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# Concrete Structure Design for Fatigue Loading—Report

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## Concrete Structure Design for Fatigue Loading—Report

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# Concrete Structure Design for Fatigue Loading—Report

Reported by ACI Committee 215

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The committee would like to thank H. Hadad for his contribution to this report.

*Fatigue is a mechanical degradation process caused by repeated loads, such as traffic loading or wind loads on a bridge, that results in irreversible damage in concrete structures. Many types of concrete elements are subjected to repeated loads, such as airport and roadway pavements, bridge girders, bridge decks, wind turbines, and prestressed concrete railroad ties. This document provides information that will benefit practicing engineers interested in the design or rehabilitation of concrete structures subjected to high-cycle fatigue—that is, stress cycles in which the material behavior remains within the elastic range. The effects of repeated loads on plain concrete, reinforcing materials, and reinforced concrete systems are discussed based on a summary of available literature. This report does not contain detailed design procedures but rather should be considered*

*a general resource providing a comprehensive overview of fatigue issues in reinforced concrete structures.*

**Keywords:** design; fabric-reinforced cementitious matrix; fatigue; fiber-reinforced concrete; fiber-reinforced polymers; prestressed concrete; rehabilitation; reinforced concrete; reinforcing materials; service life.

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## CHAPTER 1—INTRODUCTION AND SCOPE

### 1.1—Introduction

Fatigue is a mechanical degradation process caused by repeated loads, such as traffic loading or normal wind loads on a bridge, that results in irreversible damage in concrete structures. Because individual application of these service loads would not cause significant deformation or damage, fatigue damage occurs gradually from the cumulative effects of thousands or millions of load cycles. This report does not discuss the effects of high-amplitude load cycles associated with extreme events such as an earthquake or unintentional overload.

Many types of concrete elements are subjected to repeated low stress loads; common examples include airport and roadway pavements, bridge girders, bridge decks, wind

turbines, and prestressed concrete railroad ties. Although in-service fatigue failures of concrete structures and their components are rare, the fatigue behavior of reinforced concrete structures is important to consider for numerous reasons. For example, fatigue behavior can be a controlling parameter that determines the service life of concrete pavements. In other cases, fatigue damage can lead to increased cracking with resulting loss of stiffness and strength in concrete members under service loads, which can lead to failure. In statically indeterminate structural systems, changes in stiffness caused by fatigue will also influence the distribution of loads. In summary, fatigue behavior affects the serviceability, safety, and durability of concrete structures, and its effects should be recognized in design to ensure that in-service cyclic stress ranges remain at an acceptably low level.

Live load amplitudes applied to a structure tend to grow over time, while new structures are becoming more lightweight through greater design optimization and the use of high-performance materials; this increases the ratio of live to dead loads. As a result, the importance of transient, cyclic stresses in proportion to an element's total load capacity is likely to increase over the service life of a structure. Furthermore, increasing use of construction materials such as post-tensioned concrete, fiber-reinforced polymers (FRPs), and fiber-reinforced concrete (FRC) requires engineers to have a broad understanding of the fatigue characteristics of various materials and systems.

Although service loads nominally produce stresses that are within the elastic range of the material or structure, small defects or geometric discontinuities can result in local amplified stress concentrations that exceed the elastic capacity of the material and form small damage zones or nucleation sites. The preliminary stage of the degradation process, where the development of these damage zones occurs, is often referred to as the initiation period. As these loads may be repeatedly applied thousands or millions of times, the damage zones grow in size (propagation) or in number (accumulation), leading to changes in the behavior of the larger material or composite component. In reinforced concrete members, this damage manifests through the formation of cracks in the concrete, reinforcement, or at their interface. The result of propagation and accumulation is usually a net reduction in the effective cross section of the member, its reinforcement, or in the bond strength between the reinforcement and the concrete, typically resulting in a loss of member stiffness and strength.

Although real live loads vary greatly in magnitude and application time sequence, for the purposes of design, an equivalent constant amplitude load cycle (for example, a sine function) with a well-defined maximum and minimum stress or strain level is normally assumed. This constant amplitude fatigue model presents many advantages for fatigue analysis, including the use of  $S$ - $N$  curves for presenting and interpreting fatigue life data. In those curves,  $S$  represents normalized applied stress or strain amplitude and is plotted on a vertical axis using a linear (or sometimes logarithmic) scale against  $N$ , which is the number of load cycles to failure.

(also called fatigue life) and is plotted on the horizontal axis using a logarithmic scale. As the stress amplitude decreases, the number of cycles to failure increases. Thus, the fatigue strength is lower than that of the undamaged material under a monotonic (that is, static) load.

Appropriate procedures for addressing variable amplitude stress cycles remain a topic of debate among researchers and are not discussed in depth in this document. In the absence of specified fatigue loading (for example, AASHTO), for the purposes of design, it is usually sufficient to consider a constant amplitude fatigue load model where the minimum stress value is the stress corresponding to the design dead load, and the maximum stress value corresponds to the total design dead and live loads.

## 1.2—Scope

This report provides information that will benefit practicing engineers interested in the design or rehabilitation of concrete structures subjected to fatigue cycles caused by regular service loads. This report does not contain detailed design procedures but rather should be considered a general resource providing a comprehensive overview of fatigue issues in reinforced concrete structures.

This document is divided into three thematic sections:

1) Chapters 1 and 2 provide a general introduction, and notation and definitions, respectively.

2) Chapters 3 and 4 discuss the fatigue behavior of plain unreinforced concrete and reinforcing materials (steel bars and tendons and FRP components), respectively. These chapters are likely to be of interest for those interested in applications of unreinforced concrete, as well as for practitioners seeking to develop a better understanding of the mechanisms of fatigue in various materials.

3) Chapters 5 through 8 focus on the fatigue behavior of reinforced concrete systems considering the interaction between constituent materials. These chapters are intended for practicing engineers dealing with the design or assessment of common applications of structural concrete subjected to repeated loads. The content is organized by the type of reinforcing system used: conventional reinforced concrete containing internal steel or FRP bars (Chapter 5); concrete reinforced with bonded or unbonded prestressed tendons (Chapter 6); concrete reinforced with discrete fibers (Chapter 7); and external strengthening sheets or laminates (Chapter 8).

## CHAPTER 2—NOTATION AND DEFINITIONS

### 2.1—Notation

|           |                                                                                |
|-----------|--------------------------------------------------------------------------------|
| $A_t$     | = area of concrete below the neutral axis, in. <sup>2</sup> (mm <sup>2</sup> ) |
| $a$       | = crack length, in. (mm)                                                       |
| $c$       | = neutral axis depth, in. (mm)                                                 |
| $D$       | = damage                                                                       |
| $d_d/d_n$ | = crack length increment per cycle, in. (mm)                                   |
| $d_f$     | = effective depth of FRP laminate, in. (mm)                                    |
| $d_s$     | = effective depth of beam reinforcement, in. (mm)                              |
| $E_c$     | = Young's modulus of concrete, psi (MPa)                                       |
| $E_s$     | = Young's modulus of steel, psi (MPa)                                          |

|               |                                                                                                                                                            |
|---------------|------------------------------------------------------------------------------------------------------------------------------------------------------------|
| $f$           | = frequency, Hz                                                                                                                                            |
| $f'_c$        | = concrete compressive strength, psi (MPa)                                                                                                                 |
| $f_{fu}$      | = design tensile strength of FRP, psi (MPa)                                                                                                                |
| $f_{pu}$      | = ultimate strength of prestressing strands, psi (MPa)                                                                                                     |
| $f'_t$        | = concrete tensile strength, psi (MPa)                                                                                                                     |
| $f_y$         | = yield stress of steel, psi (MPa)                                                                                                                         |
| $\ell_{bf}$   | = distance from flexural crack to tip of interfacial debonding zone at FRP level, in. (mm)                                                                 |
| $\ell_{bs}$   | = distance from flexural crack to tip of interfacial debonding zone at steel level, in. (mm)                                                               |
| $\ell_c$      | = crack spacing, in. (mm)                                                                                                                                  |
| $M_{cr}$      | = cracking moment, lb-ft (kN-m)                                                                                                                            |
| $M_D$         | = applied bending moment due to dead loads, lb-ft (kN-m)                                                                                                   |
| $M_L$         | = applied bending moment due to live loads, lb-ft (kN-m)                                                                                                   |
| $N$           | = number of applied repeated load cycles $n$ needed to cause failure—that is, fatigue life                                                                 |
| $n$           | = number of applied repeated load cycles within the fatigue process                                                                                        |
| $P$           | = probability of failure, %                                                                                                                                |
| $R$           | = applied stress ratio, $S_{max}/S_{min}$                                                                                                                  |
| $S$           | = cyclic stress amplitude normalized with respect to equivalent static strength—that is, fatigue strength                                                  |
| $S_{max}$     | = maximum cyclic stress level normalized with respect to equivalent static strength                                                                        |
| $S_{min}$     | = minimum cyclic stress level normalized with respect to equivalent static strength                                                                        |
| $S_n$         | = steel stress at the primary crack after $n$ load cycles, psi (MPa)                                                                                       |
| $S_o$         | = steel stress at the primary crack after first load cycle, psi (MPa)                                                                                      |
| $S_2$         | = normalized confining stress in biaxial compression tests                                                                                                 |
| $S_3$         | = normalized axial stress in biaxial compression tests                                                                                                     |
| $T_g$         | = glass transition temperature, °F (°C)                                                                                                                    |
| $W$           | = crack width after one load cycle, in. (mm)                                                                                                               |
| $W_N$         | = crack width after $n$ cycles, in. (mm)                                                                                                                   |
| $\beta$       | = ratio of distance between neutral axis and crack measurement location to distance between neutral axis and reinforcement location after first load cycle |
| $\beta_n$     | = ratio of distance between neutral axis and crack measurement location to distance between neutral axis and reinforcement location after $n$ load cycles  |
| $\Delta K$    | = stress intensity range, ksi-in. (MPa-mm)                                                                                                                 |
| $\Delta S$    | = stress range—that is, the difference between maximum and minimum cyclic stress level, psi (MPa)                                                          |
| $\delta_n$    | = reinforcing bar slip after $n$ cycles, in. (mm)                                                                                                          |
| $\delta_o$    | = initial reinforcing bar slip, in. (mm)                                                                                                                   |
| $\varepsilon$ | = strain                                                                                                                                                   |
| $\gamma$      | = applied load factor (AASHTO)                                                                                                                             |
| $\Sigma_o$    | = sum of perimeters of tension reinforcement, in. (mm)                                                                                                     |
| $\sigma_2$    | = confining stress in biaxial compression tests, psi (MPa)                                                                                                 |
| $\sigma_3$    | = axial stress in biaxial compression tests, psi (MPa)                                                                                                     |

## 2.2—Definitions

Please refer to the latest version of ACI Concrete Terminology for a comprehensive list of definitions. Definitions provided herein complement that resource.

**amplitude**—measure of the change in stress during a load cycle, or the difference between the mean stress and the peak stress.

**cycle**—process of loading and unloading a material sample or structural element once.

**damage**—irreversible physical change of a material or structural element resulting in a loss in performance.

**defect**—material imperfection likely to develop as a fatigue crack. Defects occur at a variety of scales, such as slip discontinuities, cracks, inclusions, debond interfaces or delaminations, voids, dry or misaligned fibers in FRP materials, and nicks and scrapes resulting from handling damage.

**discontinuity**—abrupt change in geometry (such as a re-entrant corner) or stress path in a structural element (such as an anchor plate).

**endurance limit**—maximum cyclic stress below which an infinite number of constant amplitude loading cycles can be applied without causing fatigue failure; also known as the fatigue limit.

**fatigue**—process of mechanical degradation resulting from the application of repeated loads, such as traffic loading or wind loads on a bridge, that causes damage in a material or structural element.

**fatigue life**—number of applied load cycles resulting in failure of a material or structural element.

**fatigue strength**—greatest stress that can be sustained for a given number of stress cycles without failure.

**frequency**—number of load cycles applied per unit of time.

**fretting fatigue**—physical material damage associated with wearing that occurs at the contact surface between two materials experiencing relative cyclic displacement.

**static strength**—peak stress of a material or structural component loaded monotonically to failure.

**stress concentration**—localized increase in internal stress caused by a defect or discontinuity.

**stress range**—difference between maximum and minimum stress levels within an idealized fatigue loading pattern.

## CHAPTER 3—PLAIN CONCRETE

### 3.1—Scope

Whereas most concrete structural elements are reinforced in some way, certain applications (such as pavements) may consist mainly of plain concrete subjected to repeated loading. Furthermore, it is necessary to understand the fatigue behavior of concrete as a constituent material prior to considering the effects of cyclic loading on composite reinforced structures (Chapters 5 to 8). Hence, this chapter provides an overview of the fatigue behavior of unreinforced concrete.

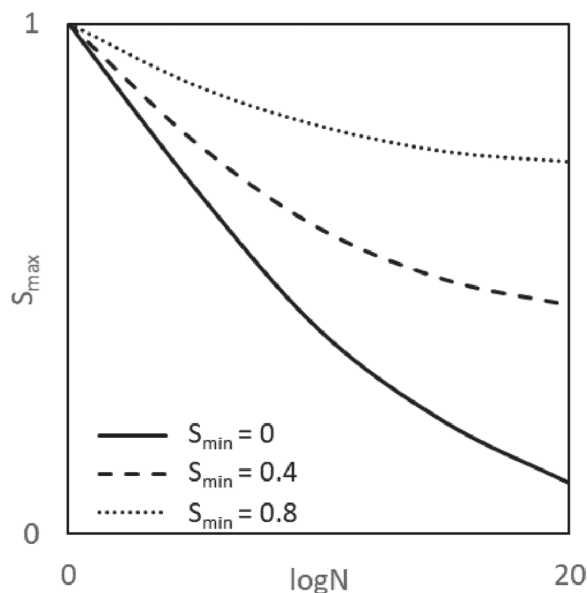


Fig. 3.2—Approximate  $S$ - $N$  curves for concrete (adapted from CEB-FIP Model Code 2010).

### 3.2—General

The classic way to interpret the fatigue performance of concrete is through the use of  $S$ - $N$  curves (Fig. 3.2), which visually indicate that a concrete element subjected to cyclic loads may fail after a number of cycles, even if the maximum applied stress is less than the concrete compressive strength. The value of  $N$  depends on both the minimum and maximum applied normalized stress levels,  $S_{min}$  and  $S_{max}$ , respectively, within the idealized fatigue loading cycle. Usually the x-axis (fatigue life) is plotted in logarithmic scale.

In general, there is significant scatter in  $S$ - $N$  data from nominally identical test specimens. This is caused by the inherent scatter of the strength of concrete (Cornelissen 1984), heterogeneity of the concrete matrix, and the known random nature of the fatigue process. Usually, it is necessary to test a large number of specimens for each stress level to define reliable  $S$ - $N$  curves (Ortega et al. 2018; Paskova and Meyer 1994; Parida et al. 1990).

### 3.3—Fatigue as a degenerative process

Similar to what happens with many biological processes, fatigue in concrete can be understood as a degenerative process of the material. One of the distinguishing characteristics of fatigue in concrete, unlike other materials (such as metals), is that the energy of the cyclic loading that is transmitted to the concrete element causes an internal structural damage that is manifested as initiation and growth of internal microcracks, resulting in diffuse damage (Fig. 3.3).

As observed in Fig. 3.3, several cracks appear inside the specimen, which means that the energy from the fatigue loads does not result in a concentrated damage—only one crack—but a distributed (or diffuse) damage—several small cracks throughout the specimen. The macroscopic consequence of this diffuse damage is a progressive variation of mechanical parameters of concrete: compressive strength and Young's modulus, as well as permanent strain, of increasing value.

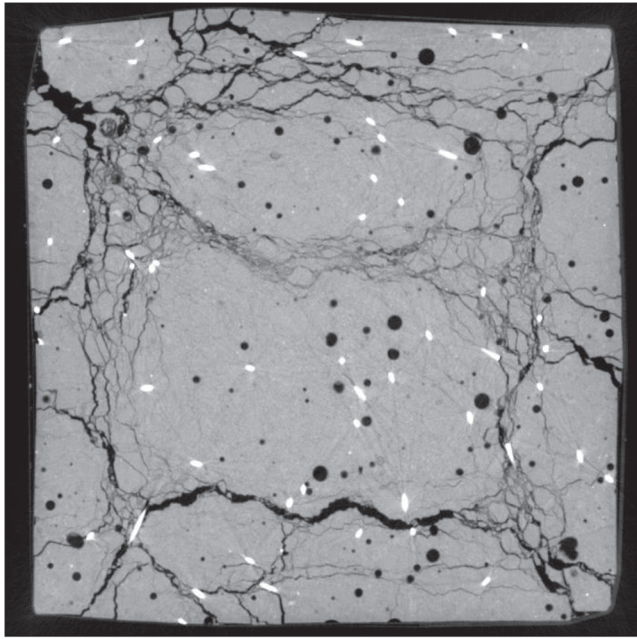


Fig. 3.3—Internal cross section image of a concrete specimen previously subjected to compressive cyclic loads, obtained using a computer tomography scan. (Image courtesy of Research Group AUSINCO – University of Burgos.)

Notably, in metallic materials, the energy transmitted by cyclic loading is concentrated at the edge of a crack, causing its growth. The material further away from the crack does not present any reduction in mechanical properties.

**3.3.1 Variation of strain**—A progressive modification of the concrete stress-strain diagram occurs during fatigue loading, characterized by the appearance and growth of permanent strain (Fig. 3.3.1).

Many researchers have attempted to apply the Paris Law (Paris and Erdogan 1963) to describe and explain the growth of cracks during fatigue, which leads to permanent deformations (Bažant and Schell 1993; Bažant and Xu 1991; Baluch et al. 1987; Perdikaris et al. 1986). The Paris Law proposes that there is a relation between the increment of the crack length per cycle and the range of stress intensity at the crack tip,  $\Delta K$ , expressed by Eq. (3.3.1)

$$\frac{da}{dn} = C \cdot (\Delta K)^m \quad (3.3.1)$$

where  $a$  is the crack length;  $da/dn$  is the crack length increment per cycle; and  $C$  and  $m$  are experimentally determined coefficients.

**3.3.2 Variation of Young's modulus**—A progressive reduction in the apparent modulus of elasticity also occurs with increasing number of cycles (Fig. 3.3.2). This is also due to the occurrence and growth of microcracks.

As seen in Fig. 3.3.2, three phases can be identified. During the first phase, a sharp decrease of stiffness occurs, caused by the occurrence and the initial growth of the microcracks and deterioration of adhesion between paste and aggregates. During the second phase, a linear reduction of the elastic modulus occurs due to a progressive and stable growth of

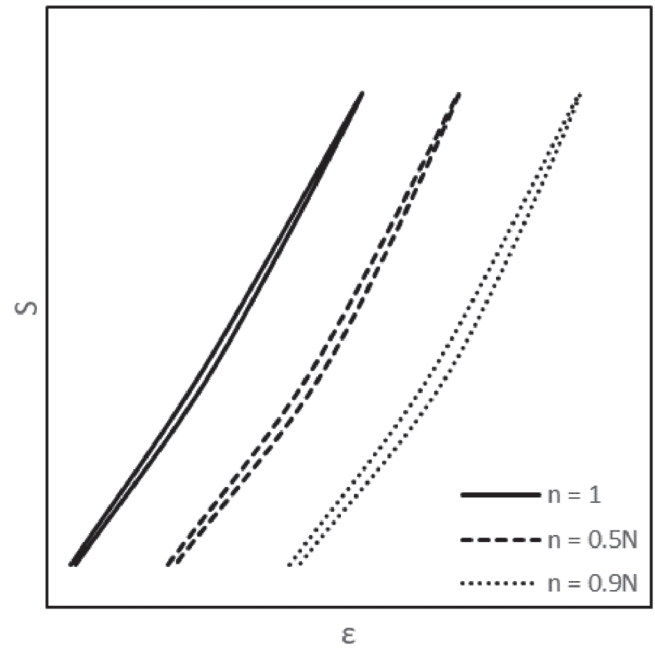


Fig. 3.3.1—Variation of the stress-strain diagram with increasing number of cycles (adapted from Petkovic et al. [1990]).

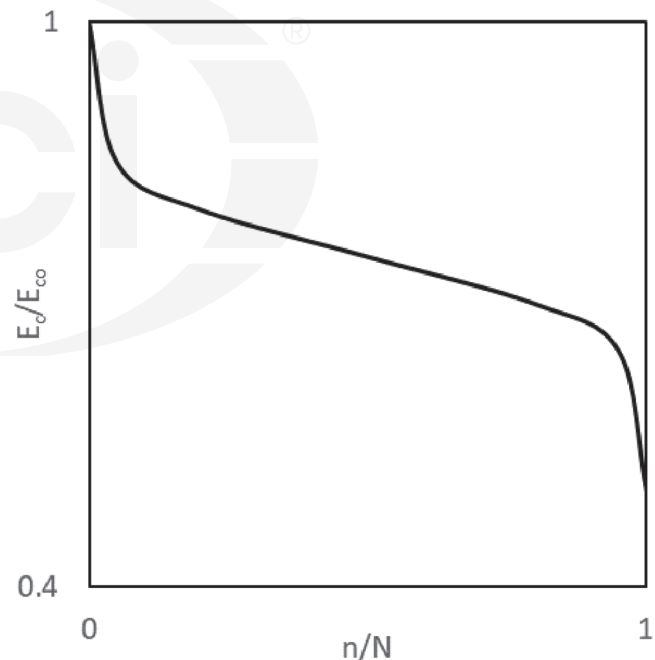


Fig. 3.3.2—Variation of the apparent modulus of elasticity due to fatigue loads (adapted from Holmen [1979]).

the microcracks. In the third phase, a rapid decrease of the elastic modulus is observed as microcracks reach their critical length and their growth becomes unstable.

It is generally accepted that the transition between the first and second stages takes place between approximately 10 and 20% of the fatigue life, whereas the third stage begins at approximately 80% of the fatigue life (Holmen 1979).

The slope of the second stage is associated with the secondary strain rate. There is a strong linear relationship between the logarithm of secondary strain rate and the loga-

rhythm of the number of cycles to failure. The secondary strain rate depends on the loading frequency and maximum strain level (Medeiros et al. 2015; Saucedo et al. 2013; Oneschkow 2012; Sparks and Menzies 1973). Zanuy (2008) and Zanuy et al. (2009) also proposed an empirical relation between Young's modulus and number of cycles, using their own testing results in addition to results from other research.

The fact that an apparent progressive reduction of the Young's modulus occurs with the number of cycles is a very important issue. It implies that, in the case of structural elements subjected to cyclic loading, a progressive loss of stiffness occurs, which has many structural consequences (Vicente et al. 2014a; Zanuy et al. 2009):

- (a) Increase of vertical deflection in bridges and viaducts (not only under live loads, but under dead loads as well)
- (b) Progressive increase in structural period
- (c) Increase of prestress losses, in prestressed/post-tensioned concrete elements

It is important to note that the modulus of elasticity obtained during cyclic loading tests corresponds to the dynamic modulus of elasticity. This is slightly different from the static modulus of elasticity obtained through the standard test (ASTM C469/C469M). During recent years, some researchers have attempted to measure the variation of static modulus of elasticity with the number of cycles (Vicente et al. 2014b). The results are inconclusive because of the limited scope of work, but this work proposed a different way of analyzing the damage caused by cyclic loading.

**3.3.3 Variation of compressive strength**—There are few studies that focus on the variation of the compressive strength of plain concrete with increasing number of cycles because it cannot be measured during cyclic loading (unlike the modulus of elasticity). The results shown by Mínguez (2012) and Vicente et al. (2014b) show that, in the range of load levels and number of cycles usually supported by current bridges and viaducts located on high-speed train lines (corresponding with very low levels of damage), compressive strength not only does not decrease but also increases slightly. However, in general, fatigue damage is believed to have a detrimental effect on concrete compressive strength as a result of the initiation and propagation of microcracks.

### 3.4—Fatigue life models

One of the essential characteristics of concrete related to fatigue life is its significant scatter. Many research projects have been developed in this field and all of them have concluded that scatter in the measured fatigue life of two orders of magnitude between two identical specimens may be expected. This fact greatly complicates the analysis of the results.

Probabilistic approaches are essential to analyze and understand the results. For example, it is more useful to present families of  $S$ - $N$  curves for different probabilities of failure ( $p$ ) (Fig. 3.4); these are sometimes called  $S$ - $N$ - $P$  curves. The probability of failure associated with each curve represents the likelihood that a randomly selected sample will fail prior to reaching the fatigue life corresponding to a given stress level.

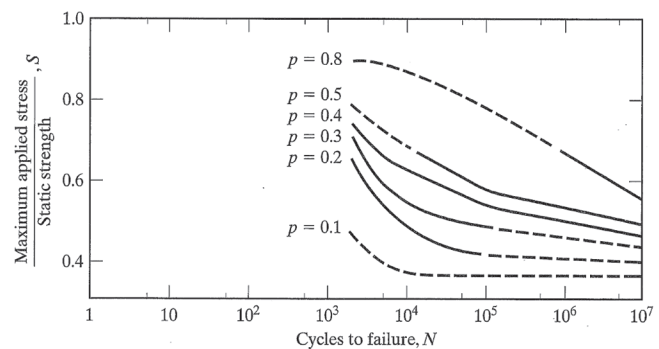


Fig. 3.4—Fatigue curves for concrete under flexure considering different probabilities of failure (Mindess et al. 2003).

**3.4.1 Deterministic models**—Deterministic models present explicit  $S$ - $N$ - $P$  curves for different values of probability of failure. In all cases, the strength of concrete is a deterministic value—either characteristic or average compressive strength. This is the classic approach and many researchers have developed expressions in this regard (Lee and Barr 2004; Cachim et al. 2002; Paskova and Meyer 1997; Kim and Kim 1996; Zhang et al. 1996; Grzybowski and Meyer 1993; Oh 1991; Petkovic et al. 1990; Kazimierz Furtak 1984; Hsu 1981; Holmen 1979; Tepfers and Kutti 1979; Aas-Jakobsen 1970).

International standards are generally in line with this approach (Eurocode 2; fib Model Code 2010). At present, the procedure developed by fib Model Code 2010 is considered the international reference in calculating the fatigue strength of concrete.

**3.4.2 Probabilistic models**—The main problem of deterministic models is that they do not account for the dispersion of the compressive strength of concrete. In recent years, probabilistic models of failure have been developed. They take into account the statistical dispersion of the strength of concrete in estimating the fatigue life (Liang et al. 2017; Saucedo et al. 2013; Przybilla et al. 2011; Castillo et al. 2008, 2009; Zhao et al. 2007).

### 3.5—Important parameters

There is no standard configuration for fatigue testing of concrete. Many of the  $S$ - $N$  curves in the literature have been obtained using different test configurations and geometry of the specimens. Fatigue data obtained from two different test configurations cannot be compared directly. Using normalized fatigue strength partially eliminates the influence of geometric variables, such as the geometry of the specimen, material composition, curing conditions, and test age.

Fatigue strength of concrete mainly depends on the following factors:

- (a) Stress range
- (b) Stress distribution
- (c) Load history
- (d) Loading frequency
- (e) Stress reversal
- (f) Multi-axial loading

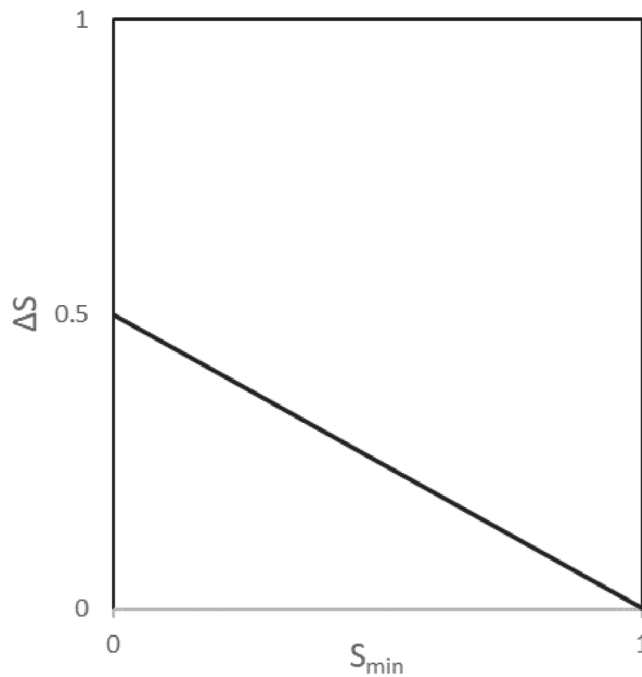


Fig. 3.5.1—Fatigue nomograph for concrete,  $N = 1$  million cycles (adapted from Cornelissen and Siemes [1985]).

Each of these variables are discussed in detail in the following sections.

**3.5.1 Stress range**—The fatigue life  $N$  of concrete depends on both the maximum and the minimum stresses, as seen in Fig. 3.2. The greater the difference between  $S_{max}$  and  $S_{min}$ , the shorter the corresponding fatigue life; this is known as the stress range  $\Delta S$ . Data may also be presented with respect to the applied stress ratio ( $S_{min}/S_{max}$ ) defined by the parameter  $R$ .

The relationship between normalized maximum and minimum stress levels is demonstrated in Fig. 3.5.1 for the case of a fatigue life of 1 million cycles. In this case, the sign of both maximum and minimum loads is the same (that is, both are compressive), so  $R \geq 0$ . This type of graph is a tool to obtain the minimum and maximum stresses that can be applied to the concrete specimen to reach a fatigue life of 1 million cycles. This simplified approach assumes that the stress range varies linearly from 50% of the static strength if  $S_{min}$  is zero, to 0 if  $S_{min}$  is equal to the static strength.

**3.5.2 Stress distribution**—Fatigue is a stress-dependent phenomenon. Therefore, when there is a nonuniform stress distribution, failure occurs when the most critical region (that is, the region subjected to the greatest stress) reaches its fatigue life. Usually fatigue testing is carried out under concentric compression load. There are limited works regarding the influence of eccentricity of loading on fatigue behavior. Noteworthy research in this field includes the work carried out by Ople and Hulsbos (1966).

Comparisons between different test results should be based on critical stresses rather than applied load ranges. In this case, eccentric loads have been observed to result in a slight increase in the fatigue life (Ople and Hulsbos 1966; CEB 1988; Mindess et al. 2003) compared to concentric load tests.

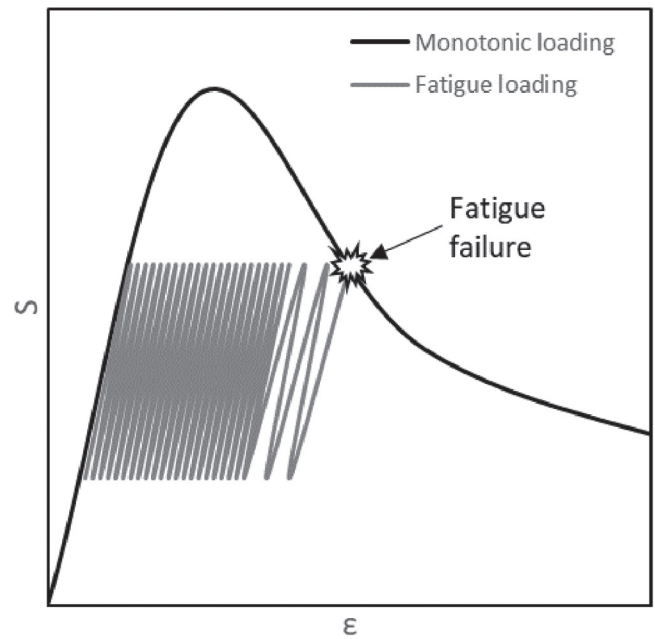


Fig. 3.5.3—Strain failure criterion for concrete in compression (adapted from Petryna et al. [2002]).

**3.5.3 Load history**—Concrete structural elements are often subjected to varying loads of random nature. To date, there are very few studies that analyze the effect of random loads on the fatigue behavior of concrete. Almost all fatigue tests have been conducted with constant values of maximum and minimum stress.

Traditionally, the hypothesis of Miner (1945) has been generally accepted to simulate the effect of a series of variable loads, which are random in nature. Miner defines damage  $D$  caused by cyclic loading with constant maximum and minimum stress values as the quotient between the number of cycles applied,  $n$ , and the total number of cycles that it can resist,  $N$ , for a given stress range  $i$ .

In addition, Miner assumes the principle of superposition of damage, as given by Eq. (3.2.3) for  $k$  different stress ranges

$$D = \sum_{i=1}^k D_i = \sum_{i=1}^k \frac{n_i}{N_i} \quad (3.2.3)$$

Applying the principle of linear superposition of damage, it is considered that a concrete element fails when  $D = 1$ . However, practice has shown that this hypothesis of linear damage proposed by Miner is not directly applicable to concrete elements subjected to variable loads of random nature (Oh 1991; Cornelissen and Reinhardt 1984; Shah 1984; Holmen 1982; Siemes 1982; Leeuwen and Siemes 1979; Tepfers et al. 1977). Miner's sum for reversed loading has been found to exceed unity (Cornelissen and Reinhardt 1984).

This topic has been of continuous scientific interest (Keerthana and Chandra Kishen 2018; Slowik et al. 1990; Holmen 1982). In recent years, this criterion is being displaced by another, based on fracture mechanics, and called the criterion envelope curve (Zaidy 2006; Zaidy et al. 2009; Petryna et al. 2002; Panner 2002; Park 1990).

According to this criterion, the stress-strain diagram from the static response of the specimen provides limit values for the fatigue response. According to [Park \(1990\)](#) and [Petryna et al. \(2002\)](#), fatigue failure occurs when the maximum strain under cyclic loads reaches the stress-strain diagram corresponding to monotonic loading (Fig. 3.5.3). Alternatively, [Pfanner \(2002\)](#) has proposed an energy model of failure in which failure occurs when the energy dissipated during the static test (area under the monotonic stress-strain diagram) is equal to the energy dissipated in the fatigue test.

Miner's damage rule as a fatigue failure criterion is usually more conservative than other approaches (it estimates a lower fatigue life), and it is currently commonly accepted. However, the effects of concrete maturity, rest periods, and sustained loads on fatigue behavior have not been sufficiently studied. Laboratory tests have shown that rest periods and sustained loads between periods of cyclic loads tend to increase the fatigue strength of concrete ([Hilsdorf and Kesler 1966](#)). However, if the sustained loads exceed 75% of the static compression strength, they result in a deterioration of fatigue life associated with ongoing creep effects ([Shah and Chandra 1970](#)). This contradictory creep effect can be explained considering that low levels of sustained loads improve the compressive strength of the concrete, whereas higher values of sustained loads lead to increased internal microcracking and facilitates failure.

**3.5.4 Loading frequency**—Early studies ([Graf and Brenner 1934, 1936](#)) showed that if the test frequency  $f$  is between 4.5 and 7.5 Hz, it has a weak influence on concrete fatigue. However, for frequency values lower than 0.16 Hz, the fatigue life is reduced. Some other researchers expanded the range ([Zhang et al. 1996](#); [Sparks and Menzies 1973](#); [Awad and Kilsdorf 1971](#); [Aas-Jakobsen 1970](#); [Murdock 1965](#)), suggesting that the test frequency had a minimal influence on fatigue life when it was between 1 and 15 Hz and the maximum stress in tested cylinders was less than 75% of the compressive strength. For test frequencies outside of this range, [Kazimierz Furtak \(1984\)](#) and [Zhang and Wu \(1997\)](#) proposed modifications to the Aas-Jakobsen  $S$ - $N$  equation to include the test frequency.

Notable works have been carried out in recent years ([Saucedo et al. 2013](#); [Medeiros et al. 2015](#)) to analyze the concrete response to low-frequency cyclic loading. The results show that the fatigue life at their lowest frequencies (1/16 Hz) is at least one order of magnitude lower than that obtained for their highest frequencies (4 Hz).

It is important to note that the results from tests conducted at high frequencies are sensitive to the ability of servo-hydraulic systems to consistently reach intended peak loads. The change in stiffness of the specimen caused by fatigue degradation must also be accounted for.

**3.5.5 Stress reversal**—Few tests have been conducted to understand the response of concrete under stress reversals, although it has been observed to affect the fatigue life ([Cornelissen and Reinhardt 1984](#); [Tepfers 1982](#)). [Zhang et al. \(1996\)](#) developed an analytical model to predict fatigue life in concrete considering the possibility of stress reversals in the load cycle. The results are shown in Fig. 3.5.5,

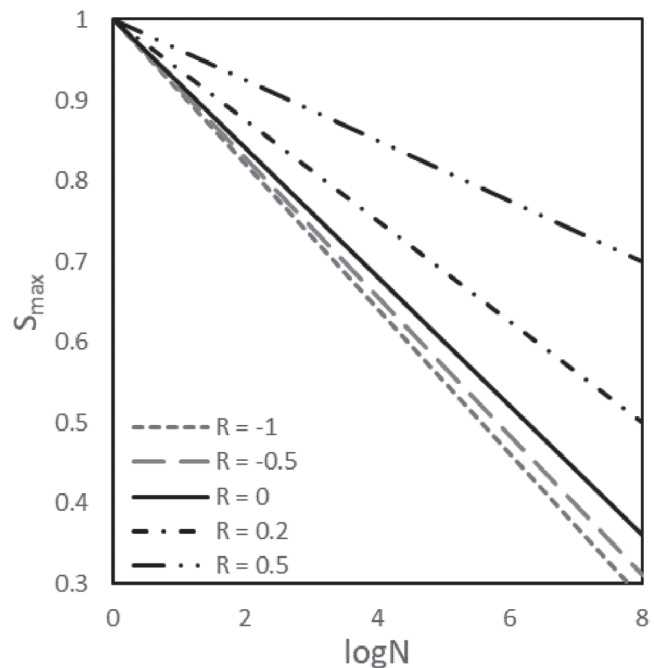


Fig. 3.5.5—Predicted  $S$ - $N$  curves for concrete showing effects of reversing sense of minimum stress;  $R < 0$  indicates max and min stresses have opposing senses (adapted from [Zhang et al. \[1996\]](#)).

where the maximum (compression) and minimum (tension) applied stress values are normalized with respect to their corresponding ultimate static stress. The model shows that stress reversal does reduce fatigue life in concrete, but this effect is small compared with the effect of conventional stress ratio ( $R > 0$ ), as described in Section 3.5.1.

**3.5.6 Multi-axial loading**—The presence of stresses in two or three directions substantially modifies the response of concrete in fatigue. However, the research carried out to date is limited.

Increasing the magnitude of orthogonal compressive confining stress in combination with applied compressive load cycles results in an increase in fatigue life ([Meng et al. 2011](#); [Wang and Song 2011](#); [Zhu et al. 2003](#); [Nelson et al. 1988](#); [Su and Hsu 1988](#); [Buyukozturk and Tseng 1984](#)). Therefore, biaxial compressive fatigue life is greater than in the case of uniaxial compression. However, the extent that compressive confining stress increases fatigue life depends on the level of confining stress. Notable increases in fatigue are seen when the confining stress is approximately half of the applied cyclic stress; for greater magnitudes of confining stresses, the benefits to fatigue life reduce. This effect is illustrated in Fig. 3.5.6.

In Fig. 3.5.6,  $\sigma_2/f_c$  is the relative maximum compressive stress along one direction—that is,  $S_2$ —and  $\sigma_3/f_c$  is the relative maximum compressive stress along a perpendicular direction—that is,  $S_3$ . The response of concrete to biaxial fatigue stresses that represent a combination of compression and tension behave differently; increasing orthogonal tensile stresses act to significantly reduce the fatigue life of concrete under cyclic compressive loads, and vice versa; the failure is normally controlled by the propagation of indi-

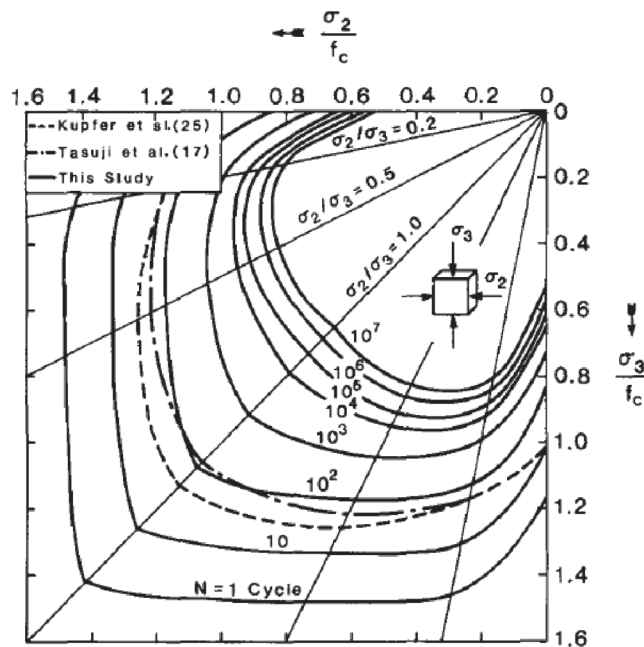


Fig. 3.5.6—Fatigue envelope diagram for compressive-compressive biaxial fatigue loads (Su and Hsu 1988). Note that  $f_c$  is equivalent to  $f'_c$ .

vidual (Mode I) cracks set up by the tensile component of the principal stress (Subramaniam 1999).

## CHAPTER 4—REINFORCING MATERIALS

### 4.1—Scope

Most structural concrete elements use reinforcement to efficiently transfer tensile forces, enhance ductility, and permit internal force redistribution. Consequently, the fatigue performance of reinforced concrete structures is strongly influenced by the properties of the reinforcing material used. This chapter focuses on the behavior under cyclic loading of two categories of materials commonly used as reinforcement for concrete structures: 1) steel (Section 4.2); 2) and fiber-reinforced composites (Section 4.3). The interaction of these materials with concrete and the associated fatigue performance of reinforced concrete systems are discussed in Chapters 5 to 8.

### 4.2—Fatigue of steel

**4.2.1 General**—Steel is the most common reinforcing material used in concrete structures and is manufactured in various forms, including bars, strands, fibers, or mesh. Other metals are not explicitly discussed in this document.

Fatigue damage in steel is governed by fracture mechanics and typically involves three main stages: 1) crack nucleation resulting from crystallographic slip in the vicinity of defects in the material; 2) stable crack growth under repeated load cycles resulting from local plastic deformations at the crack tip; and 3) unstable crack growth and fracture once a critical crack length is reached. The fatigue strength of steel depends on chemical composition, microstructure, inclusions, treatments, and other variables (Forrest 1962; Brown and Kunnath

2004). However, geometry is usually the most critical factor; it has been shown, for example, that the fatigue strength of deformed steel reinforcing bars may be only one-half the fatigue strength of smooth coupons machined from samples of the same material (Helgason et al. 1976; MacGregor et al. 1971). This is a result of stress concentrations that occur from geometric discontinuities (for example, reinforcement lugs), residual stresses (for example, weld toes), or other surface irregularities (for example, corrosion pits).

A typical steel reinforcing bar fatigue fracture is shown in Fig. 4.2.1, in which the surface that exhibits a dull, smooth appearance represents the fatigue crack propagation plane. Under magnification, striations corresponding to individual load cycles may be observed on the fracture surface, which are exaggerated in the schematic view shown in Fig. 4.2.1. The zone with a rougher pock-marked texture is the final fracture surface. The bar is shown in the orientation in which it was placed in a beam as flexural reinforcement; it is noteworthy that the fatigue crack did not start from the bottom of the bar at the location of the highest nominal bending stress, but rather started along the side of the bar, at the base of one of the transverse lugs. This is a common characteristic of most bar fatigue fractures; the highest stress in the bar typically occurs at a transverse lug at the location of a concrete crack.

The form of reinforcement is, therefore, of primary importance in evaluating fatigue characteristics, while other influencing parameters are secondary. Reinforcement in the form of steel bars, prestressing tendons, and wire mats will be examined in this section. Steel fibers may also be used as reinforcement; however, their fatigue performance in concrete is mainly a function of their interaction with the concrete matrix. The fatigue behavior of fiber-reinforced concrete composites is discussed further in Chapter 7.

#### 4.2.2 Effects of material type and geometry

**4.2.2.1 Steel reinforcing bars**—Most concrete structures in the United States are reinforced with Grade 60 (Grade 420) or 80 (Grade 550) deformed hot-rolled steel bar reinforcement conforming to ASTM A615/A615M. An approximate fatigue life envelope based on data from tests on concrete beams containing straight deformed bars made in North America is shown in Fig. 4.2.2.1a using data from multiple sources (Fisher and Viest 1961; Pfister and Hognestad 1964; Burton and Hognestad 1967; Hanson et al. 1968; Helgason et al. 1976; Lash 1969; MacGregor et al. 1971). The bars used in these studies varied in size from No. 5 (15 mm) to No. 11 (35 mm), with minimum tensile stress levels ranging from  $-0.10$  to  $0.43$  of the tensile yield strength of the bars (negative ratios indicate compressive stresses). The wide range of the failure envelope suggests that there are many variables in addition to stress range that influence the fatigue strength of deformed reinforcing bars.

Despite the wide range between the upper and lower limit of the fatigue life envelope, Fig. 4.2.2.1 clearly shows a transition from a steeper to a flatter slope in the vicinity of 1 to 2 million cycles ( $\log N = 6$ ), indicating that steel reinforcing bars exhibit a practical endurance limit.

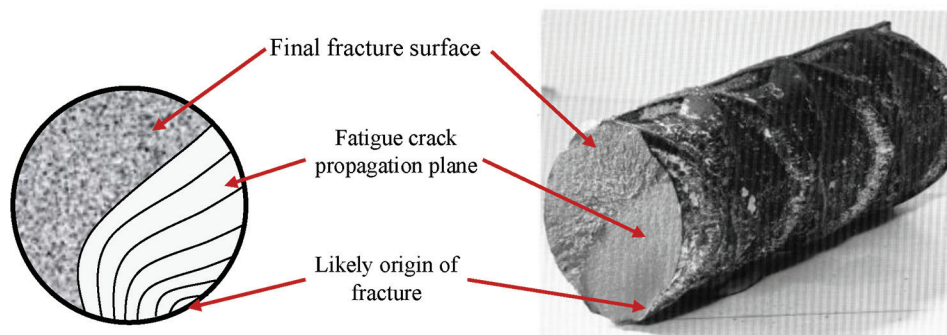


Fig. 4.2.1—Fatigue fracture of deformed steel reinforcing bar.

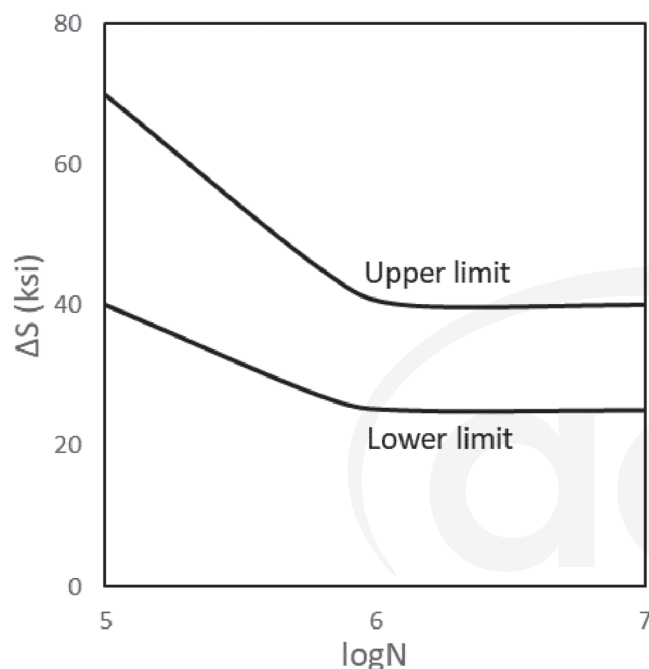


Fig. 4.2.2.1—Approximate fatigue life envelope for reinforcing bars in tension.

of specimens have been tested beyond 10 million cycles (Rabbat and Corley 1984).

**4.2.2.1.1 Stress level**—In several investigations, it has been reported that the fatigue strength of reinforcing bars is relatively insensitive to the minimum stress level (Fisher and Viest 1961; Pfister and Hognestad 1964; Rehm 1960). However, other studies (Helgason et al. 1976; MacGregor et al. 1971) found that fatigue strength decreases as the minimum stress level increases. These results are reflected in the approach used for bridges according to AASHTO (2017a), in which fatigue stress limits for straight reinforcing bars are given by Eq. (4.2.2.1.1a) and (4.2.2.1.1b)

$$\gamma(\Delta S) \geq S_{TH} \quad (4.2.2.1.1a)$$

$$S_{TH} = 26 - 22 \frac{S_{min}}{f_y} \text{ in ksi} \quad (4.2.2.1.1b)$$

$$S_{TH} = 180 - 150 \frac{S_{min}}{f_y} \text{ in MPa}$$

where  $\gamma$  is the applied load factor (taken as 1.75 for the AASHTO Fatigue I load combination);  $\Delta S$  is the live load stress range;  $S_{TH}$  is the constant amplitude fatigue threshold;  $S_{min}$  is the minimum live load stress resulting from the Fatigue I load combination; and  $f_y$  is the specified yield strength, which should be within the range of 60 to 100 ksi (420 to 690 MPa) for this calculation.

**4.2.2.1.2 Size effects**—The effect of bar size is examined in Table 4.2.2.1.2 using data from three investigations (MacGregor et al. 1971; Wascheidt 1965; Kokubu and Okamura 1965). Within each study, the bars were made by the same manufacturer, and they also were tested at the same minimum stress level. The fatigue strength in this case is taken as the stress range causing failure at 2 million cycles.

From test results on bars subjected to axial tension, the average fatigue strength of the No. 5 (15 mm) bars was approximately 8% greater than that of the No. 8 (25 mm) bars (Wascheidt 1965). A similar result was obtained for bars in concrete beams (MacGregor et al. 1971; Kokubu and Okamura 1965). The effect of beam geometry was found to be relatively insignificant (Helgason et al. 1976).

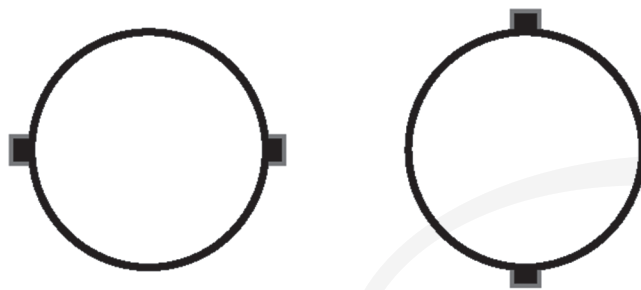
**4.2.2.1.3 Surface deformations**—Deformations on reinforcing bars provide the means of obtaining good bond between the steel and the concrete. However, these same deformations produce stress concentrations at their base, or at points where a deformation intersects another deformation or a longitudinal rib (Pfister and Hognestad 1964; Burton 1965). The locations of fatigue cracks have been observed to coincide with these deformations (Tilly 1984).

The size and shape of the surface deformations can have a significant effect on the fatigue characteristics of reinforcing bars. This includes the height and slope of the deformations, as well as the fillets at the base of the deformations. Reinforcing bar lugs generally have stress concentration factors of 1.5 to 2.0 (Derecho and Munse 1968).

Tests on bars having a fillet base radius varying from approximately 0.1 to 10 times the height of the deformation have been reported (Hanson et al. 1968; Helgason et al. 1976; MacGregor et al. 1971; Kokubu and Okamura 1965). These tests indicate that when the base radius is increased from 0.1 to approximately one to two times the height of the deformation, fatigue strength is increased appreciably. A further increase in base radius beyond one to two times the height of the deformation has little effect on fatigue strength.

**Table 4.2.2.1.2—Effect of bar size**

| Grade of bar, ksi (MPa) | Reference                 | Fatigue strength relative to No. 8 bar |               |                |
|-------------------------|---------------------------|----------------------------------------|---------------|----------------|
|                         |                           | No. 5 (15 mm)                          | No. 6 (20 mm) | No. 10 (30 mm) |
| 40 (280)                | MacGregor et al. (1971)   | 1.06                                   | —             | 0.99           |
|                         | Wascheidt (1965)          | 1.11                                   | —             | —              |
|                         | Wascheidt (1965)          | 1.05                                   | —             | —              |
|                         | Kokubu and Okamura (1969) | —                                      | 1.12          | —              |
| 60 (420)                | MacGregor et al. (1971)   | 1.08                                   | —             | 0.96           |
|                         | Wascheidt (1965)          | 1.05                                   | —             | —              |
|                         | Kokubu and Okamura (1969) | —                                      | 1.04          | —              |
|                         | Kokubu and Okamura (1969) | —                                      | 1.10          | —              |
| 80 (550)                | MacGregor et al. (1971)   | 1.20                                   | —             | 0.95           |
|                         | Wascheidt (1965)          | 1.10                                   | —             | —              |

*Fig. 4.2.2.1.3—Reinforcing bar cross section with horizontal (left) and vertical (right) orientation of longitudinal ribs.*

the height of the deformation does not significantly improve fatigue strength.

Tests have indicated that decreasing the angle of inclination of the sides of the deformations with respect to the longitudinal axis also increases the fatigue strength of a reinforcing bar (Soretz 1965; Kokubu and Okamura 1969). This increase occurs for bars with lugs having abrupt changes in slope at their bases. It has been noted that the base radius should be determined in a plane through the longitudinal axis of the bar because this is the direction of the applied stress (Hanson and Helgason 1971). The base radius determined in this plane will be substantially larger than a base radius determined in a plane perpendicular to a sharply inclined lug.

It has also been demonstrated that the fatigue strength of a reinforcing bar may be influenced by the orientation of the longitudinal ribs in a beam (Burton and Hognestad 1967). An increased fatigue life was obtained when the longitudinal ribs were oriented in a horizontal position along the sides of the bar rather than at the top and bottom of the bar (Fig. 4.2.2.1.3).

**4.2.2.1.4 Mechanical properties**—The yield strength of steel reinforcing bars was found to have little effect on fatigue strength in the range of 2 to 5 million cycles (Pfister and Hognestad 1964; Lash 1969; MacGregor et al. 1971). On the other hand, the fatigue life of Grade 40 (Grade 280), 60 (420), and 80 (550) bars subjected to a higher stress range of 36 ksi (248 MPa) was found to increase linearly as a func-

tion of yield strength (Helgason et al. 1976; Helgason and Hanson 1974).

**4.2.2.1.5 Bending**—Bent bars will tend to have a lower fatigue strength than straight bars (Pfister and Hognestad 1964; Rehm 1960; Bennett 1982). The fatigue strength of No. 8 (25 mm) bars with 45-degree bends and an internal bend radius of 3 in. (75 mm) was found to be a little more than 50% below the fatigue strength of straight bars (Pfister and Hognestad 1964). For a bend radius of 5 in. (125 mm), the fatigue strength reduction was approximately 29% for plain bars and 48% for deformed bars (Rehm 1960).

**4.2.2.1.6 Welding and mechanical connections**—Residual stresses caused by welding reduce the fatigue performance of steel. A one-third reduction in fatigue strength has been reported for bars with tack-welded stirrups compared with stirrups attached by wire ties (Burton and Hognestad 1967). All the fatigue cracks in this test initiated at the weld locations.

The fatigue strength of butt-welded reinforcing bars in reinforced concrete beams has been reported to be as low as 24 ksi (165 MPa) (Sanders et al. 1961; Walls et al. 1965). In contrast, the fatigue strength of single- and double-lap welded splices has been found to be as low as 4 ksi (28 MPa) (Paulson and Hansen 1991).

The fatigue strength of mechanically connected reinforcing bars is also reduced relative to that of continuous bars (Bennett 1982; Tassi and Magyari 1982; Paulson and Hansen 1991; Manning 1994). The magnitude of reduction in fatigue strength varies widely with the type of mechanical connector or welded joint. The rate of reduction in fatigue life with increasing stress range for connected bars is approximately the same as that for continuous reinforcing bars. There is a large variability in the fatigue limit (that is, endurance limit) stress range, with some mechanical connectors and welded joints having a mean fatigue limit as low as 4 ksi (28 MPa).

The location of fatigue fracture depends on the connection configuration. Nonthreaded mechanical connections often fracture through the reinforcing bar at the connector, while threaded mechanical connections often fracture through the threads at the first engaged thread. Some mechanical connections can also fracture through the connector itself.