Long term random dynamic loading of concrete structures

Report by RILEM COMMITTEE 36-RDL

CONTENTS

PREFACE

Already in the fall 1974 a programme on "Long term random dynamic loading of concrete structures" was presented at the RILEM Permanent Committee meeting. Spurred by the development of offshore concrete structures, other international organizations seemed to be interested in this subject. It was therefore necessary to use some time to discuss the object and scope of this proposal and its relation to other task groups. A RILEM committee, 36-RDL, was formed 1977 to work out a review of status on this subject. Most of the task has been carried out by correspondence. A few meetings have been held. Very few members were able to attend the meetings. Members of the committee have given contributions to various parts of the report. The committee's secretary has not only dealt with the contributions, but has also provided significant parts of the content of the report.

The many aspects of fatigue of concrete structures have been dealt with by presenting and discussing some selected, relevant research results.

The objective has been to give a broad overview of observed behaviour of the components of reinforced concrete structures under dynamic loading. Such an overview has not been published previously.

It is hoped that the report gives the structural engineer an introduction of the aspects of fatigue. It should also provide a basis for futher development. The report should give the researcher a general background and the list of references is a source for more profound studies.

There is a need for an international agreement on testing of materials and components subjected to dynamic loading. Small differences in material, geometry, loading or environment can make great differences in fatigue behaviour. Such a common base is needed for design recom-

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mendation as well as for international cooperation for further development.

There is a need for more realistic design recommendation for fatigue, not only with regard to fatigue life but also with regard to cracking and deformations.

The lack of satisfactory guidelines calls for actions. This report can hopefully be a contribution to such actions.

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SUMMARY

The behaviour under dynamic loading and the fatigue life of concrete structures have recently become an important factor in design of an increasing number of structures exposed to dynamic loading. For some structures calculations are required to document that the structure will not suffer a fatigue failure.

This report is a review of fatigue phenomena of concrete structures. The phenomena are dealt with one by one by presenting some selected research results and discussing these results briefly.

Plain concrete in compression and tension is considered—in relation to material quality, stress rate, eccentric loads, environment and random loading.

Reinforcing steel is dealt with rather briefly because within this subject there are several reports available.

Reinforced concrete components can suffer fatigue failure in many ways as they can under static load. However, a structural member which fails because of a bending moment under static load, may very well suffer a shear failure in fatigue. Therefore it is difficult to relate this shear capacity in fatigue to the shear capacity under static load.

Palmgren-Miner hypothesis is applied to concrete and discussed in connection with stochastic loadings. The hypothesis does not reflect the properties of concrete. However, with minor modifications the Palmgren-Miner hypothesis is applied in design, basicly because no better method is available.

In many cases the increased deformations under repeated loads are more critical than the fatigue life of a structure. These deformations, which are much greater for concrete than for steel, may cause redistributions of stresses that may result in unexpected cracking and large deformations.

Rest periods and small dynamic loadings seem to strengthen the structural member exposed to dynamic loading. This phenomenon adds to the uncertainty in relating short time laboratory tests to behaviour of real structures.

1. INTRODUCTION

1.1. General aspects of fatigue failures of concrete structures

Fatigue of concrete structures has been under investigation since the end of the last century. However, the fatigue problem has not until recently been paid much attention by designers or regulations. This increasing interest is due to several reasons:

– The use of higher strength materials together with refined design procedures have resulted in slender structures where the dead load forms a smaller part of the total load capacity.

– The use of new types of structures such as marine structures subjected to wind and wave loading.

– The effects of repeated loading on the characteristics of the materials (static strength, stiffness, etc.) may be significant under service loading even if they do not cause a fatigue failure.

It seems, that the fatigue failures observed in laboratories are not observed in practice. This is due to the fact that in practical structures the concrete stress in most cases is less than 50% of the formal concrete strength and will not lead to a fatigue failure. Moreover, the design is usually based on the 28 day's concrete strength and the increased strength with time will reduce the susceptibility to fatigue.

In design, the stresses in critical sections of the structures are controlled by conventional methods, but the real stresses can be higher and may even reach the strength of the concrete. This may occur when bending cracks appear in anchorage zones of the reinforcement, and is associated with reduced stiffness and progressive failure of concrete under repeated loading. Cracking of concrete structures is sometimes observed in industries with heavy machinery with dynamic loads. The concrete structures are repeatedly repaired and strenghtened during their use and normally these effects are not connected with fatigue even though this is the cause.

1.2. Repeated loading and fatigue in codes and regulations

Only a few regulations and national codes contain detailed rules for verifying the safety against fatigue failure in concrete structures. Some of them are written especially for offshore installations which are exposed to an environment far different from that of onshore structures.

1.3. The significance of fatigue in future structures

The use of lightweight concrete (density about 18 kN/m^3) and ordinary high strength concrete (compressive strength up to 100 MPa) in slender structures will make the ratio between dead load and fluctuating live load even less than in today's structures.

The same can also be said about small concrete elements with little weight and large bearing capacity and for structures placed on sea bed. Special attention has to be paid to floating structures, where the "dead load" is activated into a changing load due to wave action.

Tall buildings and structures with dominating wind forces may suffer fatigue as a limiting factor.

In the future, it is likely that national authorities will require a detailed documentation of the fatigue strength of all structures exposed to cyclic loading.

2. FATIGUE LOADING AND STRUCTURAL RESPONSE

2.1. General

Dynamic loading, i.e. load of variable magnitude, direction and position with time, is usually defined in two different ways:

- Deterministic dynamic loading,
- Stochastic (random) dynamic loading.

By deterministic dynamic loading, the time variation of the loading is fully known. This means that the loading can be predicted at any time and place. By stochastic dynamic loading, however, the variation is not completely known. It is, however, by use of the theory of stochastic processes, possible to estimate the probability that the load will not exceed a given value.

The analysis of the response of a structure exposed to a deterministic dynamic loading will be a deterministic analysis and the response is fully known in the time domain. On the other hand, the response to a stochasic dynamic loading can only be predicted by statistical methods, i.e. a stochastic response.

In connection with fatigue, dynamic loading is usually classified in three categories:

- Impact loading,
- Low cyclic loading,
- High cyclic loading.

Impact loading is characterized by very high rate of loading (pile driving, gas explosion, wave hammering, etc.). Low cyclic loading involves few load cycles of high stress levels at a lower rate of loading (earthquake, storms, etc.). High cyclic loading is characterized by a great number of cycles at lower stress levels and rate of loading less than that for impact loading (rotating machinery, wind and wave loading, etc.). A loading histogram may contain all three categories of loading.

A realistic fatigue prediction depends on extensive dynamic analysis of the structures.

2.2. Deterministic dynamic loading and response analysis

The response of a structure to dynamic loading is generally expressed in terms of displacement parameters through the equation of motion. In a deterministic analysis, the displacement response history in the time domain corresponding to a prescribed loading history can be established by solving the equation of motion as illustrated in figure 1.

The loading history can also be transformed to the frequency domain. Then, by solving the equation of motion the response can be expressed as a function of the frequency as illustrated in figure 2. A solution in the frequency domain is particularly suitable by response calculations of stochastic dynamic loading, but also where mass, damping and stiffness are frequency dependent.

Figure 3 shows some results from the design of a concrete gravity platform based on deterministic analysis [1]. It can be seen how the response values vary with the wave period and the soil stiffness.

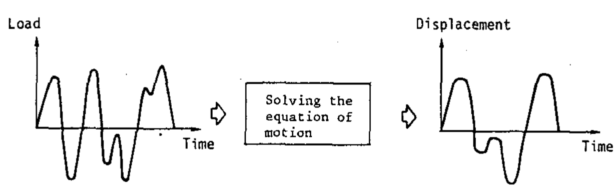


Fig. 1. — Solution in the time domain.

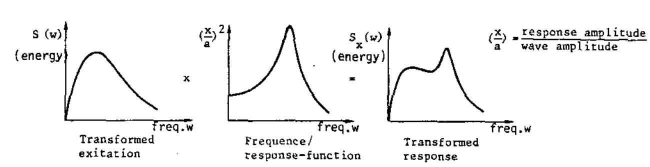


Fig. 2. — Solution in the frequency domain.

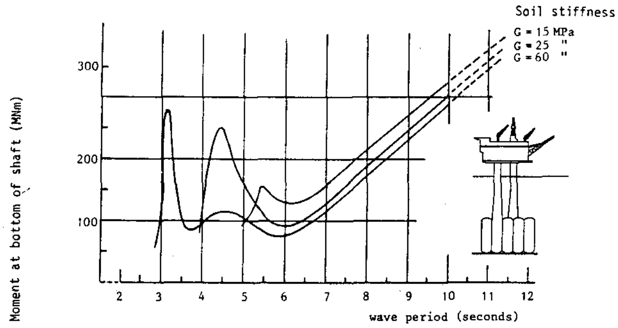


Fig. 3. — Deterministic analysis harmonic response (amplitude values) [1].

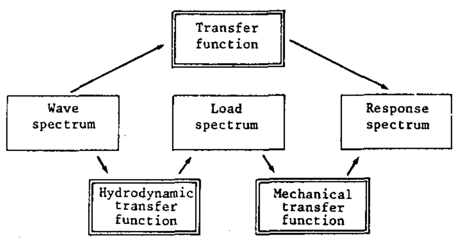


Fig. 4. — Stochastic analysis [1].

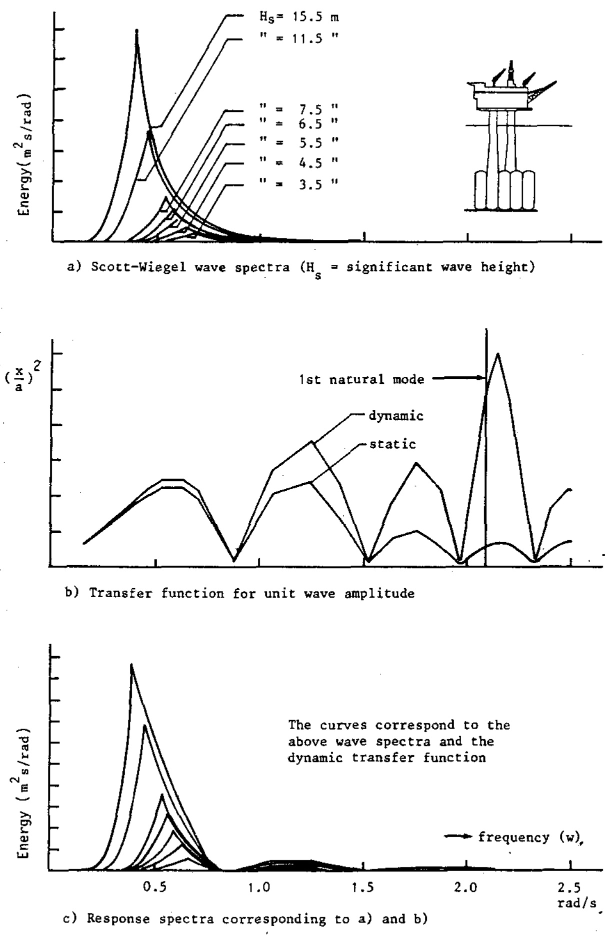


Fig. 5. — Example of stochastic analysis; top shaft moment [1].

2.3. Stochastic dynamic loading and response analysis

In order to account for the random nature of a problem, a linear stochastic analysis procedure may be used [1]. e. g. by wave loading, the short term variation of the sea surface is assumed to be a given process (Gaussian) described by a one-dimensional wave spectrum. The stochastic analysis is concerned with the determination of transfer functions by means of which the wave spectrum is transformed into a response spectrum which forms the basis for statistical predictions. Figure 4 shows schematically how the wave spectrum is transformed into a response spectrum by the use of a transfer function, the upper branch, or by going via a load spectrum by an explicit determination of mechanical and dynamic transfer functions, the lower branch.

Figure 5 shows an example from a stochastic analysis of a concrete gravity platform exposed to a wave loading expressed by the Scott-Wiegel spectrum.

A dynamic analysis, like that presented above, will give the response in all structural elements exposed to a given dynamic loading. As illustrated above, the response will be frequency dependent and may therefore be quite different from a corresponding static analysis. Considering these effects is important to achieve a realistic fatigue prediction.

3. FATIGUE PHENOMENA OF CONCRETE STRUCTURES. A REVIEW OF STATUS

3.1. Fatigue of plain concrete

3.1.1. Mechanism of fatigue

It is assumed that fatigue is a process of progressive, permanent internal structural changes occurring in a material subjected to repetitive stresses. These changes may result in progressive growth of cracks and complete fracture if the stress repetitions are sufficiently large ([2], [3]). The mechanism of fatigue in concrete is not clearly understood and several hypothesis concerning the crack initiation and propagation have been proposed.

Murdock and Kesler [4] formulated the following hypothesis:

> The initiation of fatigue failure may reasonably be attributed to the progressive deterioration of the bond between the coarse aggregate and the binding matrix, together with an accompanying reduction of section of the specimen. The final failure of the specimen occurs by fracture of the matrix. The development of the cracks may be intensified if the modulus of elasticity of the coarse aggregate exceeds that of the binding matrix.

Antrim [5] proposed the following hypothesis:

> Fatigue failure in plain concrete occurs because small cracks form and propagate in the cement paste and the resulting crack pattern weakens the section to the point where it cannot maintain the applied load. The development of this damaging crack pattern depends primarily, if not entirely, on the water-cement ratio of the cement paste and the presence of shrinkage stresses in the cement paste.

Sporadic tests conducted by other investigators have supported both hypotheses, indicating the complexity of the fatigue mechanism of concrete.

However, it may be concluded that fatigue of concrete is associated with development of internal micro-cracks, probably both at the cement matrix/aggregate interface and in the matrix itself, and that the system of fatigue cracks is more extensive than the somewhat similar cracking accompanying static compressive failure [6].

3.1.2. Examination of the crack formation and propagation

The formation and propagation of micro-cracks by cyclic loading may be detected by means of different measuring methods, e.g. by an ultrasonic pulse velocity method or by acoustic emission measurements.

The ultrasonic pulse velocity technique [7] involves measuring the transit time of an ultrasonic pulse through a path of known length in a specimen. The velocity of the ultrasonic pulse in a solid material will depend on the density and elastic properties of the material and, therefore, it will also be affected by the presence of cracks.

The acoustic emission method [7] works on the principle that the formation and propagation of the micro-cracks are associated with the release of energy. When a crack forms or spreads, part of the original strain energy is dissipated in the form of heat, mechanical vibrations and in the creation of new surfaces. The mechanical vibration component can be detected by acoustic methods and recorded, hence micro-cracking may be detected by studying sounds emitted from the concrete.

3.1.3. Determination of damage development by propagation of micro-cracks, strain variations, etc.

A few investigations are known where the development of micro-cracks by cyclic loading is quantified. Figure 6 shows the relationship between the decrease of pulse velocity and the cycle ratio for a constant amplitude test [7] with stress levels $S_{max} = 0.75$ and $S_{min} = 0.05$.

According to this measuring method the micro-crack development consists of three different stages: A rapid increase from start to about 10% of total fatigue life, a uniform increase from 10 to about 80% and then a rapid increase until failure.

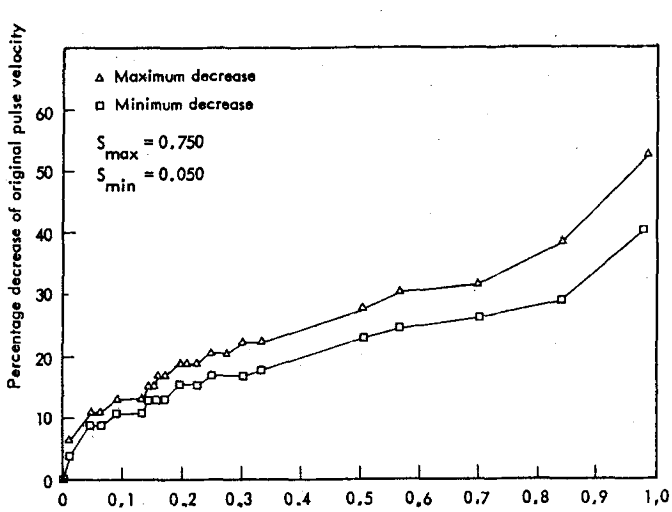


Fig. 6. — Measured decrease of original pulse velocity under constant amplitude loading.

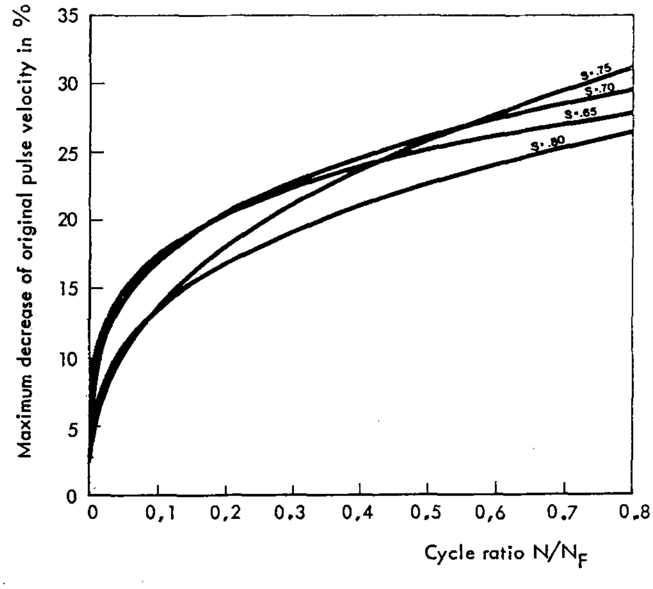


Fig. 7. — Variation of maximum decrease of original pulse velocity under constant amplitude loading. Mean curves [7].

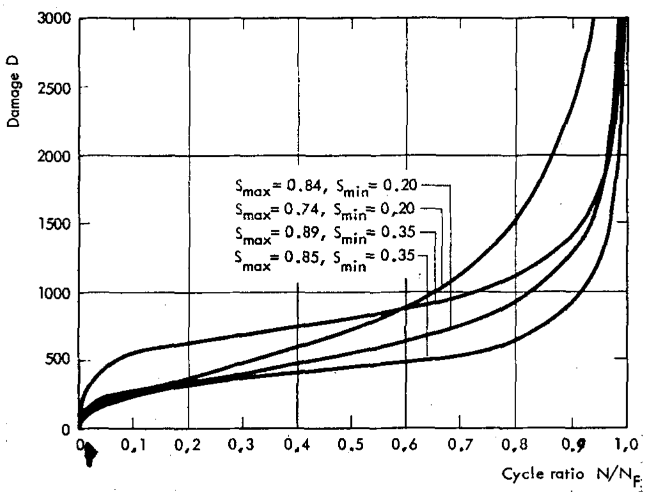


Fig. 8. — Damage accumulation under constant amplitude loading based on an acoustic emission method [8].

Figure 7 shows mean curves of the maximum decrease of the original pulse velocity for different maximum stress levels. There is statistically no significant difference between the curves.

Figure 8 shows the damage as a function of the cycle ratio based on an acoustic emission method [8]. Damage is assumed to take place when the acoustic pulse exceeds a certain threshold level.

By this method the three different stages of damage accumulation can be observed. Statistically there exists no significant difference between the damage accumulation curves in figure 8.

Measurements of the total longitudinal strain variation, i.e. the sum of elastic and inelastic strain, show a development as illustrated in figure 9. It appears that the strain variation with the cycle ratio takes the same course as illustrated in figures 7 and 8 for the microcrack development. The variation of the total strain seems to be a

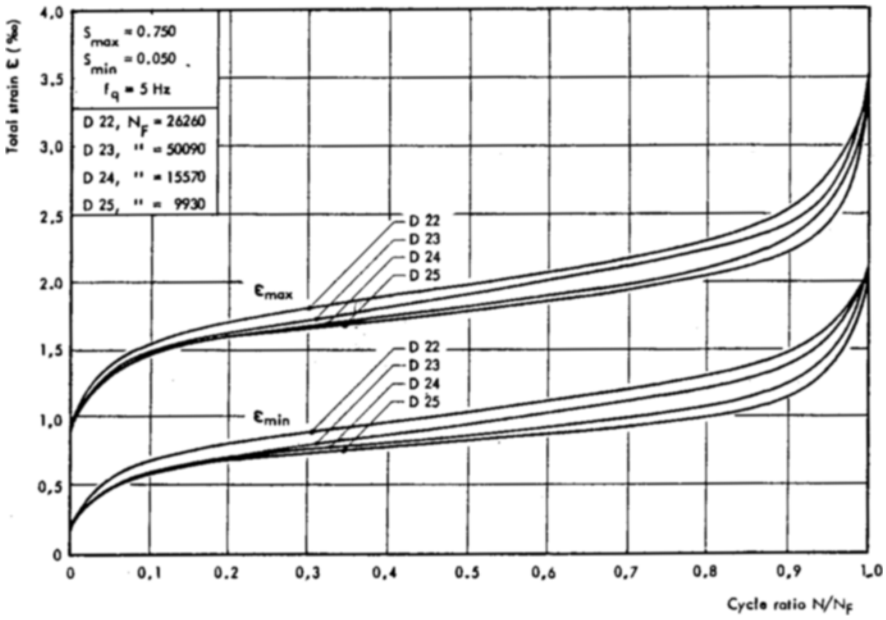


Fig. 9. — Measured variation of total longitudinal strain with the cycle ratio [9].

function of the stress level, but independent of the number of cycles to failure at a constant stress level provided that the duration of the test is less than a few hours. In tests of longer duration, the total strain increases with the time as a function of the stress level. It is suggested [9] that the total maximum strain consists of two components:

$$\varepsilon_{max} = \varepsilon_e + \varepsilon_t, \qquad (1)$$

where ε_e is related to the endurance of the specimen and ε_t is a time-dependent strain which is independent of the endurance of the specimen and can be equated with a creep deformation.

The following expressions for the total maximum strain variation ($\varepsilon_{max} = \varepsilon_e + \varepsilon_t$) have been derived:

$$0 < \left(\frac{N}{N_F}\right) \leqq 0.10:$$

$$\varepsilon_{max} = \frac{1}{\operatorname{tg}\alpha} \left| S_{max} + 3.180\,(1.183 - S_{max}) \,.\, \left(\frac{N}{N_F}\right)^{0.5} \right| + 0.413\, S_c^{1.184}\, \ln(t+1),$$

$$0.10 < \left(\frac{N}{N_F}\right) \leqq 0.80:$$

$$\varepsilon_{max} = \frac{1.110}{\operatorname{tg}\alpha} \left| 1 + 0.677 \left(\frac{N}{N_F}\right) \right| + 0.413 \,.\, S_c^{1.184} \ln(t+1)$$

where:

ε_{max}, maximum total strain (‰);

$\operatorname{tg}\alpha$, secant modulus $= \dfrac{S_{max}}{\varepsilon_0}$;

ε_0, maximum total strain in the first load cycle (‰);

S_{max}, maximum stress level;

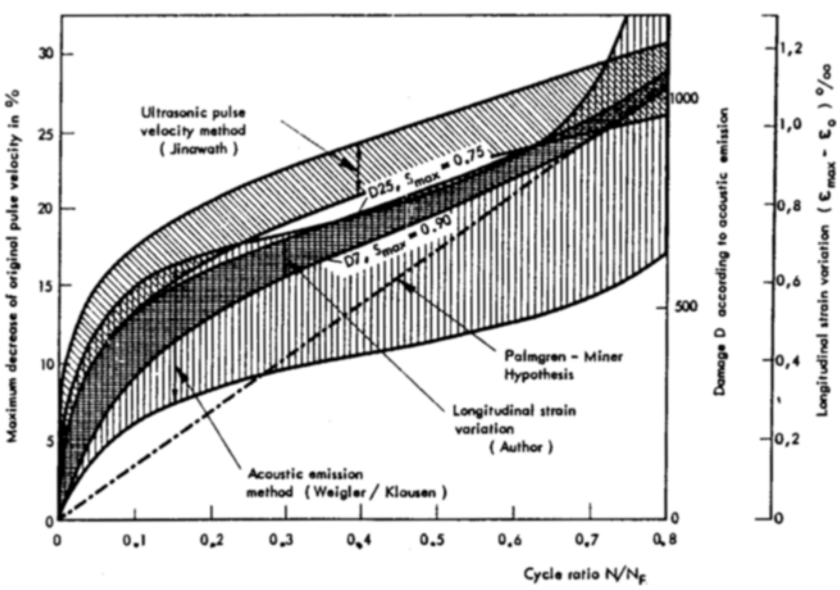


Fig. 10. — Damage accumulation under compressive constant amplitude loading based on three different measuring methods [9].

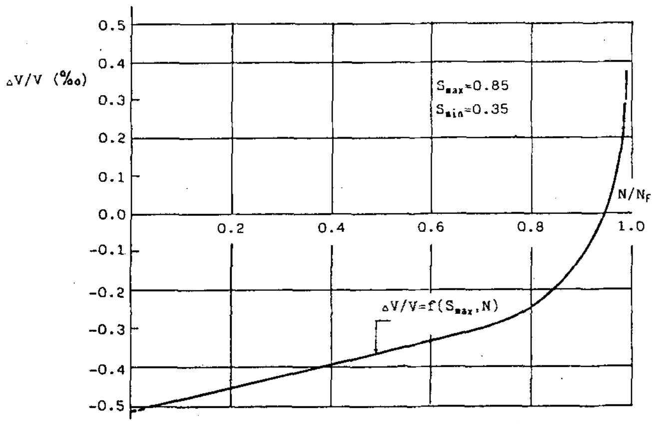


Fig. 11. — Change in volume, $\Delta V/V$, with the cycle ratio under compressive constant amplitude loading [8].

S_c, characteristic stress level $= S_m + \text{RMS}$;
N, number of load cycles;
N_F, number of load cycles to failure;
t, duration of alternating load in hours;
S_m, $1/2\,(S_{min}+S_{max})$, mean stress level;
RMS, $\sqrt{\frac{T}{T_0}}\cdot\int_0^{T_0} x^2(t)\,.dt$, root mean square value;
$x(t)$, stress as a function of time (t);
T_0, total time, i. e. the duration of the cyclic loading.

Also by variable amplitude (random). Loading the total maximum strain variation can be expressed by equation (1) [9]. Hence, by measuring the variation of total maximum strain it should be possible to assess the remaining life of a partially fatigued specimen and ultimate strain can be used as a fatigue failure criterion for concrete.

In figure 10, the damage accumulation based on three different parameters (or measuring methods) is shown [9]. It appears that the course of damage accumulation based on the three measuring methods seems to agree fairly well.

The change in volume, $\Delta V/V$, of a test specimen is another parameter which can be used to predict changes in the internal structure of damage accumulation. Figure 11 shows $\Delta V/V$ as a function of the cycle ratio based on dilatometer measurements [8]. It appears that the volume increases slowly in the first part of the fatigue life, corresponding to the two first stages in figures 7, 8 and 9. In the last stage, a large increase in volume occurs before failure. A somewhat similar change in volume has been found from strain measurements [9]. It is interesting to note that the large increase in volume seems to coincide with the transition from the second to the third stage of the damage accumulation curves based on the other measuring methods discussed earlier in this section.

3.1.4. Fracture mechanics

As mentioned in section 3.1.1 the fatigue failure is associated with development of internal micro-cracks. For metals, this development may be divided into three stages: The initiation stage during which the crack is initiated, the propagation stage during which the crack propagates, and finally the static rest failure when the remaining cross section area is so small that the crack rapidly passes it. Fracture mechanics is a branch of the solid mechanics, and is mainly treating the local conditions around the crack tip in a body.

For metals, a great deal of work had been performed on fracture mechanics in connection with fatigue, chiefly in the crack propagation stage. One of the best known crack propagation laws was suggested in 1963 for certain aluminium-alloys [10]:

$$da/dN = C\,(\Delta K)^n. \qquad (2)$$

where:

a, crack length;
N, number of cycles;
da/dN, crack propagation rate;
ΔK, stress intensity factor range ($K = \alpha\sigma_0\sqrt{a}$, $Nm^{-3/2}$)
C and n are constants;
σ_0, external stress;
α, factor between $\sqrt{\pi}$ and ~ 2.

A similar and somewhat improved formula was suggested in 1967 [11]:

$$da/dN = C\,(\Delta K)^n/((1-R)\,K_c - \Delta K), \qquad (3)$$

where:

R, the ratio of the minimum to the maximum stress intensity factor;
K_c, the critical stress intensity factor.

This way of treating the fatigue problem, based on a crack-propagation law, has been used for steel and aluminium structures, often with help of numerical techniques (*FEM*) [12].

Also for concrete activities are in progress in this area [13].

3.1.5. Plain concrete in compression

Fatigue strength is commonly defined as a fraction of the static strength that can be supported repeatedly for a given number of cycles. It can be represented by stress-fatigue life curves, referred to as Wöhler curves or S-N curves, examplified in figure 12. This is a semi-log representation with the maximum stress level S_{max} along the ordinate and the cycles to failure N_F in a logarithmic scale along the abscissa.

It should be noted that the "static" strength is usually determined from tests in which the rate of application of load may be several orders of magnitude less than the rate of loading in the fatigue tests. As the static strength of concrete is influenced quite strongly by the rate of loading and by the shape of the test specimen the resultant values of S_{max} are really nominal values related to conventional strength properties which may not truly reflect conditions in a structure under load.

Fatigue tests usually exhibit a large scatter in the number of cycles to failure at each stress level. Therefore, it is necessary to test a number of specimens at each of several stress levels in order to establish the $S-N$ curve of a particular concrete. By applying probabilistic procedures, a relationship between probability of failure (P) and number of cycles until failure can be obtained as illustrated in figure 12.

The $S-N$ curves are usually plotted for a given constant minimum stress level (S_{min} = Cte.) or for a constant ratio between the minimum and maximum stress level ($R = S_{min}/S_{max}$ = Cte.).

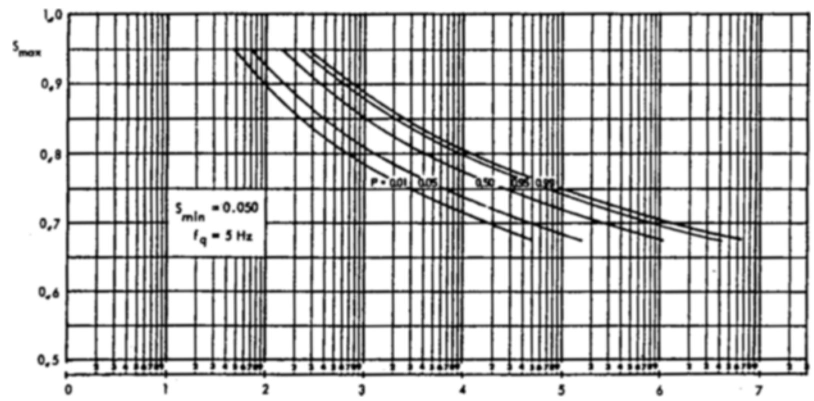


Fig. 12. — **Stress-fatigue life-probability of failure** *(S-N-P)* **curves** [9].

$S-N$ curves based on these two procedures are often erroneously assumed to be equivalent, but they are not comparable.

Using the procedure of constant R-values, tests have shown that the Wöhler curve becomes approximately a straight line.

The fatigue strength depends on the maximum as well as on the minimum stress in the cycle. This effect is commonly represented by means of a Goodman-diagram, figure 13, or a Smith-diagram, figure 14, for constant minimum (S_{min}) and constant mean ($S_m = 1/2\,(S_{min}+S_{max})$) stress level, respectively. It appears that an increase of the minimum stress level results in increased fatigue strength for a given number of cycles.

By including the ratio between the minimum and the maximum stress level as a third variable in the $S-N$ relationship, it is possible to express both the Wöhler and the Smith diagram in one equation ([14], [15]) as illustrated in figure 15:

$$S_{max} = 1 - \beta\,(1-R)\,\log_{10} N, \tag{4}$$

where:

$$R = \frac{S_{min}}{S_{max}},$$

β, material constant (values between 0.064 and 0.080 are suggested).

In contrast to steel, no fatigue limit of concrete has been found up to now. This means that no stress level is known below which the fatigue life will be infinite. Tests to 10^7 cycles have been carried out. Fatigue strengths ranging from 57 to 67% of static strength at 2×10^6 cycles and zero minimum stress level have been reported.

From recent experiments a "quasi-fatigue strength" corresponding to a range of stress of 0.4 f_c in a range of 10^{10} and 10^{11} load cycles has been suggested [16].

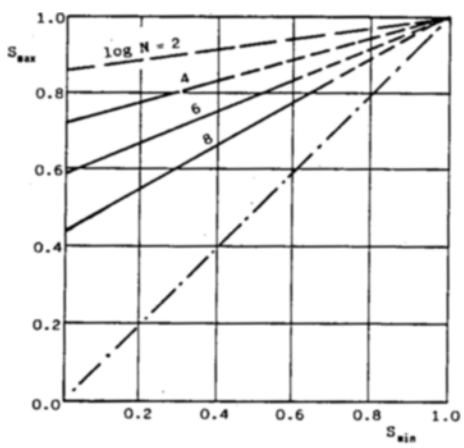


Fig. 13. — **Goodman-diagram** [14].

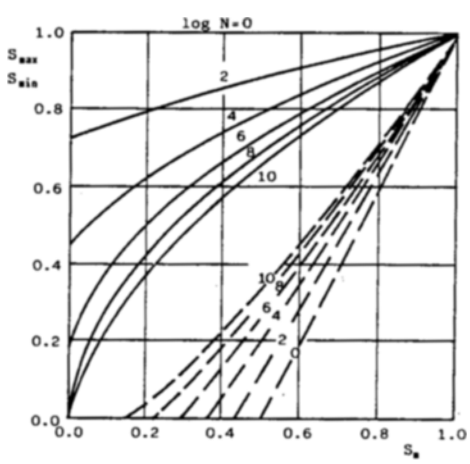


Fig. 14. — **Smith-diagram** [14].

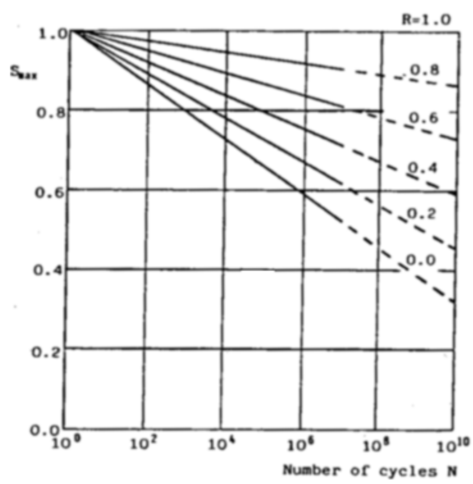


Fig. 15. — *S-N* **curves for constant** *R*-**values, where** $R = S_{min}/S_{max}$ [14].

3.1.6. Plain concrete in tension

The fatigue properties of concrete exposed to pure tensile stresses have not been studied extensively, although from the point of view of assessing the susceptibility of a structure to cracking under repeated loading the behaviour of the concrete under tension is important. One reason for the lack of tensile fatigue tests is the practical difficulty in loading a specimen in tension without introducing stress concentrations of local bending. Some limited fatigue tests on lean concrete [17] have indicated that if the fatigue strength is expressed in terms of the corresponding static strength, the results are generally similar to those obtained from flexural tests, of which a considerable number have been reported [18]. It is likely, therefore, that general observations about the effects on fatigue strength of such parameters as mix design, moisture conditions, age etc. can be applied to tensile, flexural and compressive loading.

Recent tests [19] performed on cube splitting test specimens have given valuable contributions into this area. The splitting test does not represent an entirely definite tensile stress distribution. However, distribution of stress has no effect on the results as these are set out in nondimensional form by relating the fatigue stress to the static stress. Both stresses have the same distribution, and failure in both cases is therefore initiated by the maximum tensile stress in the distribution.

These tests have shown that the susceptibility of concrete to fatigue when subjected to tensile stresses can, in principle, be described by means of the same equation as that presented for compressive stresses in equation (4) :

$$S_{r,\max} = 1 - \beta\,(1 - R)\,\log_{10} N. \qquad (5)$$

where:

$$S_{r,\max} = \frac{\sigma_{r,\max}}{f_r};$$

$\sigma_{r,\max}$, maximum cyclic splitting tensile stress;

f_r, static splitting tensile strength;

β, material constant ($= 0.0685$ is suggested).

For concrete loaded in tension (flexure) the failure strain $\varepsilon_{\max}$ after fatigue loading varies little with the number of cycles to failure [20] and amounts to approximately 0.25×10^{-3}.

As expressed by equation (1) for concrete loaded in compression $\varepsilon_{\max}$ increases with an increase of the numbers of cycles to failure and a decrease of rate of loading. Values ranging from 2.0 to 4.2×10^{-3} have been observed ([9], [21]).

3.1.7. Plain concrete in compression-tension

Only a few investigations have been done on plain concrete exposed to cyclic compression-tension stresses. Tests concerning this problem [22] give the Smith diagram in figure 16, which in the region with only compressive stresses correspond practically to the Smith diagram for centrical compressive loading.

The fatigue reaction of concrete subjected to stress reversals is also investigated with a cube splitting test [23] which is precompressed in the splitting plane. Tests are performed on 150 and 200 mm³. The scatter in results presented in a S-N-diagram, figure 17, is considerable. However, it can be stated that several specimens have not been influenced by stress reversals and perform as if they were loaded with tensile stress only with $R = 0$. This becomes more apparent for 200 mm³ with more precise load application than 150 mm³.

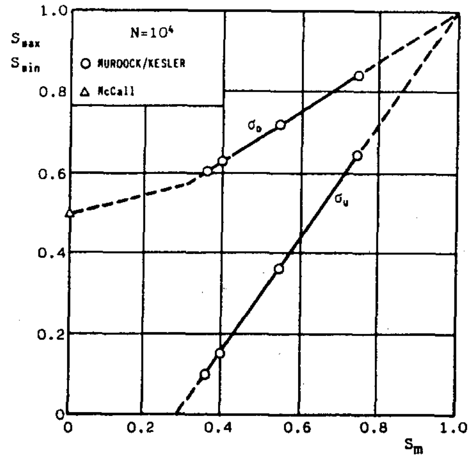


Fig. 16. — Smith-diagram (flexural stresses) [22].

3.1.8. Variability of test results

Because of the substantial scatter in test results a statistical evaluation of fatigue experiments is mandatory. In most instances log N_F follows a normal distribution ([9], [6]), however, the standard deviation increases with decreasing stress level [24]. McCall [24] suggested a method to estimate S-N-P relationships from experimental data where P is the probability of failure for a given combination of S and N. Figure 12 in section 3.1.5 shows a typical S-N-P relationship based on this method [9].

Recent tests ([9], [25]) have shown that the scatter in log N can be explained from the scatter in static compressive strength upon which the stress level S is based. This is illustrated in figure 18 where the standard deviation in

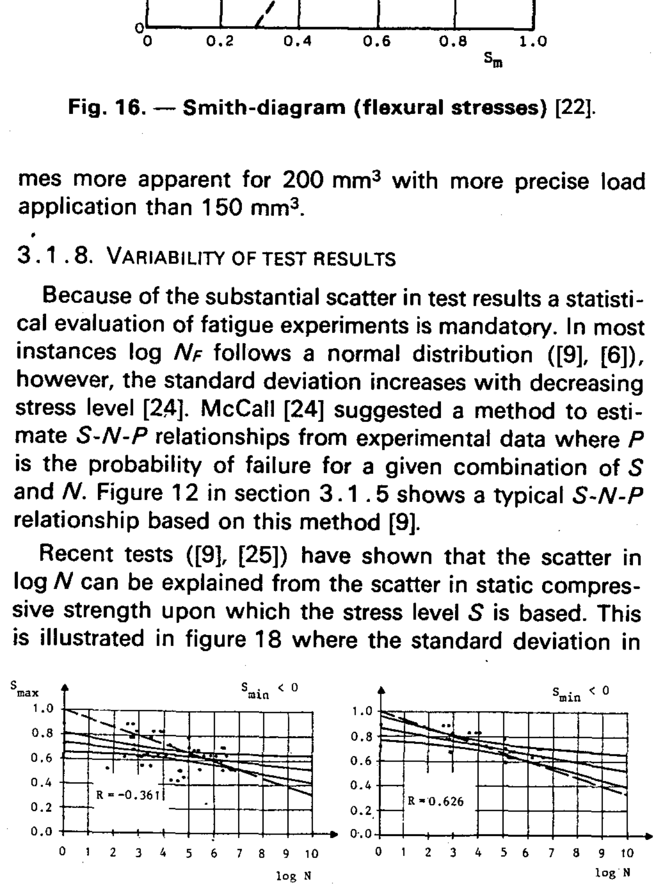


Fig. 17. — S-N-diagram for both 150 mm cubes and 200 mm cubes left and only 200 mm cubes right subjected to stress reversals. In the diagrams comparison is made with equation (5) (dotted line), which is determined for $R = 0$ [23].

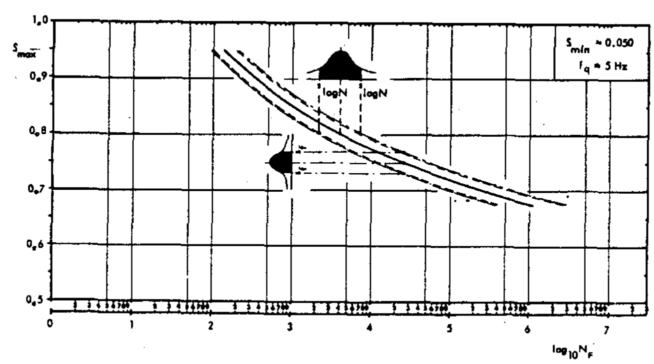


Fig. 18. — Scatter in fatigue life compared with scatter in static strength [9].

log N is compared with the standard deviation in S in compressive tests. Similar observations have been made in flexural fatigue tests [18].

3.1.9. Composition and quality of concrete

The effect of these factors is not clear. However, variables such as water-cement ratio, cement content, amount of entrained air, type of aggregates, curing conditions and age at loading do not seem to influence the fatigue strength when the fatigue strength is expressed in terms of static strength ([14], [27]). Flexural fatigue tests on three different types of concrete at ages of up to 5 years and subjected to different methods of curing gave almost identical results when expressed in terms of the corresponding static strengths [18]. These experiences are based on fatigue tests on normal concrete qualities, i.e. static strength up to 60 MPa. It is not known whether fatigue tests on higher strength concrete have been carried out.

3.1.10. Stress rate, frequency of loading

The effect of the frequency of loading has been studied by several investigators. A common conclusion is that frequency of loading between 50 and 900 cycles per minute (c.p.m.) has little effect on fatigue strength, provided that the maximum stress level is less than about 75% of the static strength [26]. At higher stress levels the fatigue strength decreases with decreasing frequency of loading ([9], [25], [24], [28]).

In compressive fatigue tests on concrete prisms [29] varying the loading rate from 0.5 to 50 MPa/s resulted in a tenfold increase in the mean fatigue life for stresses above about 75% of the static compressive strength. This suggests that accelerated fatigue tests on concrete structures may give an overestimate of their true fatigue life under loading rates which may occur in service, particularly for high cyclic stresses.

Testing at a high frequency can cause undesirable heating and it has been found that for reinforced concrete tested at 600 c.p.m. temperature rises of up to 8°C can occur [30]. However, it is not clear whether this is due to an interaction between ribbed reinforcement and the concrete, or to hysteretic action at a crack in the concrete. Either way, tests at high frequencies must be interpreted with care.

The effect of rest periods has not been studied in connection with compression fatigue. In fatigue tests of plain concrete in flexure, repeated rest periods up to 5 minutes after each block of 4,500 loading cycles, seem to raise the fatigue strength, i.e. recovery occurs during the rest period [20]. In a limited series of flexural tests in which a rest of up to 2 seconds was imposed after each loading cycle [31] there were apparent reductions in the mean fatigue life at a particular stress level but high scatter made the differences statistically insignificant.

3.1.11. Strengthening effects

The fatigue strength of concrete is defined as a fraction of the static strength that it can support repeatedly for a given number of cycles. It must be emphasized, however, that the subsequent static strength of a specimen, which has undergone a given number of cycles, cannot be used to quantify the amount of damage which has been imposed. A number of tests ([6], [7], [9], [21], [32], [33]) have shown that a specimen exposed to cyclic loading for a given cycle ratio (e.g. $N_1/N_F=0.2$) exhibits an increased strength if it is tested statically. But if the cyclic load is allowed to continue at the initial stress level, i.e. at a stress level which in reality is relatively lower than the initial one because of increased static strength, the specimen will fail when the remaining cycle ratio ($N_2/N_F=0.8$) is reached.

Increased static strength up to about 15% has been reported. The following possible explanations of the observed strengthening effects have been given [6]:

– The strength increase may be associated with the temperature rise observed as energy is dissipated in the form of heat. This would tend to cause slightly greater maturity of the specimen compared with the controls and would also increase the loss of capillary moisture resulting in slightly greater strength.

– Comparison with work on concrete under sustained compressive stress, in which slightly increased rigidity and strength were attributed to the loss of gel moisture, suggests a similar explanation for the increase of strength after repeated loading.

– The release of residual stresses due to repeated loads has also been suggested as a hypothesis for the strengthening effects.

– Strain hardening effects have been suggested as an explanation of this increased static strength.

3.1.12. Effect of moisture conditions

The effect of moisture conditions is not clear. Tests seem to indicate, however, that different moisture conditions affect the fatigue strength in the same proportion as the static strength. Flexural fatigue tests on small concrete beams which had been subjected to varying periods of drying after immersion in water [34] have shown significant differences in modulus of rupture and fatigue performance but when expressed as a proportion of the flexural strength the fatigue strength remained sensibly constant. Similar conclusions were reached from comparative tests in which specimens were saturated, surface dry or oven-dried after prolonged immersion. Differences in performance due to different moisture conditions are probably a function of moisture gradients within the test specimens, which may result in relatively high initial surface strains and possibly in local shrinkage cracks.

3.1.13. Effect of confining pressure

The fatigue strength seems to be affected by lateral confining pressure. Figure 19 shows some results from fatigue tests of sealed cylindrical specimens with and without lateral confining hydrostatic pressure applied to the circumference of the specimen [35].

It can be observed that the pressure seems to prolong the fatigue life considerably. The effect seems to be dependent on the maximum stress level of the fatigue load. It should be noted, however, that the test results given in figure 19 contain few cycles at high stress levels.

The study of this type of loading in connection with fatigue is only in the early stages and further research into this area is needed.

3.1.14. Eccentrical loading, stress gradient

To simulate the compression zone of a beam, load has been applied eccentrically in fatigue tests of concrete prisms. Figure 20 shows the results of tests [36] on 100 × 150 × 300 mm prisms under repeated compressive stresses and three different stress gradients. The stress level, S_{max}, was defined as the ratio of the extreme fiber stress to the static compressive strength.

Expressed in these terms, it can be seen that the fatigue strength is increased by eccentrical loads. But, if the stress level of the fatigue load is expressed in terms of static strength by corresponding eccentricity, the three *S-N* curves in figure 20 will practically coincide, as shown by the stippled curve in figure 20 for $e = 25.4$ mm. Thus, the static and fatigue strengths are affected by the eccentricity in the same proportion [37].

3.1.15. Loading of different waveforms

In most fatigue tests, a sinusoidal repeated loading is applied. This harmonic pattern corresponds well to that of wind and wave loadings in practical design. But special structures, such as machine foundations etc., may be exposed to other loading patterns. Tests have been done with both triangular and rectangular waveforms. Figure 21 shows some test results from an investigation with three different loading patterns on 150 x 150 x 500 mm prism [38]. 12 specimens were tested in each series and the mean values are indicated in the figure. It seems that the rectangular waveform causes failure after a shorter number of cycles than the sinusoidal one, while a greater number of triangular cycles can be sustained.

3.1.16. Deformation characteristics

The stress-strain curve varies with the number of load repetitions, changing from concave towards the strain axis to a straight line and further to a convex form as illustrated in figure 22. The degree of the convexity is an indication of how near the concrete is to failure. For lower stress levels, where fatigue failure does not take place during a finite number of cycles, the stress-strain curve seems to remain straight.

Generally, it can be said that the total strain gradually increases with increased number of cycles, figure 9. The strain development consists of three different stages: A rapid increase from 0 to about 10% of total life, a uniform increase from 10 to about 80% and then a rapid increase until failure, if failure takes place.

Recent tests [9] have shown that, at each stress level, the variation of the total strain seems to be independent of the number of cycles to failure provided that the duration of the test is less than a few hours. In tests of longer duration total strain increases faster.

As discussed in section 3.1.3, tests [9] have indicated that the maximum longitudinal strain variation by cyclic loading consists of two components, figure 23:

$$\varepsilon_{max} = \varepsilon_e + \varepsilon_t, \qquad (1)$$

where ε_e is a function of consumed endurance life and ε_t is a function of the loading time.

The variation of ε_e with the cycle ratio (N/N_F) is independent of the number of cycles to failure at each stress level. For different maximum stress levels ε_e can be expressed as a function of a secant modulus, and it seems to be independent of the minimum stress level.

ε_t can be expressed as a function of a characteristic stress level of the cyclic loading, figure 24.

Also by variable amplitude (random) loading the total maximum strain variation can be expressed by equation (1).

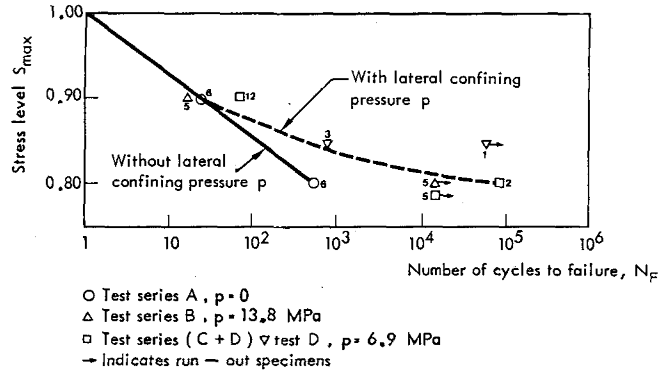


Fig. 19. — *S-N* relationship for specimens with and without lateral confining pressure [35].

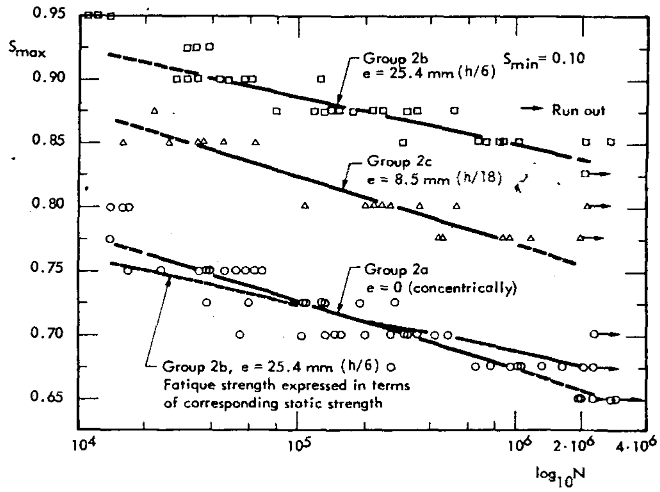


Fig. 20. — Influence of stress gradient on fatigue strength ([36, [37]).

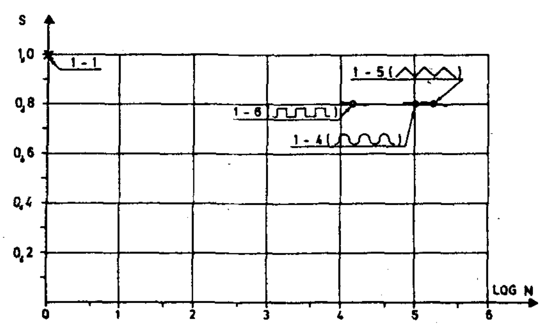


Fig. 21. — Test readings plotted in a Wöhler diagram [38].

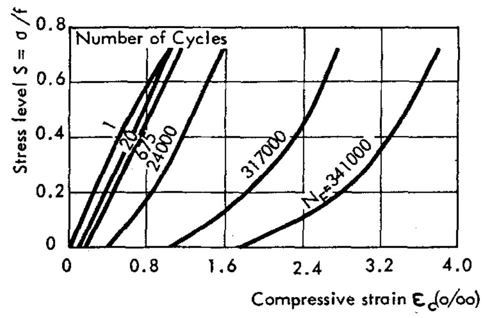


Fig. 22. — Effect of repeated load on concrete strain (loading branch) [9].

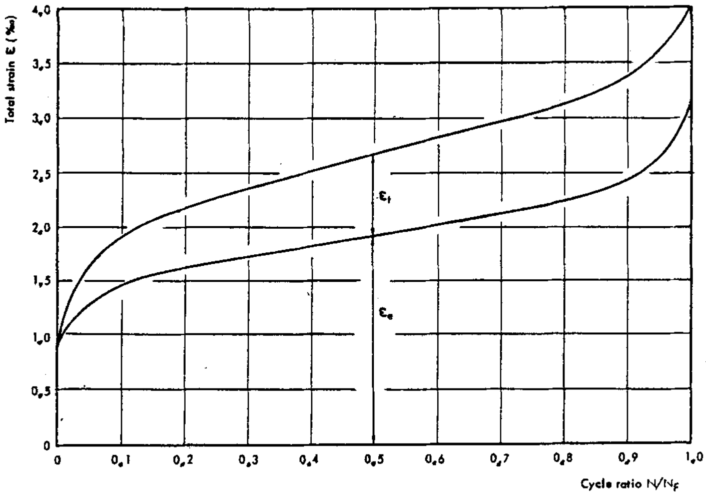


Fig. 23. — In tests [9] of long duration the total strain consists of two components, ε_e and ε_t.

The elastic strain also increases progressively with the number of load repetitions. This is shown in figure 25 by the reduction in the secant modulus of elasticity with an increase in the percentage of the fatigue life. On the other hand, "run-out"-specimens, i.e. specimens tested at low stress levels where fatigue failure is not reached, do not seem to exhibit any reduction in the secant modulus [6].

The lateral strain is also affected by the progress of cyclic loading; the Poisson's ratio seems to increase with the number of cycles for all specimens which fail in fatigue [7].

3.1.17. VARIABLE STRESS LEVELS

Most studies on the fatigue characteristics of concrete have been restricted to determining the influence of constant amplitude loading. In actual structures, however, the stress cycles vary greatly in both magnitude, number and order.

A number of tests have been done with multi-stage loading to study the validity of the Palmgren-Miner (PM) hypothesis. The PM hypothesis is very simple because it supposes that damage accumulates linearly with the number of cycles applied at a particular load level. The failure criterion is written as:

$$\sum_{i=1}^{k} \left(\frac{N_i}{N_{Fi}} \right) = 1.0, \qquad (6)$$

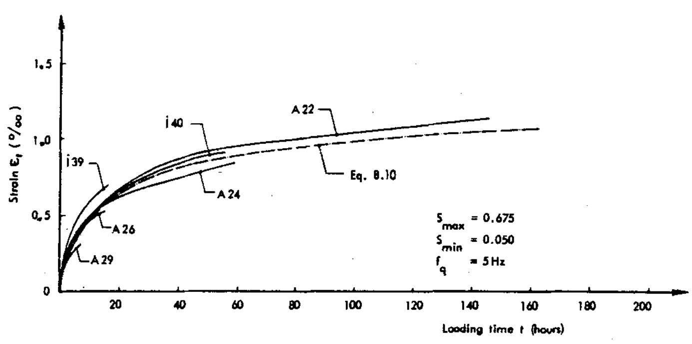


Fig. 24. — Variation of time-dependent strain ε_t under cyclic loading [9].

where:

N_i, number of cycles applied at stress level i;

N_{Fi}, number of cycles to failure at stress level i.

The hypothesis was first suggested by Palmgren [39] in 1924 for ball-bearings and was used by Miner [40] in 1945 in tests with notched aluminium specimens.

From multi-stage block loading tests both conservative and unsafe predictions of the PM hypothesis have been reported ([9], [7], [8], [25], [21], [41]).

Recent tests ([9], [42]) with random loading, approximating more closely to service loading conditions, have shown that variable amplitude loading seems to be more damaging than predicted by the PM hypothesis. There exists sequence-effects which the PM hypothesis does not take into account.

Table I shows a summary of test results from an investigation where 100 x 250 mm cylinders were tested under compression for different loading histograms [9]. Each series consisted of 30 specimens. Two main different loading histograms were used: One where the minimum stress level was kept constant ($S_{min}=0.05$) and one where the mean stress level was kept constant throughout ($S_m=0.50$). These loading histograms could be modified by omitting small amplitudes or by truncating large ones to investigate the influence of these cycles. All tests were performed with a frequency of 300 c.p.m.

The test results given in table I show that the presence of small amplitudes in a loading histogram seems to reduce the sequence-effects and increase the cycle ratio at failure.

Based on test results a modified PM hypothesis has been suggested by introducing an interaction factor ω. The failure criterion can be expressed as:

$$\frac{1}{\omega} \Sigma \frac{N_i}{N_{Fi}} = 1,$$

or:

$$\Sigma \frac{N_i}{N_{Fi}} = \omega,$$

where ω is a function of loading parameters.

From variable amplitude tests two main features of the loading histogram which influenced the fatigue life have been identified:

(*a*) The average minimum stress level $\bar{S}_{min}$ (*fig.* 26).

A lower $\bar{S}_{min}$ seems to reduce the cycle ratio at failure.

With a given loading histogram, the simple empirical curve in figure 27 may be used to evaluate the factor ω (or M) corresponding to that particular histogram. Together with a conventional $S-N$ curve and linear summation of cycle ratios, a more accurate endurance prediction than that from the *PM* hypothesis should be provided.

Loading histograms from wind and wave loadings will generally contain a great number of small amplitudes. They will increase the ratio ($\bar{S}_{min}/S_c$) and, if figure 27 is correct, ω will be close to 1.0 for such loading histograms.

The empirical relationship in figure 27 is not valid for multi-stage constant amplitude loading. It should also be emphasized that the factor ω may be influenced by other parameters, e.g. other types of random loading, loading frequency, etc.

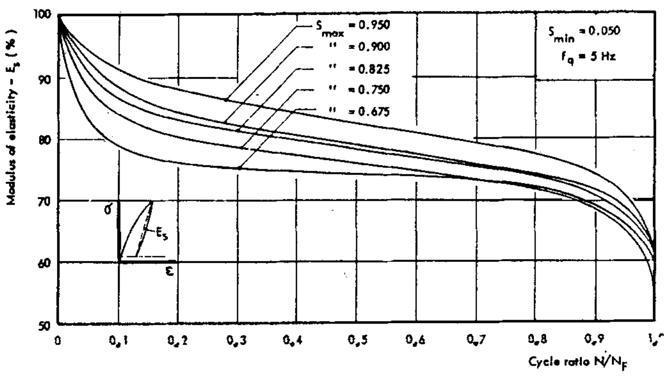


Fig. 25. — Percentage reduction of the secant modulus with the cycle ratio. Mean curves for different stress levels [9].

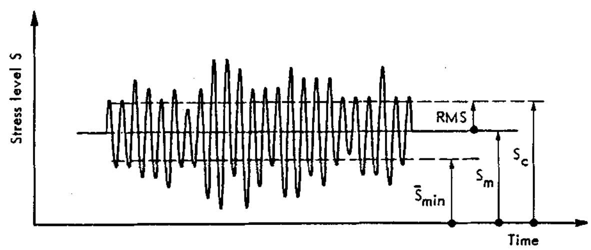


Fig. 26. — Loading parameters, S_{min} and S_c, which seem to influence the fatigue life [9].

TABLE I

TEST RESULTS FROM VARIABLE AMPLITUDE LOADING.

LOADING HISTOGRAM	MEAN MINER-SUM	90% CONFIDENCE INTERVAL FOR MEAN M
Model 2, unmodified	0,59	0,43 - 0,81
Model 2, small ampl. omitted	0,28	0,21 - 0,36
Model 3, unmodified	0,53	0,39 - 0,72
Model 3, small ampl. omitted	0,39	0,29 - 0,53
Model 3, large ampl. truncated	0,30	0,22 - 0,40
Model 1, unmodified	0,84	0,64 - 1,11
Model 1, small ampl. omitted	0,75	0,52 - 1,06

3.2. Fatigue of reinforcing steel

3.2.1. Mechanism of fatigue

As with fatigue of plain concrete, the fatigue of reinforcing steel consists primarily in the formation and propagation of micro-cracks. The formation of micro-cracks are often associated with points of stress concentration in the material. These stress raisers may be flaws present in the material (usually at the surface), sites of damage caused by use of the reinforcement (mechanical damage) or caused by discontinuities in the geometry of the structure (stress raisers at badly designed ribs) [43].

In general, the fatigue process involves two stages [43]; an initiation phase in which a small local point of failure develops to a sufficient size to form the start of a growing fatigue crack, and secondly, a propagation phase during which the crack grows to such proportions that structural failure eventually occurs.

(b) The characteristic stress level S_c (*fig.* 26). S_c is strongly influenced by the number of small amplitudes in a histogram, i. e. a higher S_c means few small amplitudes in the loading histogram and reduced cycle ratio at failure.

The factor ω (equal to the Miner-sum M at failure) has been expressed as a function of the ratio $(\bar{S}_{min}/S_c)$. Mean

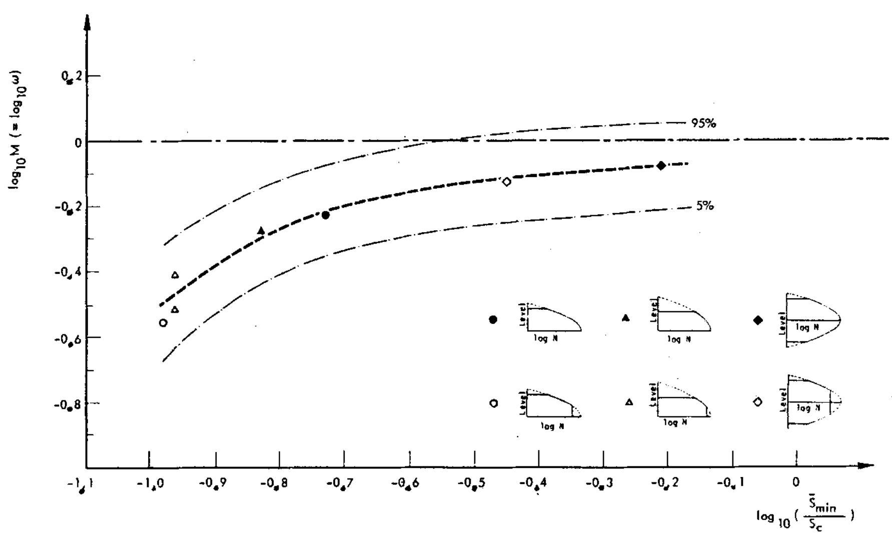


Fig. 27. — Empirical relationship between loading parameters and the factor ω (= Miner-sum at failure) [9].

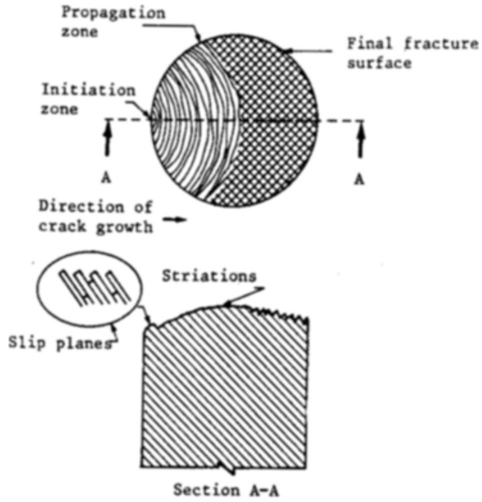


Fig. 28. — Fatigue failure surface in a reinforcing bar [43].

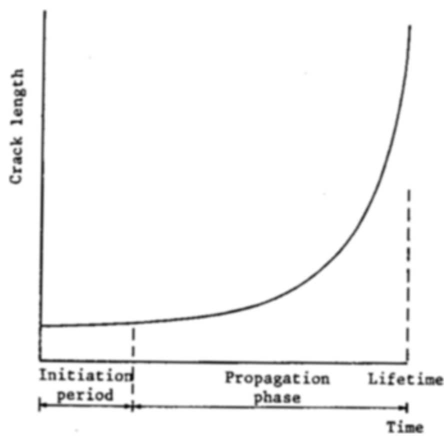


Fig. 29. — Crack length *v.* **Lifetime of a section** [43].

values of ($\overline{S}_{min}/S_c$) and M for the different loading histograms, summarized in table I, are plotted in a logarithmic diagram in figure 27. 90% confidence interval for M is indicated.

Figure 28 shows the basic features of the surface of a reinforcing bar that has failed in fatigue. The direction of crack growth is from left to right.

The first stage of the failure process is in the development of the crack from an initial flaw and its slow initiation. With a very smooth surface this stage is associated with slip planes from the stress cycle producing an irregular surface profile on the metal leading to the formation of micro-cracks. The most common site for formation of microcracks during cyclic loading are inclusions or surface flaws, as described above. The fatigue strength of a material is in this way related to surface inhomogeneities and material properties such as grain size.

In propagation stage of fatigue the surface microcrack grows at an increasing rate until the cross-sectional area of the bar is so reduced that it cannot support the load and failure occurs.

Figure 29 indicates the way in which crack growth accelerates towards the fatigue failure of a section.

The relative periods of initiation and propagation vary, depending on the material and environment, but the propagation phase always shows a slow initial period of growth continuing exponentially to failure. For most of the life of the component, the crack length is therefore small.

3.2.2. FATIGUE STRENGTH

For reinforcing steel, the fatigue strength is defined by the stress-range (σ_{max}-σ_{min}) that it can support repeatedly for a given number of cycles. As illustrated in figure 30, the $S-N$ curve differs from that of concrete in one essential way in that steel has a fatigue limit of practical significance. Thus, the $S-N$ curve becomes horizontal after a certain number of cycles, usually 1-2 mill. It is practical to divide the $S-N$ curve in two regions: The finite life region characterized by the fatigue life at a given stress level, and the long life region characterized by the fatigue limit.

The mean stress, σ_m, affects the fatigue strength to the extent approximately indicated by a modified Goodman diagram with a straight line envelope [44], figure 31.

The effect of increasing the mean stress is to reduce the allowable stress range for a given number of cycles to failure. The introduction of a compressive phase in the cycling produces a disproportionate effect and endurances are longer for a given stress range.

In studies of the effects of mean stress on the behaviour of Unisteel 60, it was shown that an increase in mean stress from 159 N/mm^2 caused a reduction of about 40 N/mm^2 in the stress range to give 10^6 cycles to failure, *see* figure 32.

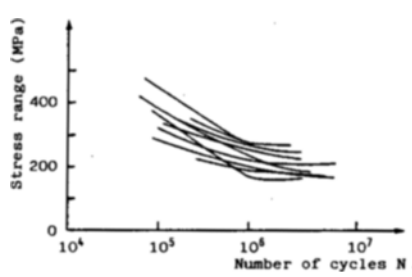


Fig. 30. — Compilation of ***S-N*** **curves for deformed bars from various investigations** [44].

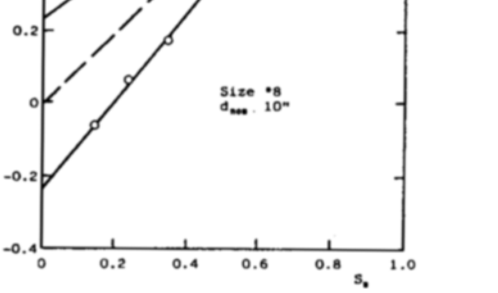


Fig. 31. — Modified Goodman-diagram for reinforcing bar [44].

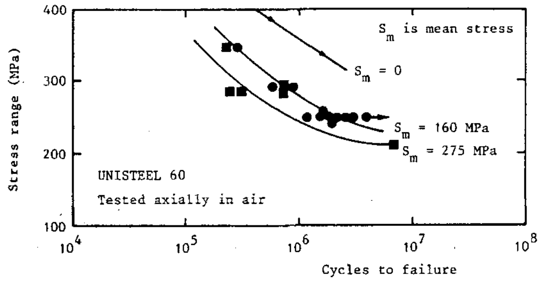


Fig. 32. — Effect of mean stress [45].

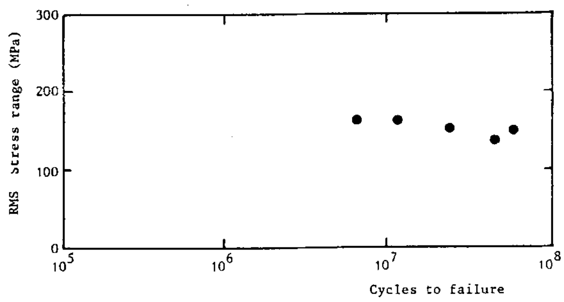


Fig. 33. — Narrow band random loading [48].

Effects due to mean stress are sometimes overlooked because during recent years there has been considerable publicity given to fatigue of welded joints in plated structures. Here residual stresses of yield stress magnitude develop in the vicinity of the welds and it has been found that as a result stress range is the dominating parameter and mean stress is comparatively innocuous. Although conditions are different in reinforcement bars, whether continuous or butt welded, it is convenient to consider fatigue strength in relation to the stress range. However, effects due to differing mean stresses can confuse comparisons between data from different investigations and it is necessary to make due allowance.

Investigations of the effects of frequency in general, indicate that there is unlikely to be an effect for reinforcement tested up to 150 Hz. The investigation of rest periods does not seem to affect the fatigue strength ([46], [47]).

3.2.3. Variable stress ranges

By block loading there are certain systematic deviations from the linear cumulative damage theory. If low stresses are applied in the beginning of the test, usually $\Sigma n/N$ becomes > 1, and when high stresses are applied first $\Sigma n/N$ usually becomes < 1. When high and low stresses are mixed the theory gives $\Sigma n/N$ closer to 1, but gives systematically unsafe results at higher stress levels [37].

Little work has been done on random fatigue of reinforcing steel. In a recent investigation [48], however, narrow band random loading spectra were used for axial tests on 16 mm Tor Bars. The tests were run at 150 Hz. In figure 33 the random loading data are expressed in terms of RMS (root mean square) stress ranges and compared with values calculated from constant amplitude data. Two representations were used for the calculations; the mean analysis and the mean -2 standard deviations. It was found that the calculated endurances are conservative in relation to the experimental values.

3.2.4. Material properties

For specimens with transverse ribs, as most reinforcing bars, the ribs will act as stress-raisers. These stress concentrations may be responsible for the initiation of cracks and the fatigue strength will no longer be proportional to the ultimate static strength of the specimen.

Figure 34 shows a compilation of test results for deformed bars from different investigations [49] and it can be seen that the fatigue strength in most cases is rather independent of the ultimate static strength.

The reason can be both the stress concentrations caused by the transverse ribs of the bars, and decarburization of the surface zone. The effect of different ribs will be discussed later on. Decarburization of the surface of the specimen may produce an effect similar to that of stress concentrations. This means that the surface zone due to lower carbon content is weaker than the rest of the material, and therefore fatigue cracks can be initiated at lower stresses. In polished machined specimens this effect is eliminated since the surface zone is cut away, but in reinforcing bars it may be considerable.

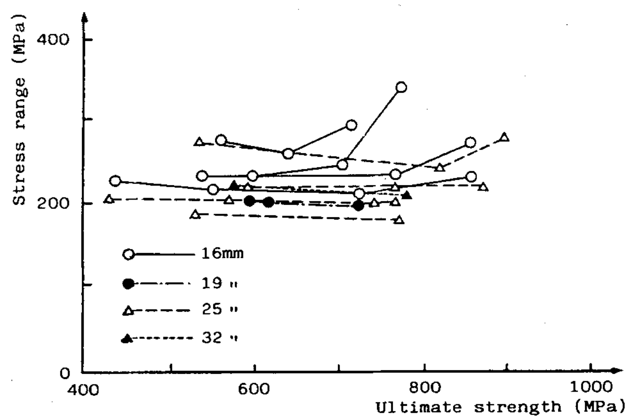


Fig. 34. — Fatigue limits for deformed bars from different investigations as a function of ultimate strength and diameter [49].

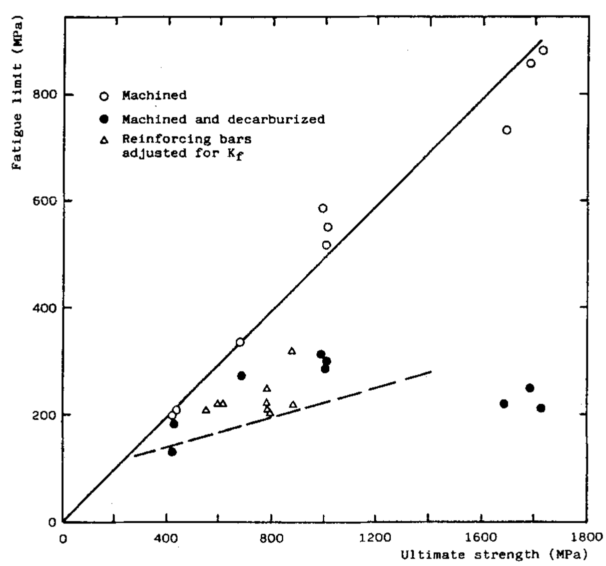


Fig. 35. — Effect of decarburization on fatigue limit [49].

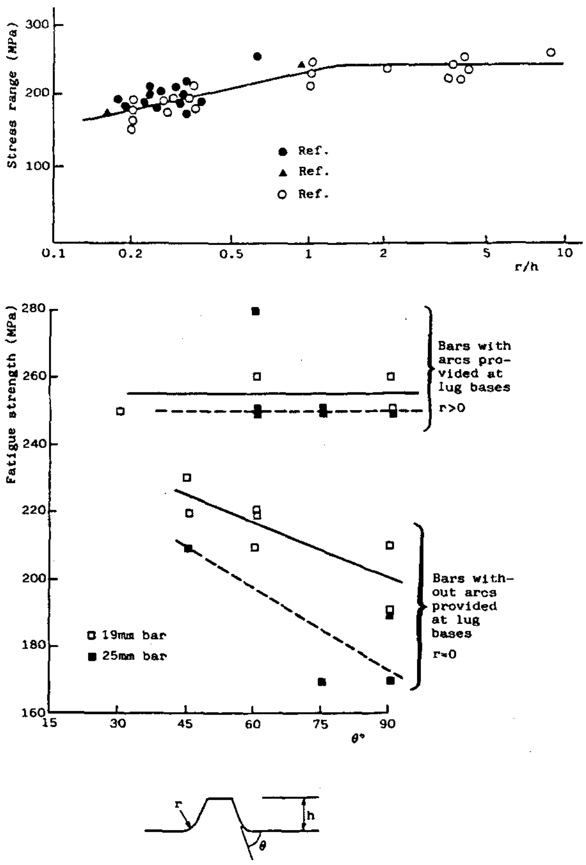


Fig. 36. — Effect of shape of ribs on fatigue and deformed bars. The effect of size is also seen in the lower diagram [51].

This is demonstrated in figure 35 where fatigue strength values for deformed reinforcing bars, multiplied by theoretical stress concentration factors, are compared with values for un-notched specimens with and without a decarburized surface zone [49]. The values for deformed bars, for which the effect of stress concentrations is compensated, agree well with the values for un-notched specimens with a decarburized surface zone. This implies that decarburization is a contributory cause for the fatigue strength of deformed reinforcing bars not to be proportional to the static ultimate strength.

3.2.5. Surface geometry

As mentioned above transverse ribs cause stress concentrations which reduce the fatigue strength compared to that of plain bars. Theoretical calculations of stress concentration factors have given values of 1.5 to 2.0 for normally shaped ribs. Many attempts have been made to establish relationships between fatigue strength and rib shape ([49], [50], [51]). Some results are shown in figure 36. It is obvious that the shape of the ribs has a considerable influence.

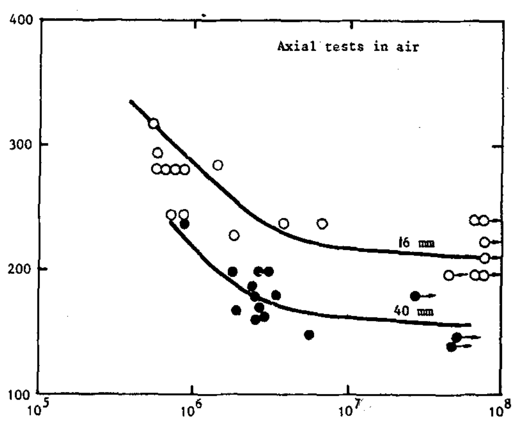


Fig. 37. — Effect of diameter for Torbar [45].

3.2.6. Bar size

The dimensions of reinforcement as well as concrete can be expected to influence the fatigue strength of the reinforcement in a bent structure. The stress gradient across a reinforcing bar is usually small, but increases with increased bar diameter and reduced depth of cross section. Increased bar diameter in itself also means a reduced fatigue strength ([45], [48]) as illustrated in figure 37.

This is partly due to statistical size effect and partly due to increase of grain size with diameter [52]. However, the effect of the stress gradient seems to be negligible, according to tests of beams with the same bar diameter but different effective depths [53].

3.2.7. Naked and embedded bars

Reinforcing bars embedded in concrete beams are likely to be affected by some factors which might influence the fatigue strength compared to that of free bars in tensile tests:

– Stress gradient due to the curvature of the beam;

– Local curvature peaks at cracks (*fig.* 38);

– Concrete pressure on transverse ribs (*fig.* 38);

– Friction effect (abrasion) because of movements between reinforcement and concrete near cracks;

– Failure in embedded bars always occur at a concrete crack because of the stress peak there.

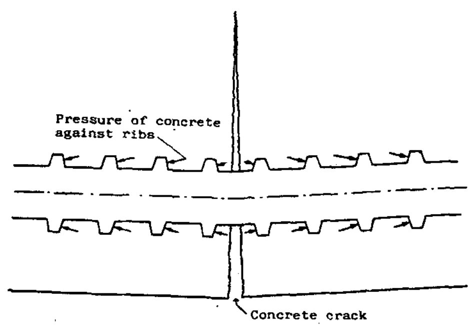


Fig. 38. — Local curvature peak in reinforcement at concrete crack; transmission of bond stresses through pressure of concrete against transverse ribs of reinforcing bars.

As mentioned above the effect of the overall stress gradient is usually negligible. However, it might be considerable under certain conditions. Thus, in one investigation it was found that the orientation of the longitudinal ribs of the reinforcing bars could have a significant effect in beam test; in beams with the longitudinal ribs in the vertical (=bending) plane the fatigue strength was around 40% lower than when these ribs were in the horizontal plane [54]. This was explained by the fact that for the type of bar tested most fatigue cracks are initiated in the points where the transverse and longitudinal ribs meet, and when the latter were located in the vertical plane these points were subjected to the somewhat higher stress in the bottom of the bar.

The influence of local curvature peaks or abrasion at cracks has not been directly verified in tests. Such influence ought to be greater the wider the cracks are, i. e. the poorer the bond. It has been shown in beam tests that good bond is beneficial for the fatigue strength [55]. This is so because friction effects are avoided.

The pressure of concrete against the transverse ribs should give tensile stresses at the base of the ribs, which add to the stress concentrations due to the change of cross section and the tensile stress in the bar. However, such an effect has not been verified in tests.

It is well established in tests that fatigue failures in reinforcing bars in beams occur at concrete cracks. This gives a purely statistical effect, in that the location of cracks has little probability to coincide with weak points of the bar. On the other hand, in a tensile test failure always occurs in the weakest point of the free length of the bar.

The overall effect of embedment in concrete is a matter of disagreement between different authors. It seems that negative effects have dominated in some investigations, and positive effects in others. The only certain conclusion is that the scatter band ought to be widened in both directions in beam tests compared to tensile tests on bare bars.

3.2.8. Bending

Those who have studied the influence of bending have come to unanimous results, namely that bending will reduce the fatigue strength of deformed bars ([45], [56]).

The reduction is a function of the pin diameter (*D*) and the bar diameter *(d)*. The following reductions have been reported from tests ([57], [58], [59]) on hot rolled 26 mm bars bent through an angle of 45 deg.:

$D/d = 15$, 16-22%,
$D/d = 10$, 22-41%,
$D/d = \ 5$, 52-68%.

It appears that the reduction is marked when D/d is reduced from 10 to 5. For D/d greater than 25, bent bars show no reduction in fatigue strength compared with the straight ones.

The reduction seems to increase with reduced bar size ([57] à [61]). This is illustrated in figure 39 for bar diameters of 16 and 26 mm, respectively.

The main factors which have been attributed to the reduced fatigue strength of bent bars are:

– The effect of cold drawing,

– The effect of residual tensile stresses on the inner side of the bend,

– The effect of transverse stresses from the concrete in the bending area,

– Damage to the ribs caused by the bending tools.

3.2.9. Welding

Welded joints in reinforcement bars, whether cross welded or butt welded, and tack welding of stirrups lower the fatigue strength. There is, however, some question about the extent of the reduction [51].

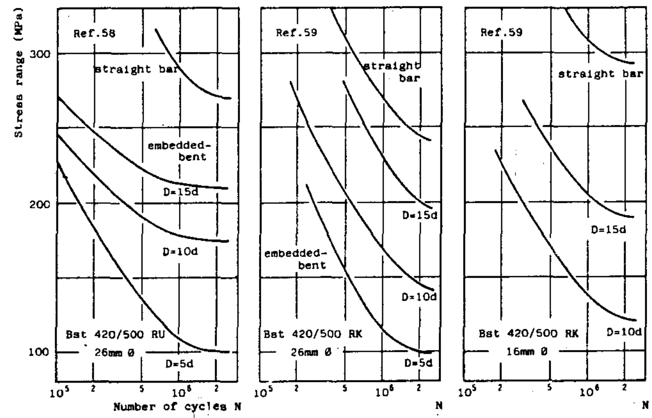


Fig. 39. — Fatigue strength of bars, straight and embedded reinforcing bars, 16 and 28 mm Ø ($\sigma_{max} = 0.8 . \sigma_{0,2}$ [57].

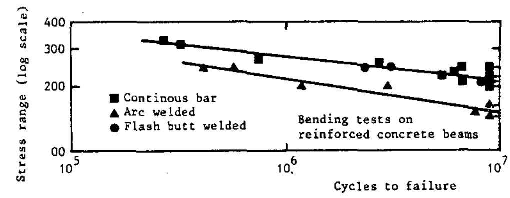


Fig. 40. — Effects of type of butt welded joint for Torbar [62].

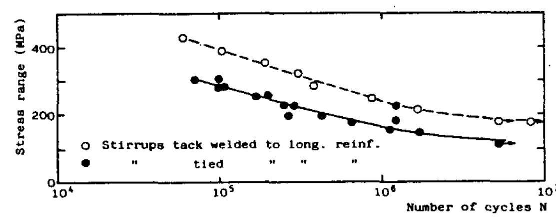


Fig. 41. — Effect of tack welding stirrups to longitudinal reinforcements [54].

In uniaxial tests on different types of butt joint, strength of 40-50% of those for plain specimens at about 10^6 cycles has been reported ([5], [62]).

For tests on reinforced beams welding seems to cause a much smaller effect [62] as indicated in figure 40. It appears that the fatigue strength of flash butt welds is apparently the same as for plain bars whereas arc welded joints have a strength reduction of 15% at 2×10^6 cycles.

There have been several investigations of the effect of tack welded stirrups and it has been found that there are substantial reductions in strength [54]. This can be seen in figure 41 where the effect of tack welded stirrups to the longitudinal reinforcement in beams is indicated. Reductions of about 35% for endurances in the range 10^6 to 5×10^6 cycles have been reported.

From the above it can be concluded that welded joints cause greater reductions in strength for axial tests than for bending of reinforced beams. This may be due to the fact that cracks in the concrete usually develop in the vicinity of transverse ribs so that maximum stress occurs at a vulnerable part of the reinforcement although not necessarily at the worst defect. For butt welded joints the superficial surface profile is comparatively rounded in relation to the ribs so that cracks in the concrete do not necessarily coincide with the part of the joint containing the worst

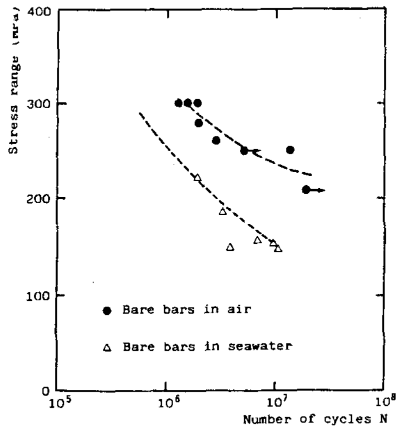


Fig. 42. — Effect of seawater on fatigue strength of reinforcing bars [43].

fatigue features. In consequence welded joints can have a better fatigue strength when tested in concrete beams than in air.

3.2.10. Effect of seawater

There have been a number of recent studies of corrosion fatigue in relation to concrete of offshore structures. Figure 42 shows some test results [43] of bare bars tested in air and in seawater at 145 Hz. It appears that at 10^7 cycles there was a reduction of about 35% in fatigue strength due to corrosion.

The effect of encasement in concrete on the corrosion will be discussed later in connection with structural members in section 3.3.6.

3.2.11. Prestressing steel

In fully prestressed members, where cracking does not occur under service conditions, the stress variations in the reinforcement are so small that fatigue of the reinforcement and its anchorages can hardly become critical. Even under the worst conditions the stress variations do not exceed 5% of the ultimate strength of the steel, which generally is not enough to cause fatigue failure [63]. In partially prestressed members, where cracking is accepted under service conditions, the stress variation is greater. The following figures have been mentioned: 12, 18 and 24% of ultimate strength for crack width of 0.1, 0.2 and 0.3 mm respectively. With such stress ranges fatigue cannot be ignored, since they exceed the fatigue limit of many types of prestressing tendons. Some figures can be mentioned, which are compiled from various investigations [26]. The figures indicate the following stress range values at 2 million cycles and a minimum stress of 60% of the ultimate strength. For individual plain wires 12 to 26% of ultimate strength, i. e. around 220 to 470 MPa (assuming the ultimate strength to be 1800 MPa).

For strands 9 to 14% of ultimate strength, i. e. around 160 to 250 MPa.

For bars at least 10% of ultimate strength, i. e. at least 100 MPa.

The stress range at low minimum stresses is hardly of any interest since the minimum stress in a prestressed member is practically equal to the prestress after losses, i. e. usually 50 to 70% of the ultimate strength.

In unbonded post-tensioned systems the stress variations are transmitted directly to the anchorages. Although most anchorages can develop the full static strength of the tendon, they can seldom develop its fatigue strength. Thus, it is the fatigue strength of the anchorages that has to be considered, not that of the tendon. In figure 43 test results for different types of wire anchorages are shown. All these anchorages could sustain the full static strength of the tendons, but most of them had less than half their fatigue strength.

Furthermore, tests like these, performed on single wires, are likely to overestimate the fatigue strength of multi-wire or multi-strand anchorages, since higher local stresses are possible due to for instance angularity between anchorage and tendon.

In members with bonded tendons stress variations in cracked sections are usually not transmitted to the anchorages, and in this case the fatigue strength of the tendons, not the anchorages, is important. On the other hand, the increase in stress in the tendons due to cracking is much greater in bonded members than in unbonded members. In the latter case the non-prestressed reinforcement, which must be present to control cracking, will take the major part of the stress increase, and hence the stress variation from the fatigue load; in the former case the tendons will have high local stress variations at the cracks even if there is non-prestressed reinforcement.

Another phenomenon may appear in prestressed members with bonded tendons with a short anchorage length. For strands, the anchorage between the strand and the concrete hardly fails completely, but some movement along the anchorage length can occur at rather low loads (if the anchorage length is short). This movement causes the prestress in the strand in the cracked section to drop almost completely, whereafter the stress range in the strand increases very much. This is catastrophic for the fatigue strength of the prestressed member since the fatigue limit of the stress range in the strand is rather low.

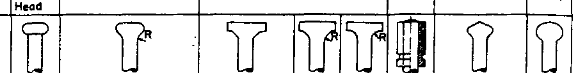
Tests performed on concrete sleeper [64] and bond tests [13] indicate a fatigue limit of the stress range in the strand of only 5% of the ultimate stress at 2 million cycles.

3.3. Fatigue of structural members

3.3.1. General

The fatigue properties of reinforced concrete structures is closely related to the properties of the component materials, concrete and steel, and the connection between

Country	America	Japan								Switzerland		Britain
Anchorage Type	Conical Button Head	Button Head			Hammer Head				Swagged Nut	Conical Head		Conical Head
Mark	Series 3	B5	B7	B8	T5	T7A	T7B	T7C	E8	Button Heads	Conical	
Wire Diameter, in.	0.250	0.197	0.276	0.315	0.197	0.276	0.276	0.276	0.315	0.276	0.276	0.276 0.250
Radius, R / Diameter	-	0.25	0.29	0.28	0	0	0.49	0.89	-	-	-	-



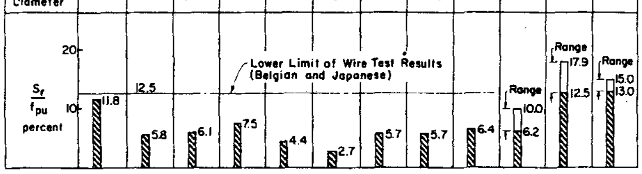


Fig. 43. — Fatigue strength of different types of wire anchorage at two million cycles [26].

them. Thus, for underreinforced members failing in bending the fatigue properties are directly related to those of the reinforcement, with due consideration of the factors already discussed in section 3.2.

However, in the case of compression failures in overreinforced members or shear and bond failures it is not as simple, since the stress conditions are more complicated.

Another fact that must be kept in mind is that the static failure and the fatigue failure of a structural member can be of quite different nature especially for prestressed members. For example, in a series of tests of prestressed railway sleepers [64] it was found that the static failure was caused by a concrete compression failure in some cases, by a bond failure in some cases and by a shear failure in some cases. However, during fatigue loading all of the specimen failed due to tension failure of the prestressed strands.

The fact that several failure modes occurred in the static tests shows that the sleepers were optimally designed (against static load). The fact that another failure mode came about in the fatigue tests was due to the load level, which in the fatigue tests were rather low (compared with static tests). This resulted in different stress distribution in the sleepers. Fatigue in the concrete was not likely to occur since the concrete compression zone was rather large at the small fatigue loads, and because of that the stress range in the concrete became small. These conditions were the more pronounced since the structure was prestressed.

Fatigue failure because of shear did not occur, because the prestressing load gave so large compression stresses in the "critical shear area", that shear cracks did not form. Since no shear cracks formed, bond failure never occurred for the fatigue loading. Usually the minimum stress (or the mean stress) in prestressed reinforcement is rather high at working loadings (because of the prestressing), compared with unstressed reinforcement. The absolute stress range is rather large too when cracking occurs, although it becomes proportionately small compared with the ultimate stress (high strength steels).

From a general point of view the above indicates that, concerning fatigue in prestressed concrete structures, fatigue in prestressed reinforcement often can be expected to be the most critical cause of failure, pre-supposed that cracking is allowed.

Some failure modes and effect of various parameters will be discussed in more detail below.

3.3.2. Compression failure

The fatigue properties of concrete in uniaxial compression are well known, but can they be applied to the compression zone of a beam, where the stress gradient influences the strength and deformation properties as under static load?

In one investigation a direct comparison was made between overreinforced beams and uniaxially loaded prisms with the same dimensions and concrete quality as the compression zone of the beams [64]. The beams, which failed due to fatigue of the compression zone, could sustain 70% of their static ultimate load for 10^7 cycles, whereas the same value for the prisms was about 60%.

In another investigation the conditions in a beam were simulated by loading concrete prisms eccentrically [36]. It was found that the fatigue strength, expressed as the stress in the most compressed edge, computed from the stress-strain curve in static uniaxial compression, increased with the eccentricity. However, the relative fatigue strength in terms of the total load on the prisms was not affected by the eccentricity. Thus, the static and fatigue strengths were affected by the eccentricity in the same proportion as discussed in section 3.1.14. The conclusion from these two investigations is that it is on the safe side to apply fatigue values for uniaxial compression to the compression zone of beams.

3.3.3. Failure of tensile reinforcement in bending

As far as fatigue failure of tensile reinforcement in bending is concerned, it should be mentioned that the relative fatigue strength can be very low. For tensile reinforcement consisting of high strength deformed bars values as low as 44% of the static ultimate load have been obtained [76] in beams reinforced with deformed bars having a yield strength of 600 MPa as shown in figure 44.

In figure 44 the test results for beams in which the tensile reinforcement failed in fatigue are plotted. It appears that grades Ks40 and Ks60 both give the same reduction in fatigue resistance.

3.3.4. Shear failure

Shear failure appears seldom in practice under static load. Such failures, like compression failures, are avoided due to their brittle nature, if this is practical design. The "desirable" type of failure is yielding of the longitudinal reinforcement, which gives a ductile type of failure. This is usually considered in design regulations, making brittle failures more unlikely under static loads.

However, in some cases beams have failed in shear under cyclic loading although similar beams failed in bending under static load [65]. In some investigations remarkably low fatigue strengths have been obtained for beams failing in shear or bond. Therefore, these types of failure require special attention. Figure 45 shows some results from different investigations (the fatigue strength is related to the static ultimate loads of similar beams and corresponds to about 10^6 cycles and minimum loads of about 10%).

The fatigue strength in shear varies a great deal between the different investigations. For beams without shear reinforcement, (*fig.* 46), a comparison with the tensile strength of plain concrete seems logical.

Such a comparison is shown in figure 47. The shear fatigue values are on an average clearly lower than those for bending of plain concrete. On the other hand it should be observed that all the shear values are within the scatter band of the tension values.

For beams with shear reinforcement, (*fig.* 48), the fatigue properties are of course to a great extent depending on the properties of the reinforcement. However, the application of results from tensile tests on straight bars is not as straightforward as for bending reinforcement.

Firstly, the stress history in the stirrups might be very complicated even in constant load level tests (*fig.* 49). Secondly, failures often occur at the lower bends of the stirrups [66], and we know that bending can have a very negative effect on the fatigue strength. Thirdly, the shear cracks usually cross the stirrups at skew angles, which causes additional stresses due to dowelling and other effects that might impair the fatigue strength.

3.3.5. Bond failure

The effect and fatigue of bond is affected by factors such as the type of reinforcing bar, the geometry of the

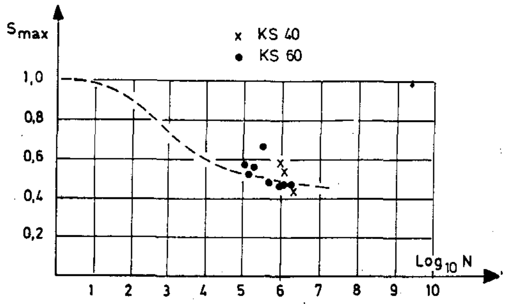


Fig. 44. — Wöhler diagram for the tensile reinforcement [76].

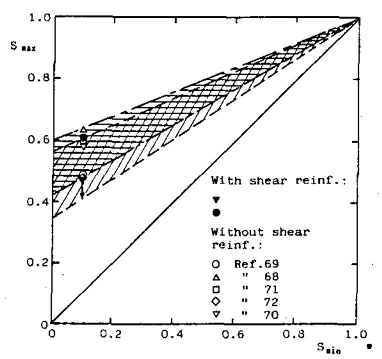


Fig. 45. — Shear fatigue failure in beams with and without shear reinforcement [66].

surrounding concrete, existence of confinement, transverse pressure or tension, load time function and the maximum load level in comparison with the ultimate static load. Three types of failure can be distinguished:

(1) Failure due to excess of shear strength between the bar and the concrete (chemical or friction type).

(2) If the shear strength between the bar and the concrete is high enough the principal tensile stress will crack the concrete and the bond forces will become directed outward from the bar. The surrounding concrete may be split away by the pressure exerted by the anchored reinforcing bar. Hence, the tensile strength of concrete in an ultimate splitting failure pattern determines the bond strength.

(3) If the splitting resistance of the surrounding concrete is high enough, then bond failure in the case of a deformed bar will also occur as shear failure in the concrete along the perimeter of the bar lugs. This bond strength, which is the maximum possible, is determined by the shear strength of the concrete.

The second type of bond action and failure is the most common in concrete structures reinforced with deformed bars, while the third type is seldom reached.

The reduction in bond strength due to fatigue must accordingly depend on how each of these three strengths is affected by alternating loads. In most cases, where fatigue of bond has been tested, the distinction of these three types of stresses has not been made. A complication is that the bond stress usually is not evenly distributed along the bond length. A redistribution of bond stresses takes place during the loading sequence and the final failure may have a "Zipper character".

The first type of failure is very little investigated in fatigue partly because of the various types of chemical bonds and mainly because they are very seldom used in practice. However, for those types of chemical bonds where only the concrete determines the strength, it can be assumed that the same reduction in bond strength because of failure is applicable as has been stated for concrete strength under compression and tension ([14], [67], [68], [69]).

One study [70], which concerns bond of smooth bars, suggests that the anchorage resistance only is accomplished by surface irregularities of the steel. When casting, the concrete does not grow into irregularities completely. This causes the contact zone between the steel and the concrete to be much softer compared to concrete in complete fitting. When loading a pull-out test for the first time, comparatively large nonreversible slipping will occur and the movement causes the steel and the concrete to change and fit closer to each other. When after unloading, loading is performed a second time, the contact zone behaves much stiffer, due to the newly improved fitting in this direction, *see* figure 50. When increasing the load above a certain level, the anchorage resistance will change to a more "frictional" type of resistance at about 0.01 mm. The final failure occurs at a slip between the steel and the concrete of about 0.5 mm.

The second type of bond failure is most usual for deformed reinforcing bars with ordinary concrete covers and has been analysed by a number of investigators ([7], [81]).

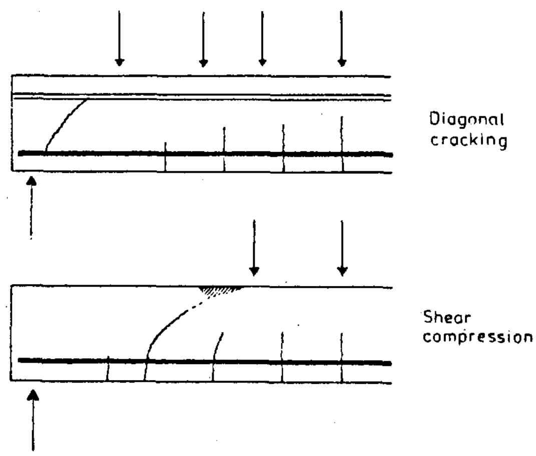


Fig. 46. — Possible shear fatigue failures in beams without shear reinforcement.

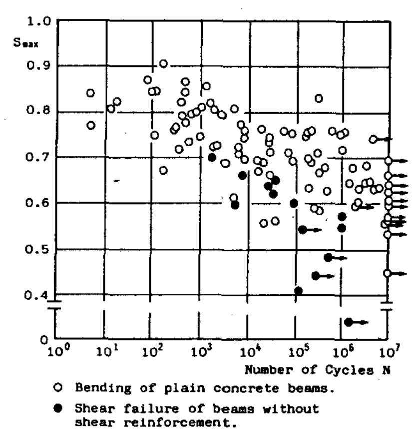


Fig. 47. — Comparison between fatigue strength in shear of beams without shear reinforcement and fatigue strength in bending of plain concrete [66].

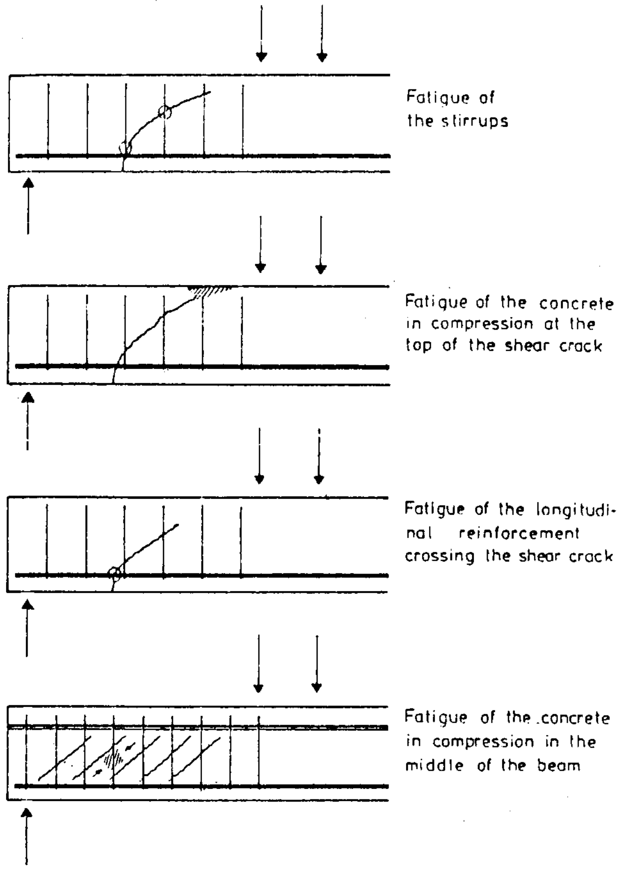


Fig. 48. — Possible shear fatigue failure in beams with stirrups.

Splitting is also the usual cause of failure for the anchors of prestressing steel tendons. At the splitting the tensile strength of concrete is exceeded in an ultimate failure pattern. If the loads are repeated the splitting strength of concrete, which means the same as the tensile strength, is affected by fatigue in the same degree as it is stated to

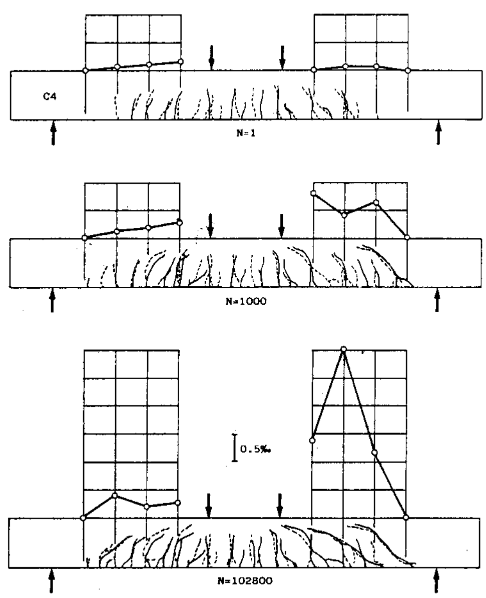


Fig. 49. — Development of cracks and strains in stirrups under cyclic loading [66].

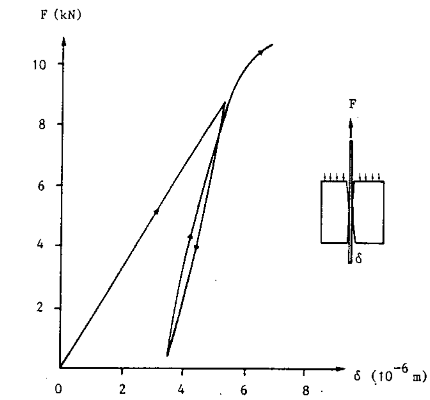


Fig. 50. — Load-slip relation for a pull-out specimen with short anchorage length (50 mm) [70].

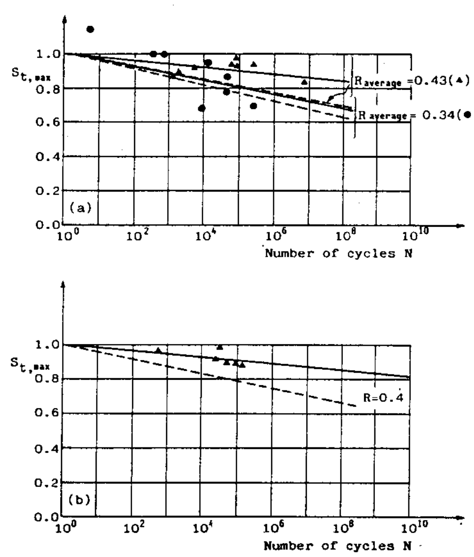


Fig. 51. — Wöhler-curves for the tensile strength of concrete in ultimate failure pattern for spliced bars. Dotted lines represent Wöhler curves for concrete under uniaxial compression [76].

be for concrete under compression or tension according to many observations ([76], [82]). Disturbances in this fatigue relation of bond may however appear.

The deterioration by fatigue is usually determined in relation to the static strength of the same material or structure. For an anchored reinforcing bar the bond stress picture at static failure in the surrounding concrete is not always the same as at the maximum load level in the beginning of the fatigue loading sequence. During the load repetitions, there takes place a redistribution of stresses. The deformations due to cyclic creep of concrete under repeated load are extensive according to many observations ([38], [82]-[85]). Because of the cyclic creep, it can be assumed that after a certain number of load cycles, a stress distribution corresponding to that at the static failure is obtained.

This development in the anchorage zone means that the stress levels and amplitudes change during the cyclic loading sequence. The fatigue damage on the concrete during the stress changing period is probably less than what is obtained during the same time period with the final failure stresses.

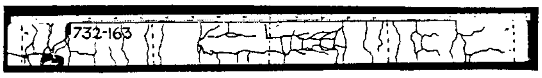


Fig. 52. — Longitudinal cracks in the concrete cover following a tensile overlap splice in a beam [76].

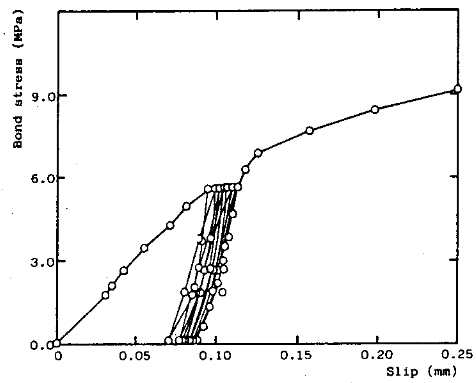


Fig. 53. — Experimental bond-slip curves [88] for 16 dia. hot-rolled high-yield (410) deformed bar under repeated loading (0.5, 6 MPa). Concrete cover = 35 mm.

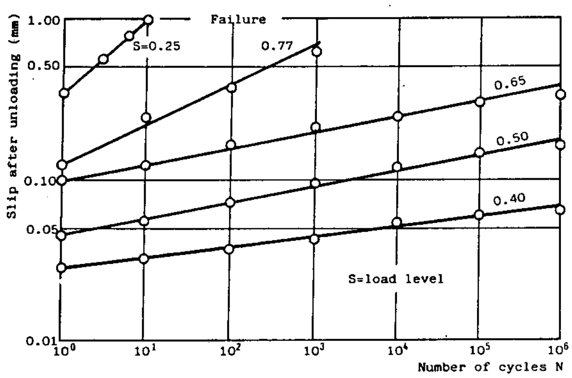


Fig. 54. — Increase of slip at the free bar end during cyclic load as a function of the number of load reversals *n* (f_c = 23.5 MPa, d = 14 mm, bond length 3 d) [75].

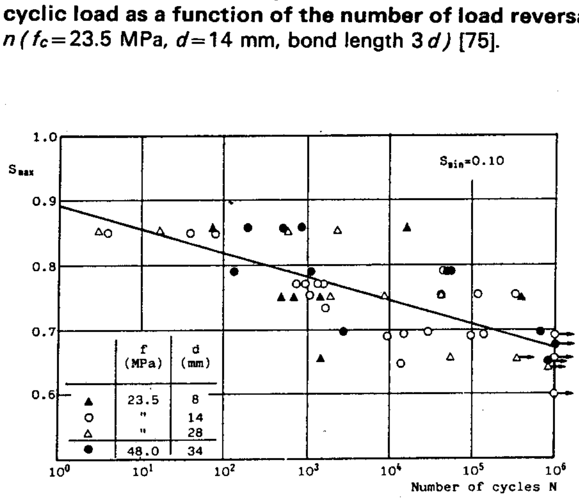


Fig. 55. — Influence of the bond stress max $\tilde{\tau}_1$ under load (related to the static bond strength $\tilde{\tau}_{1u}$) at constant lower load on the number of load cycles until failure [75].

The consequence of the changing stresses is that the anchorage zone resists a greater number of load cycles compared with a situation, where the final stresses had acted from the beginning of loading sequence, figure 51.

The changing of stress picture under cyclic load may be very radical and imply a sliding between quite different anchorage stress stages. For instance while the final static and also fatigue failure take place by exhaustion of the tensile strength of concrete in an ultimate failure pattern with completely developed longitudinal cover cracks, figure 51, the stresses at the beginning of the cyclic load sequence can be quite different because no longitudinal cover cracks may exist. During the repeated loading cracks in the longitudinal cover open up due to fatigue and the stress pattern at ultimate failure is developed due to the cyclic creep of the concrete.

The fatigue resistance of the anchorage can be determined from the final stress pattern as if it had existed all the time. The fatigue deterioration of the concrete can be assumed to be the same for compression and tension. The fatigue resistance for the bond represented in form of a Wöhler curve, figure 51, will, then, be somewhat more favourable than for concrete under uniaxial compression.

The third and most resistant type of bond fatigue failure—shear failure when the splitting resistance of the surrounding concrete is large—is investigated in pull out tests with long and short bond lengths ([82], [83], [86], [87], [88]). It is concluded that repeated bond force of the same sign has about the same influence on the deformation and failure of the concrete as it is observed for uniaxially compression loaded concrete. Repeated load, which does not lead to fatigue failure, does not decrease the static failure bond load. The cyclic creep in the bond zone of the concrete leads to a redistribution of bond stresses. The bond-stress-slip curves under repeated loading are characterized by residual slip at zero load and hysteresis loops formed by the loading and unloading paths. The hysteresis loops shift by a small amount during each cycle, but this shift tends to diminish with the number of cycles applied, figure 53.

The slip increases with the intensity of fatigue load in comparison with the ultimate static load, figure 54. The Wöhler curve, however, for varying $R = S_{min}/S_{max}$ value, determined in pull out tests with short bond length is shown in figure 55.

At the present stage of knowledge, it must be concluded that the bond is deteriorated by fatigue in the same way and degree as it is observed for concrete under compression and tension. The lapses of deformations qualitatively seem to be identical. However, it is necessary to identify the real bond stresses and to base the calculation of the bond resistance and the influence of fatigue on them.

Confinement in an anchorage zone, due to compressive stresses in the concrete reducing the tensile splitting stresses in the bond region, can reduce the deterioration by fatigue. If the combination of compressive and tensile stresses lead to alternating stresses under repeated loads, it is not clear if the stress reversals have a negative influence on the fatigue of concrete.

A confining effect in the anchorage zone can be obtained from surrounding reinforcement in the form of stirrups, transverse bars in the concrete cover or spirals, figure 56. The surrounding reinforcement strengthens the bond reinforcing the longitudinal cover cracks and contributing in the ultimate splitting failure patterns with a stress, which corresponds to the deformation of concrete when failure starts. The stresses in the transverse reinforcement in longitudinal cracks can be high but no fatigue failures have been reported of this reinforcement.

3.3.6. STRESS RATE, FREQUENCY OF LOADING

Tests performed on underreinforced beams in air [89] have shown that the test cycle frequency in the range from 0.17 to 5 Hz has small effect on the fatigue life as illustrated in figure 57. This was to be expected from corresponding tests separately on steel and concrete.

Tests performed on corresponding beams [89] in sea water indicate that high stress-cycle frequencies seem to reduce the fatigue life compared with test at low stress-cycle frequencies. The presence of time dependent corrosion fatigue and water pumping may change the effect of the loading frequency. Corrosion fatigue will be discussed in section 3.3.8.

3.3.7. EFFECT OF WATER

Conflicting results have been reported about the effect of water on the fatigue strength of reinforced members.

In some tests on underreinforced beams a significant reduction in the fatigue lives because of submersion in seawater has been reported [90] as illustrated in figure 58.

Other corresponding tests have shown that the seawater does not appear to have a deleterious effect on the fatigue

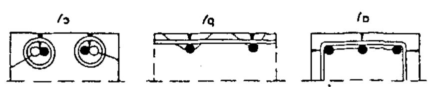


Fig. 56. — Confining transverse reinforcement in form of *(a)* stirrups *(b)* transverse bars *(c)* spirals [76].

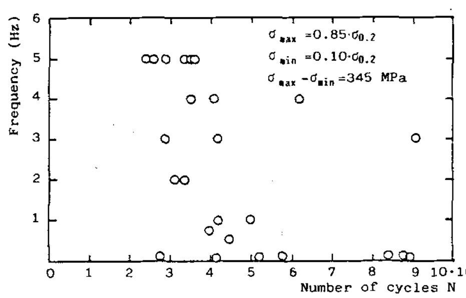


Fig. 57. — Effect of frequency of load application on fatigue life of underreinforced beams [89].

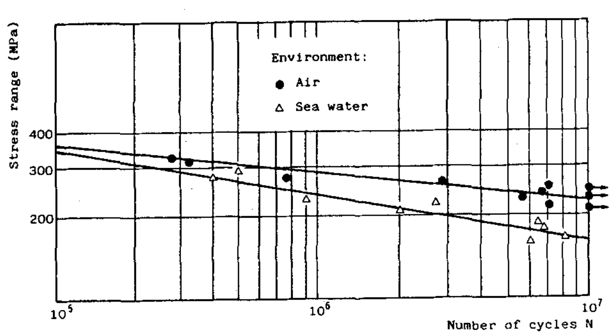


Fig. 58. — Effect of seawater on fatigue strength of under-reinforced beams [90].

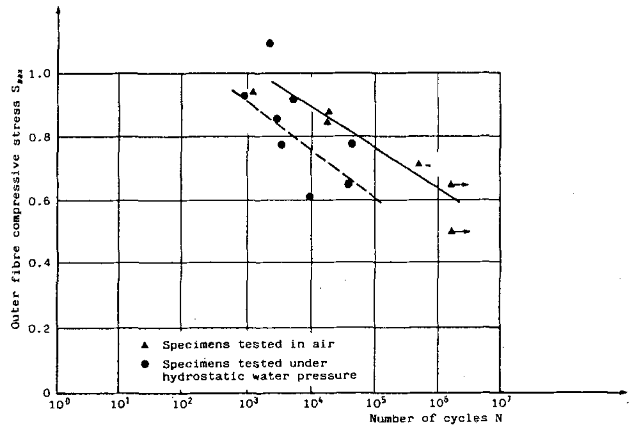


Fig. 59. — Effect of hydrostatic water pressure on fatigue strength of precracked specimens [91].

life. On the contrary, fatigue lives in fully submerged conditions may be longer in seawater than in air [89].

Tests performed with repeated flexural stresses on precracked specimens under hydraulic pressure to investigate the effect of pumping of water in and out of cracks [91] have indicated a reduction in fatigue life compared with corresponding tests in air as indicated in figure 59. A stress condition in a specimen cycling between a cracked condition and a compression level in the outer edge in a combination with water pressure seems unfavourable. The pumping effect in cracks will give rise to quite different time-stress function in the vicinity of the cracks in comparison with the time-stress function, which is expected from the external load fluctuations.

The type of cracking will, however, be an important feature because cyclic loading causes a progressive breakdown in the bond at points where the cracks reach the reinforcement in a manner likely to occur in service but not simulated by static load.

3.3.8. EFFECT OF CORROSION ON FATIGUE

As discussed in section 2.2.1, fatigue crack initiation is known to take place at areas of local stress concentration such as preexisting flaws in the material or at surface discontinuities or changes in shape. Local corrosion damage caused by various chemicals can have a drastic effect on static strength and particularly on the fatigue life. Pitting arising through the ingress of seawater and oxygen to the steel reinforcement is also likely to create initiation sites for fatigue cracks.

Tests have shown that fatigue lives of reinforced beams are reduced due to corrosion effect. This is illustrated in figure 60 where data from tests on Tor Bar reinforcement in concrete beams [90] are collated.

The figure shows tests both in air and in seawater, and it appears that the seawater environment reduces the fatigue life. In addition, it is reported that in seawater the tensile cracks tended to be wider than the cracks in corresponding beams tested in air due to pumping effects.

Figure 60 also shows that for corresponding beams tested in a seawater environment and with sacrificial anodes attached to the reinforcement, the fatigue resistance of Tor Bar is restored.

The crack width does not seem to have any significant influence on the amount of corrosion in the situation where the bar passes more or less perpendicularly through

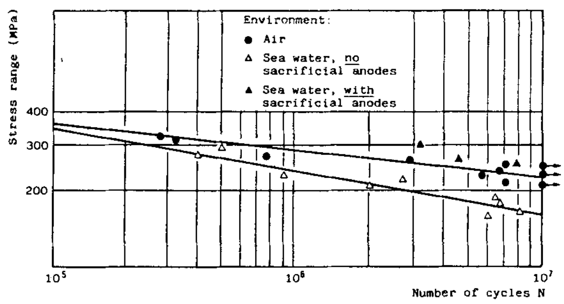


Fig. 60. — Corrosion effect on fatigue strength of reinforcing bars [90].

the crack [92]. A greater corrosion risk is likely by cracks that follow the line of bar. However, the effect of cracks along the bar is not systematically investigated.

The effect of the thickness of concrete cover on fatigue life in seawater is not quite clear [43]. Tests with 25 mm cover have shown no increase over bare steel [63] as illustrated in figure 61. Some tests with 65 mm cover have, however, showed an improvement over bare steel performance [43]. The test results might be disturbed by stress concentrations due to pumping effects.

4. SUGGESTION FOR FUTURE INVESTIGATION

4.1. Plain concrete

Considerable information of fatigue of plain concrete has been generated by experimental investigations, particularly the last decade. The relation between laboratory tests and fatigue life from 10 years to 100 years of real structures is not known. Rest periods and small repeated loads have probably a strengthening effect, but it has only been discussed qualitatively as yet.

A previous larger loading shortens the fatigue life of concrete subjected to many repeated, smaller loadings. However, we do not fully know the relation between the magnitude of the loadings and the fatigue life. In other words the Palmgren-Miner hypothesis is not quite satisfactory to predict the life of concrete under dynamic load.

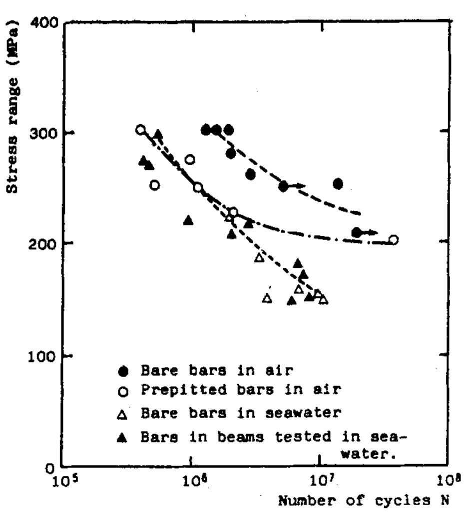


Fig. 61. — Effect of 25 mm cover on fatigue life in seawater [63].

These problems alone indicate that more research is needed to reach a reasonable level of knowledge for practical design. More specific problems can be mentioned: effect of water pressure on concrete subjected to fatigue loading, effect of aggressive environment and effect of high and low temperature.

4.2. Reinforcement

Since fatigue of steel has been studied much longer and more intensively than concrete, the fatigue life of the reinforcement can be predicted with greater certainty than that of concrete. However, more information is needed on points such as fatigue life under high and low temperatures and fatigue life in corrosive environments.

Increasing use of partial prestressing means much higher stress range in the prestressing steel than for conventional prestressing of concrete structures. Prestressing steels should therefore be tested under various conditions, especially with regard to high and low temperatures and corrosive environment.

4.3. Reinforced concrete structures

Knowledge about the fatigue strength of reinforced concrete is missing in many important areas. In comparison with the separate material properties of steel and concrete, respectively, the knowledge about interaction problems between the two materials and their performance in reinforced concrete structures is small. The knowledge of the performance during random loading is also very limited.

Almost all existing knowledge, as far as fatigue of reinforced concrete structures is concerned, consists of empirical formulae based on test series of limited generality. Theoretical analyses, where stress redistributions and progressive fracture during the failure course is modelled, are almost completely missing. Such a non-linear analysis is facilitated by a discretization in space of the structure, e. g. by using the finite element method (FEM). If a discretization also in time is performed, the FEM can be used to execute the analysis for an arbitrarily varying load. Due to the gradually decreasing costs of the computer processing investment in the future, such analyses seem to be more and more practicable.

A combination of such theoretical analyses and basic material properties estimated in experiments, seems to be the most efficient method to increase the knowledge of fatigue loaded reinforced concrete structures.

5. TERMS AND DEFINITIONS

Fatigue strength	fraction of static strength that can be supported repeatedly for a given number of cycles;
Fatigue limit,	stress level below which the fatigue life will be infinite;
S-N relationship,	a semi-log representation of stress level against number of cycles at failure;
S-N-P relationship,	*S-N* relationship where the probability of failure *P* is included as a third variable;
Cycle ratio (N/N_F),	ratio of the applied loading cycles to the number of cycles causing failure at the same stress level;

PM hypothesis, Palmgren-Miner hypothesis: $\sum_{i=1}^{k} \frac{N_i}{N_{Fi}} = 1.0$;

Level, in a loading histogram the total range of possible peaks and troughs is divided into a number of identical intervals represented by levels;

a, amplitude;

d, bar diameter;

D, damage imposed on a specimen by cyclic loading, pin diameter;

E_s, $\frac{\sigma_{max} - \sigma_{min}}{\varepsilon_{max} - \varepsilon_{min}}$ = Secant modulus of elasticity for concrete;

E_c, modulus of elasticity for concrete;

f, ultimate static compressive strength;

f_q, loading frequency (Hz);

f_r, static splitting tensile strength;

N, number of cycles;

N_F, number of cycles to failure;

P, probability of failure;

R, S_{min}/S_{max};

R.M.S., root mean square value $= \sqrt{\frac{1}{T_0}\int_0^T x^2(t)\,dt}$;

S, $\frac{\sigma}{f}$ = stress level = ratio of stress (σ) to ultimate static strength (f);

S_{max}, $\frac{\sigma_{max}}{f}$ = maximum stress level of a load cycle;

S_{min}, $\frac{\sigma_{min}}{f}$ = minimum stress level of a load cycle;

S_m, $1/2\,(S_{max}+S_{min})$ = Mean stress level of a load cycle;

S_r, $\frac{\sigma_r}{f_r}$;

S_c, characteristic stress level $= S_m + RMS$;

Stress range, $\sigma_{max} - \sigma_{min}$;

T_0, total time, i.e. the duration of the cyclic loading;

V, volume;

ΔV, volume change;

$\sigma(t)$, stress as a function of time t;

β, material constant;

ε, strain (‰);

ε_c, compressive strain by static loading;

ε_e, compressive strain dependent on consumed endurance life by cyclic loading;

ε_{max}, $\varepsilon_e + \varepsilon_t$ = total compressive strain at S_{max};

ε_{min}, total compressive strain at S_{min};

ε_t, time-dependent compressive strain by cyclic loading;

σ, stress;

σ_r, cyclic splitting tensile stress.

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RÉSUMÉ

La durée de vie des ouvrages en béton et leur comportement sous sollicitation dynamique devient un facteur important dans l'étude d'un nombre croissant de structures soumises à des sollicitations dynamiques. Pour certains ouvrages, on exige des calculs afin d'établir que l'ouvrage ne subit pas de défaillance en fatigue.

Ce rapport examine un par un les phénomènes de fatigue dans les ouvrages en béton et présente une sélection de résultats de recherche qui sont succinctement discutés.

On considère le béton non armé sous compression et sous traction, en relation avec la qualité du matériau, le taux de contrainte, les charges excentriques, le milieu et les sollicitations aléatoires.

On ne s'attarde pas sur l'acier d'armature, sujet très bien traité par ailleurs dans diverses publications.

Les composants du béton armé peuvent subir une défaillance en fatigue de diverses façons comme ils le faisaient sous charge statique. Cependant, un élément de structure qui cèderait à cause d'un moment de flexion sous charge statique pourrait fort bien subir une défaillance d'effort tranchant en fatigue.

Il est par conséquent difficile de mettre en relation cette capacité d'effort tranchant en fatigue à la capacité d'effort tranchant sous charge statique.

On applique ici au béton l'hypothèse de Palmgren-Miner et on la discute en relation avec les charges stochastiques. Cette hypothèse ne traduit pas les propriétés du béton. Cependant, avec quelques petites modifications, l'hypothèse de Palmgren-Miner s'applique au calcul en fait parce qu'on ne dispose d'aucune méthode meilleure.

Dans de nombreux cas, les déformations croissantes sous charges répétées sont plus critiques que la capacité de durée d'une structure en fatigue. Ces déformations, qui sont plus grandes pour le béton que pour l'acier, peuvent déterminer des redistributions de contraintes, ce qui peut aboutir à une fissuration imprévue et à d'importantes déformations.

Des charges dynamiques de faible amplitude avec des périodes de repos paraissent renforcer l'élément de structure soumis à des sollicitations dynamiques. Ce phénomène contribue à l'incertitude qui ressort de la comparaison entre les essais de laboratoire de courte durée et le comportement des ouvrages.