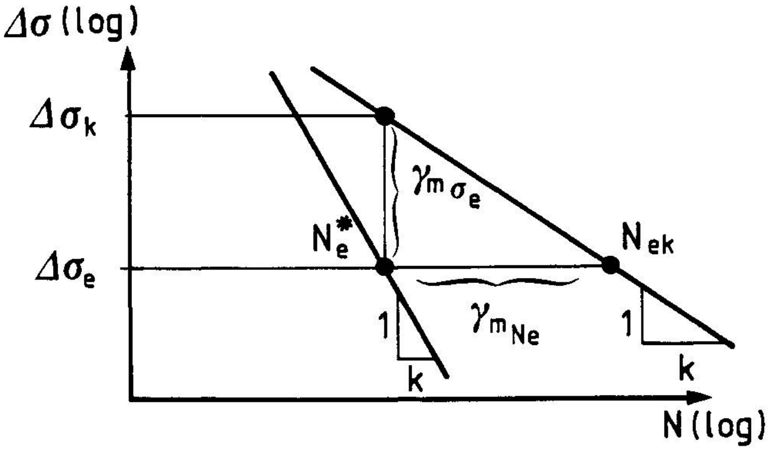
follows and the verification is performed by

$$N_e^* - \sum n_i = 0 \tag{11}$$

The equivalent verification in terms of stresses ca be carried out using $\gamma_{m\sigma}$ according to fig. 13.

In fig. 14 γ_{m_N} - values and $\gamma_{m\sigma}$ - values related to the 50 %-S-N-lines given as characteristic values are specified for different slopes and β-values.

For steel structures β = 2,0 is considered to be sufficient, if the appropriate values for $\Delta\sigma_i$ and n_i are correct.



$$\boxed{\frac{\Delta \sigma_k}{\gamma_{m\sigma}} - \Delta \sigma_e = 0}$$

Fig. 13: Safety check in $\Delta\sigma$ scale with equivalent-damage stress range $\Delta \sigma_e$.

$\gamma_{m\sigma}$ - Values

$k_{50\%}$ \ β		1,0	1,5	2,0	2,5	3,0
3,75	N_{10^5}	1,12	1,19	1,26	1,33	1,41
	$N_{2\cdot10^6}$	1,16	1,25	1,34	1,44	1,55
3,50	N_{10^5}	1,13	1,20	1,28	1,36	1,45
	$N_{2\cdot10^6}$	1,17	1,27	1,37	1,48	1,60
3,00	N_{10^5}	1,16	1,24	1,33	1,43	1,53
	$N_{2\cdot10^6}$	1,20	1,32	1,44	1,58	1,73

γ_{mN} - Values

$k_{50\%}$ \ β		1,0	1,5	2,0	2,5	3,0
3,75	N_{10^5}	1,54	1,91	2,36	2,93	3,63
	$N_{2\cdot10^6}$	1,73	2,28	3,00	3,95	5,21
3,50	N_{10^5} / $N_{2\cdot10^6}$	"	"	"	"	"
3,00	N_{10^5} / $N_{2\cdot10^6}$	"	"	"	"	"

Fig. 14: γ_m-values for safety checks in N-scale or $\Delta\sigma$-scale using 50 % values as characteristic values

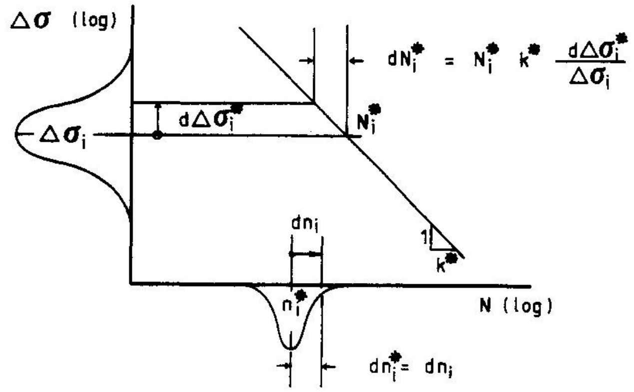
4. ASSESSMENT WITH VARYING $\Delta\sigma_i$-LEVEL AND VARYING NUMBERS OF CYCLES n_i

Assuming that the values of $\Delta\sigma_i$ and n_i are only correct in the mean of all design cases and that they vary with $d\Delta\sigma_i$ and dn_i around the correct mean value with the variation coefficients 0,10 (which corresponds to the variance of specified dead loads), the sensitivity factors α_i for all variables can be derived from the characteristic equation in fig. 15 using

$$\alpha_i = \frac{\frac{\partial g}{\partial x_i} \; s_i}{\sqrt{\sum_j \left(\frac{\partial g}{\partial x_j} \cdot s_j\right)^2}} \qquad (12)$$

These sensitivity factors can be combined into $\alpha_R = 1,0$ for the "resistance model" and $\alpha_S = -0,30$ for the "acting model".

The γ_m-values and γ_{SYS}-values for verifications in terms of time or stresses using the 50 %-S-N-lines as characteristic values are illustrated in fig. 16.



<u>characteristic equation</u> :

$$N_i^* - N_i^* \, k^* \, \frac{d\Delta\sigma_i^*}{\Delta\sigma_i} - n^* - dn^* = 0$$

<u>Estimated datas</u> : $N_{50\%} = 2 \cdot 10^6$; $V_N = 0{,}55$; $\sigma_N = 1{,}1 \cdot 10^6$

$d\Delta\sigma_{50\%} = 0$; $\sigma_{\Delta\sigma} = 0{,}10 \, \Delta\sigma_i$

$n_{i50\%} = n_i$; $V_n = 0$; $\sigma_n = 0$

$dn_{i50\%} = 0$; $\sigma_{dn} = 0{,}10 \, n_i$

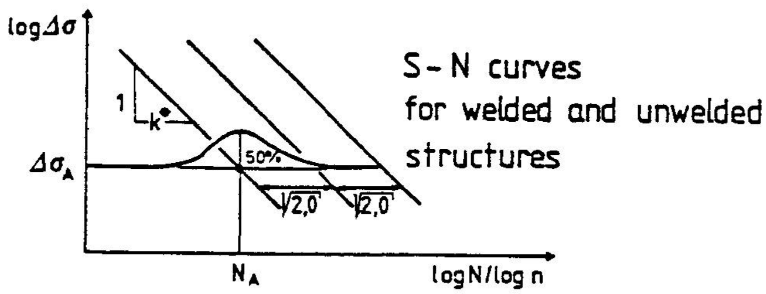
<u>Sensivity - factors</u> :

$\alpha_N \sim 1{,}0$; $\alpha_{\Delta\sigma} \sim -0{,}30$; $\alpha_{ni} \sim 0$; $\alpha_{dn_i} \sim -0{,}10$

<u>Overall Sensivity factors</u> :

$\alpha_R \sim 1{,}0$; $\alpha_S \sim -0{,}30$

Fig. 15:Consideration of a scatter for $\Delta\sigma_i$ and n_i.



$$\frac{N_A}{\gamma_{m_N} \cdot \gamma_{sys_N}} \geq n_e = \Sigma n_j \left(\frac{\Delta\sigma_j}{\Delta\sigma_A}\right)^{K^*}$$

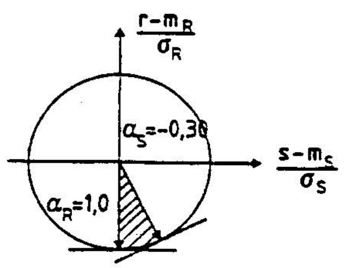
For $K^* = 3{,}0$; $V_R = 0{,}55$; $V_S = 0{,}10$

β	1,5	2,0	2,5	3,0
γ_{m_N}	2,30	3,00	4,00	5,20
γ_{sys_N}	1,15	1,20	1,25	1,30
γ_{Glob_N}	2,65	3,60	5,00	6,80

$$N_R^* \geq n_S^*$$

$$N_A \exp(-\alpha_R \beta V_R - 0{,}5 \, V_R^2) \geq \sum_j n_j \left(\frac{\Delta\sigma_j \, (1 - \alpha_S \, \beta \, V_S)}{\Delta\sigma_A}\right)^{K^*}$$

$$\frac{\Delta\sigma_K}{\gamma_{m\sigma} \cdot \gamma_{sys\sigma}} \geq \Delta\sigma_e = \left[\frac{\Sigma n_j \, \Delta\sigma_j^{K^*}}{\Sigma n_j}\right]^{1/K^*}$$



$V_R \geq 0{,}50$

$V_S \leq 0{,}10$

β	1,5	2,0	2,5	3,0
$\gamma_{m\sigma}$	1,30	1,45	1,60	1,75
$\gamma_{sys\sigma}$	1,05	1,06	1,08	1,09
$\gamma_{Glob\sigma}$	1,40	1,55	1,75	1,90

Fig. 16: Safety check for fatigue taking into account of a scatter for $\Delta\sigma_i$ and n_i.

In the Euorcode 3 the S-N-lines have been defined as (m - 2s)-values so that γ_m, may be defined, $\gamma_m = 1{,}0$.

5. MEAN STRESS INFLUENCE

The mean stress influence in Eurocode 3 is generally considered to be negligible; if the mean stress influence is significant, for instance for small welded elements or stress relieved components with minor residual stresses, the verification according to Eurocode 3 is on the safe side. In addition the mean stress influence can be considered by taking into account a bonus factor for the stress-ranges, which depends on the mean stress according to fig. 17.

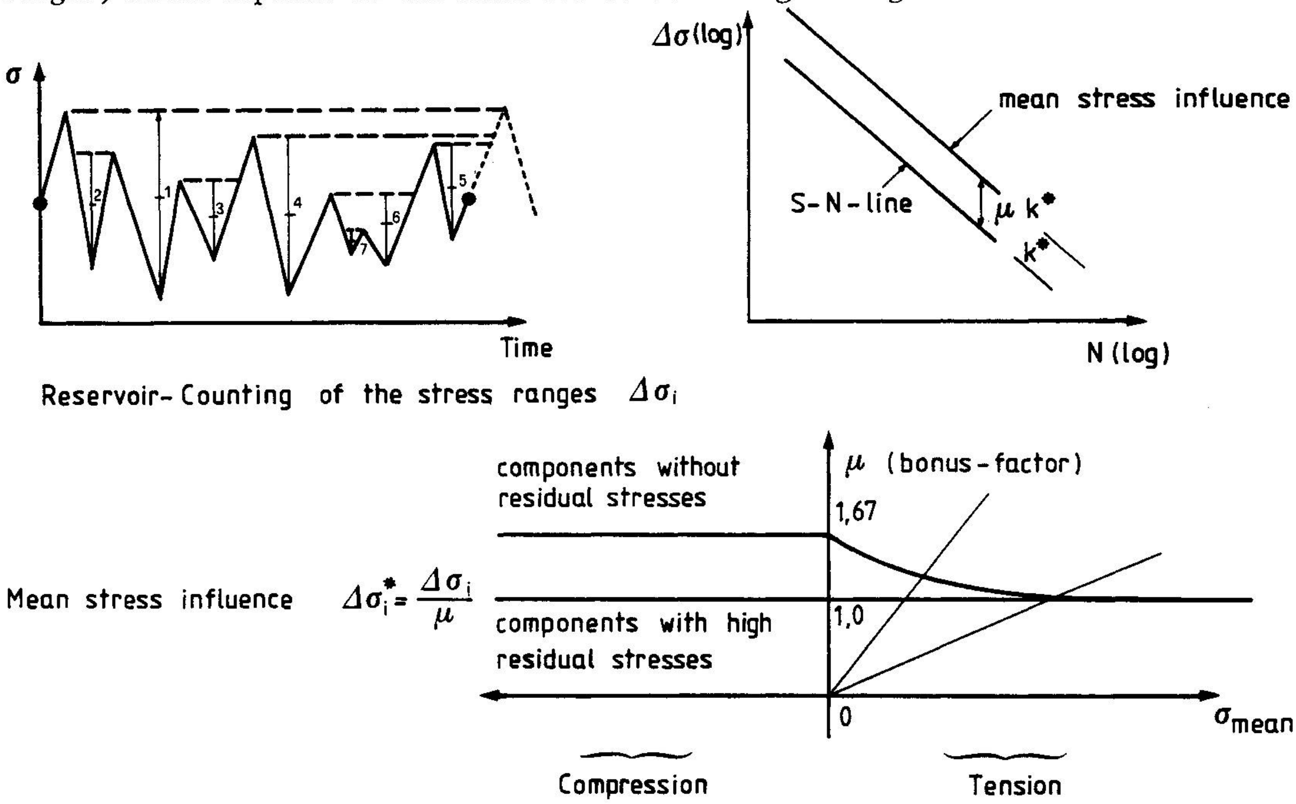


Fig. 17: Bonus factor for mean-stress-influence.

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Basis for Fatigue Design of Aluminium

Bases pour le dimensionnement à la fatigue des constructions en aluminium

Grundlagen für Berechnungen von Aluminium bei Ermüdungsbeanspruchung

D. KOSTEAS
Prof. Dr. -Ing.
Technische Universität München
München, BRD

SUMMARY
Fatigue design by empirical rules is no longer accepted as being efficient or safe for present day applications. However, at an international level, a unified, widely applicable and accepted approach is only just beginning to take shape. Elements and concepts of the new ECCS draft for fatigue design in aluminium are presented in this paper.

RESUME
Le dimensionnement à la fatigue basé sur des règles empiriques n'est plus acceptable, car il n'est ni économique ni du côté de la sécurité. Un procédé unifié, largement applicable et admissible, commence à prendre forme, même sur le plan international. Les éléments et concepts du nouveau projet CECM pour le dimensionnement à la fatigue des constructions en aluminium sont présentés.

ZUSAMMENFASSUNG
Die Dimensionierung schwingbeanspruchter Bauteile auf der Grundlage einer empirischen Anpassung ist keine akzeptable Methode für neue Entwicklungen, da sie weder leistungsfähig noch sicher ist. Ein einheitliches Vorgehen, breit anwendbar und annehmbar, beginnt erst jetzt feste Form anzunehmen, nun auch auf internationaler Ebene. Bestandteile und Hilfsmittel eines neuen EKS Entwurfs für schwingbeanspruchte Aluminiumkonstruktionen werden dargestellt.

1. INTRODUCTION

Our efforts to develop specifications for the design of aluminium structures in the last thirty years have been characterized, naturally, by the fact of a fluctuation between theoretical and experimental research, drafting recommendations and experience in practice.

Viewing the special problem of specifications for the fatigue design of aluminium structures we will recognize that in respect to research after a period of reconnaissance in the sixties and early seventies - i. e. small specimen testing, fundamentals of test planning and data evaluation - as well as a period of consolidation of our knowledge in the seventies - with more specimen testing filling in the blanks and statistical reevaluation of test data - we have now entered in the eighties a period of full-scale component fatigue behavior testing and application of knowledge to specifications, the latter for the first time on an international scale [1].

In respect to codes it seems that about every decade new efforts are undertaken to define in specifications the design of aluminium structures in fatigue. A review of several national proposals is given in [2] along with a brief analysis and comparison of relevant elements [3]. Two years ago work was initiated on committee T2 of the ECCS towards european recommendations for fatigue design, following the first issue of the respective recommendations for statically loaded structures [4, 5]. This is the framework with which most of the national and international activities concerning applied research and development of fatigue design concepts for aluminium structures will have to orientate.

2. CODE DRAFTING AND INTERRELATIONS

Development of standards for welded aluminum structures will have to be in conformity with the corresponding development of standards of steel structures. This is an important fact of the consultations in the last years. So although past experience such as in the ASCE Paper 3341/1962, the british code CP 118:1969 (the most extensive code for aluminium, which is also under revision currently) and elements from the scandinavian codes will have to be taken into account the projected ECCS code will follow parallel developments of the ECCS steel code, the Eurocode 3, Chap. 9 on Fatigue and respective steel codes, here especially DS 804 (German Federal Reilways) and DIN 18800/6, SIA 161 (Switzerland), BS 5400 (UK), AASHTO Specifications (U.S.A.) and the Ontario Code 1979 (Canada). In order to accomplish this goal of multilevel adaptation but in the same time provide for flexibility due to the variety of alloys, production, handling jointing and environmental parameters as well as for future concepts and needs it is imperative that an underlying pattern of theoretical interrelations and pragmatic assumptions will have to be agreed upon. Accepting this pattern beforehand is a rather new approach [6], but we have been working partly towards this direction for aluminium structures since some time [7].

Task Group 4 of the ECCS (chairman Mr. Trüb, Alusuisse, Switzerland), with members form several european countries, representing industry, research and university institutions forms the nucleus. A vivid transfer of experiences with other bodies, as mentioned above, takes place, nevertheless, through numerous affiliations of its members with these committees. The Institute for Steel Structures (ISS) at the Technical University of Munich has been charged with the task to prepare the draft of the ECCS fatigue recommendations.

On the other hand, as mentioned in [7, Ref. 3 - 4 - 5], there exists a background with sufficient data on specimens for most of the typical structural alloys and their jointing methods. In October 1980 a new international "Committee for Aluminum Fatigue Data Exchange and Evaluation" (CAFDEE) was formed in Munich, after the iniative and under the chairmanship of R.A. Kelsey, Alcoa. There are two representatives from each country - one from industry and one from a code making body. The declared purpose is "to collect and analyze fatigue data on aluminium and, when required, recommend tests needed to develop appropriate information, which can be used by the various organizations active in writing design specifications for fatigue loading of aluminium structures and components". A first view of the documentation has been given [9, 10] and we are now going to produce a comprehensive set of fatigue date based on already available information on the Fatigue Data Bank at Iowa State University and recent new data. For this purpose a special questionnaire for registering data on welded or bolted joints has been distributed to potential suppliers of information, Fig. 1.

BASE METAL	STATIC PROPERTIES	BIBLIOGRAPHIC
Alloy / Temper / Treatment Mechanical Properties Chemical Composition	Strength Values Loading Pattern Test Details	DATA CODE
JOINT DESCRIPTION	FATIGUE TEST RESULTS	
Welded Bolted Geometry (Pass sequence)	Stress/ Cycles to Failure (define!) WELDING PARAMETERS	
LOCATION OF FAILURE	Process / Procedure / Treatment	
	WELD QUALITY NDT Testing / Fracture Surface	

Fig. 1 CAFDEE Fatigue Data Questionnaire

In retrieving data parameters such as *alloy* and *temper*, *specimen* and *joint type* must be specified. *Welding procedure*, *special treatment*, *stress ratio* are optional. Probably within the next year the bank information will be made available to the ISS in Munich, where the second stage of statistical and regressional analysis by means of specially developed computer programs along the principles outlined in [1, 2, 7] will take place. Still, since experience shows, that any conflict may arise in the area of data analysis, a subcommittee of CAFDEE will develop a consensus and a methodology on the presentation of fatigue data [11].

Lastly experimental research projects are a further source of information. These involve current investigations in industry and the ISS with (a) full-size welded beams - in accordance with the opening remarks on the relevance of such information - and (b) on crack development and propagation in welded structures.

3. THE ECCS RECOMMENDATIONS

An outline of the first draft [5] was presented in [3, 8] and it is the purpose of this paper to summarize only some of the relevant points, adding comments after the subsequent discussion in ECCS, as well as in relation with some developments in the draft of Eurocode 3. Further work, concerning mainly final assumptions on the data analysis and presentation as well as definition of standard constructional detail cases with respective standardized stress-life design curves, will be dealt with during the presentation of this paper.

The underlying pattern of theoretical assumptions involves a model of the theory of fatigue strength, followed by a model of the theory of damage accumulation, a model of the theory of safety and probability of failure and closes with a specified network of stress-life curves, allowing a unified adjustment of notch and loading cases and materials to "design categories" [6, 2]. Through this unified, yet modular arrangement of parts of the whole, in functional interdependency to each other, one can easily adapt them to corresponding problem areas or future needs.

Fig. 2 outlines the contents of the recommendations.

1 INTRODUCTION	5 STRUCTURAL DESIGN STRESSES	8 ALLOWABLE STRESSES
2 PRINCIPLES	6 FATIGUE STRENGTH	9 SUPPLEMENTARY FATIGUE PREDICTION METHODS
3 MATERIALS	7 SAFETY / RELIABILITY	10 FABRICATION CONTROL / INSPECTION MAINTENANCE
4 LOADING		

APPENDIX: A - DEFINITIONS / SYMBOLS
B - REFERENCES
C - EXAMPLE ON USE OF DATA ITEMS
D - STANDARD LOAD SPECTRA
E - SCHEME FOR ASSESSEMENT OF RESIDUAL FATIGUE LIFE

Fig. 2 ECCS CT2 Recommendations Aluminium Fatigue

Parts 1 and 2, as mentioned in Fig. 2, do not differ significantly from corresponding specifications for steel. Yet it is intended to provide here recommendations not only for traditional civil engineering structures but also for structures of various sea or land transport means. Relevant parameters for the expression of fatigue strength curves will be alloy dependent, presumably; especially in the lower cycle life areas. (This fact may imply different slopes for S/N-curves). For practical reasons a limit cycle number below which no fatigue assessment is necessary, depending on the field of application and in accordance with static design allowable stresses, will be defined. A similar provision, as a fatigue threshold, may be set up through the applied stress value not exceeding a certain limit. Still these regulations will have to be based on empiricism rather - as a result being somewhat conservative - than on analytical derivation. With more and new information becoming available, through quantitative description of fatigue crack mechanisms this goal seems possible, yet at a later stage. Current projects contribute to this, but will have to be evaluated on a common basis eventually.

Whether there is a significant effect of R-ratio, mean stress or residual stress on fatigue strength must still be judged. There seems to be a mean stress dependency for R-ratios up to $\pm$ 0, but still data from different codes is contradictory [3]. An analytical quantification of the residual (mean) stress on fatigue strength, as this is defined by the cyclic stress-strain history on the basis of the critical fatigue notch factor may express the significance of the phenomenon more clearly [12].

The fatigue behavior of aluminium alloy components and weldments at low temperatures may be assumed as identical to that at room temperature.

Relative to loading and fatigue spectra derivation, parts 4 and 5, principles similar to those for steel are adopted. The rain-flow or reservoir cycle counting method is stipulated and spectrum loading data is related to constant loading data by means of Miner's rule (its validity assumed even for different R values). Various spectrum transformations depending on the design purpose are possible.

Depending on the application with a characteristic stress history the definition of a fatigue damage equivalent stress (f.d.e.s.) for o n e loading event may be computed, thus simplifying design procedures, Fig. 3. Codes then state the f.d.e.s. value for a certain detail and application defined relative to the specification design load spectrum. Differences to the actual load spectrum are expressed by the ratio λ of real to design stresses. For such computations assumptions as to shape and analytical expression of the fatigue strength curve are required.

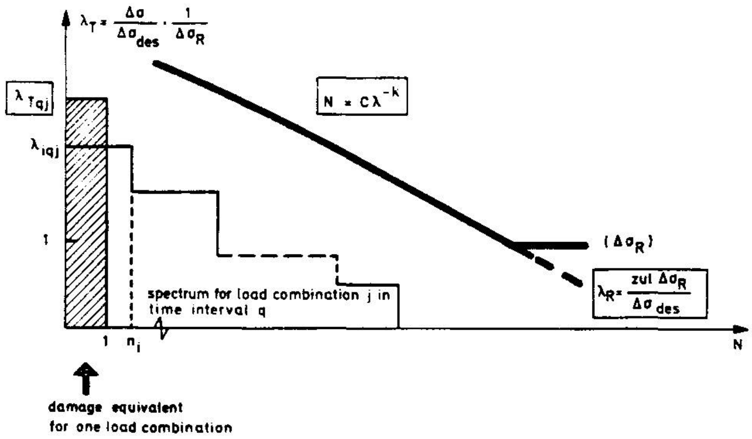

Fig.3 Computing $\boxed{\lambda_{Tqj} = \left[\Sigma n_i \lambda_{iqj}^k\right]^{1/k}}$ the fatigue damage equivalent stress amplitude

(depending on material, notch and component / system characteristics - $\Delta\sigma_R$ - and relative to the specification load spectrum - S_3 ; $\Delta\sigma_{des}$ -)

A definitive presentation of the evaluation of fatigue strength will still have to follow, as already mentioned; yet practically all elements of fatigue data analysis and depiction in form of P-S-N curves have been dealt with amply, as in [1, 6, 7, 8] and numerous other publications. The design elements of the proposed fatigue curves, as given in Fig. 4, also allow an unproblematic adaptation to recent decisious within the Eurocode: slope with a higher value than for steel, transition point respective to endurance limit different from curve reference value. Opinions about the definition of the transition point only in relation with the slope and after accounting for all data points in the 10^6 - 10^7 cycle region [13] conform to former proposals [2] to perform with aluminium fatigue data an optimization of position and slope of the mean

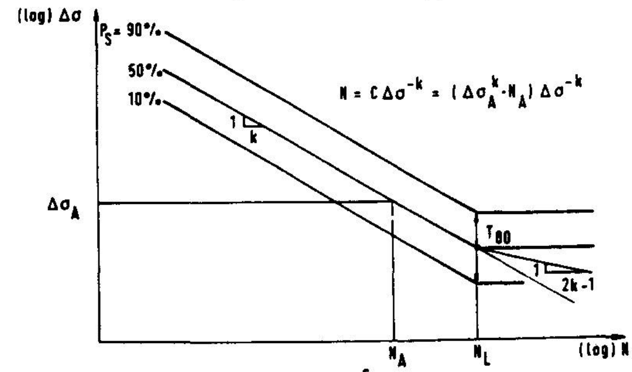

Fig. 4

Reference fatigue life $N_A = 2 \cdot 10^6$; $\Delta\sigma_A$
Endurance fatigue life N_L = ?
possibly different for various alloys
Parallel P-S-N curves ; $T_{80} = 1.50$; k = ?
k dependent on alloy (?) and notch case possibly a priori definition
Universal definition possibility through relation $\lambda = \frac{\Delta\sigma}{\Delta\sigma_A}$
Equidistant gradation $\Delta\sigma_{A(m)} = \Delta\sigma_{Ao} \cdot 1.122^{-2m}$
Mean stress σ_m dependence

curve by quantifying run-outs and computing parameters by means of the maximun-likelihood-method. A computer program to be used with the CAFDEE data has been developed at the ISS. Provisions about allowable S-N curves in the Eurocode proposal, as a lower probability of survival range do not include enough background information about scatter etc. - a common coefficient of variation on life cycles has been assumed yet for all cases - so that experimental data cannot be readily correlated. In the case of aluminium, mean curves as well as parallel lower bounds will be given [7, 8]. Thus beneficial design curves could be derived based on experimental evidence.

In the course of classifying standard S-N curves for various structural detail classes (up to six different cases for practical reasons) existing codes - i.e. the british BS 5400 on steel, CP 118 on aluminium, the german DIN 15018, also north american proposals as to a user-oriented display of structural details [1] etc. - and the new Eurocode classification based on well-founded studies of the IIW together with proposals emerging from industrial experience [14] will have to be considered.

It is a remarkable fact that along with similar notions in international codes for steel the ECCS recommendations for aluminium seek to provide for supplementary fatigue prediction or residual life estimation methods. That is, parallel to the conventional semi-probabilistic S-N concept, low-cycle-fatigue concepts for the crack initiation phase and fracture mechanics methods are presented. These methods may be used to classify new or non-classified details within the framework of the tables or to introduce possibilities for failure analysis and assessment of existing or projected structures. The methodology and especially characteristic design values must yet be developed for use in specifications and this is the task of several industrial and university (ISS) research projects. The literature is voluminous on the subject and we only indicate here recent publications concentrating on the applicability to aluminium structures [15, 16].

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