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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

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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 2×10^6 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 3×10^6 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 2×10^6 cycles was 21.7 ksi. A specimen subjected to a stress range of 18.9 ksi had still not failed after 4×10^6 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 0.75×10^6 cycles, while in the latter case the value was 14.5 ksi at 1.5×10^6 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 0.6×10^6 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 5×10^6 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 2×10^6 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 3×10^6 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:

$$f_{cr} = 0.5\ f'_c - \frac{2}{3} f_{min} \tag{A1}$$

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

$$f_{rr} = 20 \text{ ksi} \tag{A2}$$

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:

$$f_{tr} = 0.10\ f_{pu} \tag{A3}$$

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

$$f_{tr} = 0.04\ f_{pu} \tag{A4}$$

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).