

Corrosion Fatigue in Concrete for Marine Applications

By P. D. Arthur, John C. Earl, and Trevor Hodgkiss

Synopsis: Structural concrete is widely used in marine environments, but a relatively recent development has been its use in structures such as oil production platforms, ships, wave energy devices etc., where fatigue loading can be significant. It is well known that the effect of a corrosive environment on structural steelwork is to reduce its fatigue life, and this paper describes work in progress to determine whether or not the same is true for structural concrete, both reinforced and prestressed.

Reinforced and prestressed concrete beams are being tested in unidirectional bending, and in reverse bending, in jackets containing sea-water, at slow cycling rates (about 0.17 Hz) which approximate to sea-wave frequencies. The sea-water is pH and temperature controlled and is continuously circulated from a storage tank. Control specimens are tested at higher frequencies (3 to 5 Hz) and these show the expected reduction in fatigue endurance, compared with tests in air.

However, the wave-frequency test results show that deposits are formed in the flexure cracks after 3 to 4 days of cyclic loading, and this has the effect of increasing, rather than decreasing, the fatigue lives of the beams - certainly when the bending is unidirectional. Under reverse bending this effect is not yet confirmed, although the crack-blocking is observed to take place.

Electron-microscopy of the failure surface is being utilised to establish the mechanism by which corrosion fatigue failure occurs under these conditions.

Keywords: beams (supports); bending; concretes; corrosion; cracking (fracturing); fatigue (materials); fatigue tests; marine atmospheres; offshore structures; prestressed concrete; reinforced concrete; sea water.

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INTRODUCTION

Structural concrete has been widely used for many years in marine environments but a relatively recent development has been its application to structures such as oil production platforms, ships and boats, and the many projected wave energy and other ocean-based energy devices. In all of these cases fatigue loading may be significant and it is obvious that corrosion is also a possibility. It is well known that the effect of a corrosive environment on unprotected structural steelwork is to reduce its fatigue life, (1,2) and it would be expected that the same would be true for structural concrete, both reinforced and prestressed.

At the time when the work described in this paper was started there were, unfortunately, few data available on the fatigue behaviour of concrete members in seawater. Those which were available applied only to reinforced concrete (3-5) and had yielded rather conflicting results. (1,6,7). The work had been restricted to high frequency loading conditions (3 Hz. or more) and so was of limited applicability where the fatigue loading was likely to be applied at low frequencies, as is the case with wave-loaded structures. In addition, tests at 3 Hz. or above are usually of only short duration (one day or so at high stress ranges) and thus they do not reproduce the effects which any time-dependent environmental processes such as corrosion might have on the fatigue endurance of the material.

The work described in this paper is an initial step in helping to provide a clearer understanding of the inter-relation between corrosion and fatigue in reinforced concrete in a marine

environment so that designers may be able to draw on the results to interpret, and extrapolate from, available data with increased confidence. It began with tests on reinforced concrete beams bent in one direction only, but reverse bending is now being studied. The paper presents some interesting results and suggests hypotheses to account for the effects observed.

SPECIMENS AND TEST PROCEDURES

The specimens were beams of identical overall dimensions, shear reinforcement, and cover. Beams intended for bending in one direction only were reinforced with two 10 mm dia Torbars on the tension side, with two 8 mm dia mild steel lacing bars to provide anchorage for the stirrups, while those intended for reverse bending tests had two 10 mm Torbars top and bottom. Full details are given in Figs. 1a and 1b. Details of the steel and concrete are given in Table 1, and full particulars of the method of production, curing, etc. are given in a previously published article. (8) For stress level calculations the concrete strength (cube tests) was taken as 60 N/mm^2 , and the elastic modulus of the steel as 200 kN/mm^2 . Stress levels were referred to the specified characteristic proof stress of 460 N/mm^2 .

In the main series of tests carried out up to the present beams in seawater have been subjected to ten applications of load per minute (0.17 Hz.), which is the approximate frequency of very large waves in the North Sea. A few tests were done in seawater at 5 Hz. in order to study the effect of frequency on endurance. As control tests, and for comparison with other published results (5) tests were done in air over the same frequency range (0.17 to 5 Hz.).

The central section of the beam was enclosed in a transparent plastic water jacket containing seawater. The seawater used was a synthetic type which included all the inorganic constituents of natural seawater including trace elements. Its composition is given in Table 2. Initially the seawater was drip-fed from a header tank to waste, but in later tests it was rapidly circulated and dosed to maintain a constant pH level. Its temperature was controlled in the range 18 to 21°C by means of simple combined heaters and thermostats, and it was continuously aerated in the jackets to maintain full saturation with oxygen. The beams were generally immersed in seawater almost up to their top surfaces, but in one case involving uni-directional bending the water jacket was filled only to the level of the main tension steel to study the effect of "splash-zone" corrosion.

Symmetrical two-point loading was applied, as shown in Fig.2, on a span of 2.8 metres, the shear span being 1.0 metre. Two different types of water jacket have been used and their dimensions are also given in Fig. 2. In the unidirectional bending tests on "singly reinforced" beams a minimum stress level of 10% of the specified characteristic yield strength of the reinforcement was

set, i.e. 46 N/mm^2 , and the upper stress levels varied from 92% to 40% of the characteristic strength, giving stress ranges varying from 377 to 138 N/mm^2 . The corresponding ranges of compressive stress in the extreme fibre of the concrete varied from 20 to 5 N/mm^2 . For concrete with a cube strength of 60 N/mm^2 this meant that there was little chance of fatigue failure of the concrete before the steel failed.

The initial tests in reverse bending have all been carried out with the load controls set to give a range of stress of from 281 N/mm^2 to -9 N/mm^2 in the bars on the side more highly stressed in tension, i.e. the lower bars. The corresponding stress range on the less highly stressed side is from 207 N/mm^2 to -14 N/mm^2 , so there is little chance of failure occurring on this side.

The simple fatigue machines used for the slow tests (Fig. 3b) employed a 0.75 kW hydraulic pump-set operating a single acting ram, in the case of the unidirectional bending tests, and a double-acting one for the reverse bending tests. The load was monitored by means of strain gauge load cells mounted in spherical seats attached to the ram body. Load variation was achieved by means of pressure control valves and cycle frequency controlled by a constant speed cam. The wave-form of load against time was arranged to be nearly sinusoidal by the adjustment of fine control valves operating on the ram inlet and outlet pipes, and, for the single-acting rams, the lower load was controlled by a further flow restriction in the oil return line. Seven unidirectional bending, and three reverse bending machines have been constructed. The higher frequency tests used Losenhausen servo-controlled equipment with a demountable packless ram mounted on a loading frame bolted to a hollow structural testing floor, (Fig. 3a).

During the tests measurements were made of beam deflection, variation in concrete crack width, strain in the concrete, and in some cases, strain in the main tension steel. The seawater pH and the reinforcement electrode potential were also monitored. Failure normally occurred by the fracture of one or other of the main tensile bars due to fatigue, usually followed immediately by yield of the remaining bar. In some cases it was possible to stop the test before the second bar yielded. Occasionally two bars failed simultaneously in fatigue. After the completion of a test the beam was broken up and the main bars removed for microscopic examination to study the nature of the failure surfaces and determine the frequency of other incipient fatigue cracks.

RESULTS

To date about 87 tests have been completed, 77 in unidirectional bending and 10 in reverse bending. The results are given in Tables 3a to 3e and 4a and b. They are shown graphically in Fig. 5 in which intended stress range is plotted against fatigue endurance.

1. Unidirectional Bending

a. Tests in air-- The effect of frequency of load application on fatigue life in air is shown in Fig. 4 in which all the results of unidirectional bending tests in air at an upper stress level of $0.85f_y$ are plotted. This shows that the test cycle frequency in the range from 0.17 to 5 Hz. has no noticeable effect on the endurance of beams tested in air. This result was expected but its confirmation made it possible to do the majority of the air control tests at about 5 Hz. and so save time. These tests also agreed generally with the work of Bannister (5). In the air tests failure always occurred at main flexure cracks in the concrete, and there were usually about 8 to 10 of these in the maximum bending zone. In 9 cases the failure was between the load points, and in three it was at a load point. This could represent a statistical spread throughout the 8 to 10 concrete cracks.

b. Tests in seawater -- i. Tests at 5 Hz. As shown in Table 3c the endurances at a stress range of 345 N/mm^2 were all less than 200,000 cycles. Failure occurred in all cases but one, close to midspan and within the seawater environment.

ii. Tests at 0.17 Hz. In most of the initial tests the load range applied was that calculated to produce a stress range of 345 N/mm^2 in the tension steel, and Table 3d and Fig. 5 show that the scatter in the fatigue lives of the seawater tests in the short tanks is similar to that of the tests in air (Tables 3a, 3b). Failure occurred inside the tank in only two of the short tank tests. In all the other tests it took place at the first or second crack in the concrete outside the tank, and in some of these cases seawater could be observed seeping from such cracks during part of the test period. It is presumed that this seawater reached these external cracks by some internal path.

Because the early tests did not produce consistent failures within the corrosive environment the use of tanks double the original length was adopted. Using these tanks the fatigue lives in seawater at a load range calculated to produce a stress range of 345 N/mm^2 yielded the surprising result (See Table 3e and Fig.5) that the fatigue endurances were greater than those found using the short tanks. Of the four beams which have failed to date one has done so outside the extended tank, i.e. in the shear span, where the moment is less than is required to produce the intended stress range. Two beams which failed within the environment had fatigue endurances of 8.29×10^6 cycles or greater - more than ten times that to be expected in an air test at the same intended stress range. Three tests on beams in long tanks are still running, with endurances already greater than in any air or short tank tests at the same intended stress range.

It has been observed that, during the first ten or so days of a test, a deposit builds up at each immersed crack in the concrete. This deposit increasingly inhibits crack closure on reduction of load and deflection measurements, together with observations of crack width changes, have confirmed this effect which will be considered in more detail later in the paper. The deposit consists of calcium carbonate, (Ca CO_3), and/or magnesium hydroxide, (Mg(OH)_2) and experiments carried out elsewhere have confirmed this.

Another feature of the early slow seawater tests was that the pH of the seawater within the tanks was generally higher, at 8.5 to 9.5, than that of newly prepared synthetic seawater whose pH, at 7.8 to 8.2, was typical of natural seawater. Because of the obvious importance of the crack-blocking effects mentioned above, and with the knowledge that deposition of chemical species from seawater generally becomes more prevalent as the pH rises, it was decided that all subsequent tests should be undertaken with the pH controlled to between 7.8 and 8.2. This has been done by a system of rapid circulation of seawater through the jackets from a large tank which is dosed with hydrochloric acid as necessary. Beams tested under this regime continue to show the effects of crack-blocking and increased endurances.

2. Reverse Bending

a. Tests in air -- Up to the present time a total of six reverse bending tests in air have been completed (See Table 4a) all at the same stress range. Endurances have ranged from 161,000 to 573,000 cycles, and all failures have taken place either between the load points or at a load point. These results overlap with the scatter of the air tests and fast seawater tests at the same range of stress in unidirectional bending but are generally lower.

b. Tests in seawater -- Only four results are available to date from the programme of reverse bending tests in seawater and the results are listed in Table 4b. Endurances range from 161,000 to 208,000 cycles; short tanks were used initially and one failure has taken place within the tank, the other two occurring at load points. One long tank test has been completed and the failure was within the tank. The results fall within the scatter of the air test results in Table 4a but are all at the low end of the range. Although there is evidence from crack width measurements that crack blocking is occurring, as it did in the unidirectional bending tests, there is no indication of any increase in fatigue endurance.

DISCUSSION

As is indicated in Tables 3a to 3e and 4a and b and in Fig.5 most of the tests carried out so far have been at high stress amplitudes. The first phase of the work was planned in this way to enable as many tests as possible to be completed quickly in

order to obtain early indications of the effects to be expected. In seeking useful conclusions, therefore, only these high-stress results may be drawn upon at present, and this discussion will be limited to that section of the research programme. The study of the detailed mechanism of corrosion fatigue deterioration in reinforced concrete will be left to another occasion.

1. Unidirectional Bending

The evidence from these preliminary results is that at high stress-cycle frequencies, and high stress ranges, which are conditions in which failure normally occurred in the tests after about one day, the effect of submersion in seawater was to reduce significantly the fatigue life of reinforced concrete compared with that in air. This result confirms the conclusions of Bannister (5) but conflicts with those of others (3,4). In these high frequency tests no signs of crack-blocking or reduced deflection range were observed. The reduced fatigue lives observed in this type of test are in agreement with results for unclad metals.

However, at 0.17 Hz. under conditions of high stress range (345 N/mm^2 , 49,300 psi) the seawater did not appear to give a deleterious effect on the fatigue lives of the beams. On the contrary, as the tests with the long tanks indicated, fatigue lives in the fully submerged condition may be longer in seawater than in air. This is the opposite effect to that observed in unclad metals where reduction of loading frequency progressively accentuates the deleterious effect of seawater, at least until extremely low frequencies of around 0.01 Hz. are reached.

Measurements of beam deflection range during a test yield the typical results shown in Fig. 6, and crack opening ranges show similar trends. It can be seen that from the first day these ranges start to decrease significantly and after only a few days the changes are quite marked. The fall-off continues until a state of stability is reached, after which the crack opening and deflection ranges remain virtually constant. In all tests, and particularly in the lower stress tests, a large proportion of the specimen lifetime is spent cycling under these substantially less severe conditions.

In the tests at the lower stress ranges the final crack opening displacement is much less than in the higher stress tests, but the time taken to reach the stable condition of reduced crack opening is approximately the same. This indicates that, while the effect of crack blocking on the stress range in a cycle is marked there is a limit to it, and the stress range never falls to zero, but only to a more or less constant fraction of the initial range. After about 10^6 cycles of load this fraction is approximately 0.5. This reduced stress range in the bars is the reason for the greatly extended endurance observed in the slow tests compared with those found in high-speed, and therefore short-duration, tests. In the slow tests it is clear that the deleterious effects of the

environment are greatly outweighed by the beneficial effects of the stress range reduction due to crack blocking. The results shown graphically in Fig. 7 are untypical of the majority of the tests, but they are included for interest as they demonstrate clearly the correlation between the measured parameters that indicate blocking of the cracks, crack opening, concrete strain and beam deflection. It can be seen from all three parameters that at around 700,000 cycles partial unblocking of the cracks occurred, and it is interesting to note that blocking was re-established at about 1,400,000 cycles, precisely double the time required to block the cracks initially in the period before the unblocking event. There is no evidence to suggest a reason for the unblocking taking place - the pH, for example, was controlled well within limits and no sudden dissolution could be possible. It would seem that the event was mechanical, perhaps the effect of a branching crack reaching a free surface or some other similar happening. This argument is not wholly convincing as several blocked cracks suffered unblocking, and it might be the case that all of the submerged cracks were involved. This is apparent because the concrete strain was measured at different locations from either of the crack-opening gauges and yet the measured effects show the same indications. In addition, the change in deflection was consistent with the effect being widespread and was too great to be due to the changes in only one or two cracks.

This beam failed within the corrosive environment at 2.3×10^6 cycles of a load range intended to produce a stress range of 345 N/mm^2 (49,300 psi) in the tension steel. This fatigue life implies that the stress range was very much lower than intended for the larger fraction of its life, but not so low as would apply at the first crack outside the tank. The stress history of the beam would be of this nature:

No. of cycles		0	12,000	100,000	1,000,000	2,300,000
Inside tank	$f_{\min}/f_y$	0.1	0.19	0.4	0.46	0.61
	$f_{\max}/f_y$	0.85	0.85	0.85	0.85	0.85
	Stress range/ f_y	0.75	0.66	0.45	0.39	0.24
At first crack outside tank	$f_{\min}/f_y$	-----		0.08	-----	
	$f_{\max}/f_y$	-----		0.67	-----	
	Stress range/ f_y	-----		0.59	-----	

From the above figures it is clear that it is not possible to quantify the effect on fatigue life by these methods, but it is also clear that the reduction in the severity of the stress conditions, which is of the nature shown in Fig. 8, is significant and begins early in the life of the beam. The beam (Test No. 98)

failed within the tank whereas most of the others in that test series failed outside the tank. It may be that this was due to the unblocking which occurred during the test and which did not necessarily occur in the other specimens.

The interpretation of the concrete strain readings requires consideration of the changes in shrinkage strain which take place during the duration of a test. The ages of the beams at the start of test varied from about 40 days to, in some cases, as much as 200 days, so considerable shrinkage strain could have taken place before loading was applied. The effect of submersion would be to permit the recovery of some of that shrinkage strain and allowance was made for this.

The following hypothesis is suggested to explain the phenomenon of crack-blocking and its effect on fatigue lives of reinforced concrete beams cycled slowly in unidirectional bending in a corrosive environment. As the cracks are geometrically similar blocking should take place at approximately similar rates irrespective of initial crack width. The product near the crack tip will be compacted as the beam recovers its deflection on each cycle, thus increasingly inhibiting the beam in its attempt to return to its original minimum deflection on reduction of load. The salts away from the crack tip experience no such compaction and may remain less dense. However, they are still able to prevent mass transfer into and out of the crack, causing the deposition process to slow down at the crack tip. New salts are thus no longer available for further compaction and blocking ceases to affect the stress range when this condition of equilibrium is reached.

2. Reverse Bending

The results of the reverse bending tests are too limited in number, and in the range of parameters covered, to justify doing more than suggest a few tentative indications. Although crack blocking does occur with reverse bending at slow cycling rates there are no indications of increased fatigue endurance as a result. Our seawater test results are within the scatter of air test results, but they are concentrated at the low end of the range. This suggests the likelihood that, as more test results become available some reduction in fatigue life in seawater will be shown. Tentative support for this conclusion is given by suggestions that the effect of crack-blocking under reverse bending is to prestress the beam, locally, at each crack site, in tension. This would increase the mean stress level in both top and bottom main steel, while leaving the stress range virtually unchanged, and so it would be expected that there would be some reduction in endurance, irrespective of any corrosive effects which might also take place.

3. Mechanism of Corrosion Fatigue Failure

Observations, both visual and microscopic, confirm that corrosion of the reinforcing steel does take place in seawater tests. Rust stains are seen in the region of cracks at an early age in the duration of a test, and it is clear that the hypothesis that the alkaline environment of the concrete would prevent corrosion does not hold when cyclic loading is applied. Visual examination of the failed bars taken from the beam after a test confirms this, and electron microscopy applied to the failed parts of the bars is beginning to yield interesting results which may help to elucidate the mechanism of failure in this type of situation.

FUTURE WORK

In addition to completion of a full programme of reverse bending tests covering a more complete range of the relevant parameters future work will be directed towards gaining a better understanding of the major mechanisms operating in the corrosion fatigue failure of reinforced concrete. To this end it is likely that more small scale tests, and model testing and analysis, will be carried out.

Plans are also in hand, and equipment is being manufactured, to allow some beam tests to be carried out under hydrostatic pressure equivalent to that experienced at sea-bed depths of up to 200 m (650 ft).

A programme of tests on post-tensioned prestressed concrete, which is also widely used in marine environments, has recently been started.

CONCLUSIONS

The results of the programme of testing carried out up to the present time enable the following conclusions to be drawn, for the range of variables studied:-

1. Reinforced concrete beams immersed in seawater and loaded cyclically in unidirectional bending applied at low, ocean wave, frequencies experience progressive blocking of the cracks on the tension side. The effect of this is to reduce the range of bar stress, with an increase in the mean stress, and to increase the fatigue lives of the beams, compared with their expected fatigue lives in air.
2. Otherwise identical beams, when tested at higher, 3 to 5 Hz., frequencies, do not experience crack blocking, and have slightly reduced fatigue lives, compared with their expected fatigue lives in air.

3. Beams immersed in seawater and loaded cyclically in reverse bending applied at low, ocean wave, frequencies experience progressive blocking of the cracks on both sides. The effect of this is to prestress the bars locally at the flexural cracks on both sides, and this increases the mean stress there. The resulting fatigue lives are not enhanced, as would be the case in unidirectional bending, and there are indications that they may, on the contrary, be reduced compared with their expected fatigue lives in air.

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NOTATION

$f_{\max}$	: stress in reinforcing bar at maximum load on beam
$f_{\min}$	: do. do. do. minimum do. do.
f_y	: characteristic strength of reinforcing bar
$\Delta\epsilon_{co}$	: range of concrete crack opening
$\Delta\delta$	: range of beam central deflection
$\Delta\epsilon_c$	: range of concrete strain

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Table 1. Details of Concrete and Steel

<u>Concrete:</u>	Designed characteristic cube strength at 28 days	40 N/mm ² (5700 psi)
	Cube strength at start of test	52 to 75 N/mm ² (7450 to 10,700 psi)
	Water/cement ratio by weight	0.45
	Cement type	Ordinary Portland
	Cement content	400 kg/m ³ (690 lb/yd ³)
	Aggregates	Irregular morainic gravel and sand
	Slump	50 mm (2 in.)
<u>Steel:</u>	Tension bars (unidirectional bending)	2 No. 10 mm Torbar (Approx. 3/8 in.)
	Tension bars (reverse bending)	2 No. 10 mm Torbar in each face
	Lacing bars (unidirectional bending)	2 No. 8 mm mild steel (Approx. 5/16 in.)
	Links	28 No. 6 mm mild steel (Approx. 1/4 in.)
	Cover to main bars	25 mm (1 in.)
	Chemical analysis of Torbar	Carbon 0.28% Phosphorus 0.017% Sulphur 0.035%

Table 2. Composition of Synthetic Seawater (Parts per million)

B	4.6	K	382.0
Br	65.0	Mg	1356.0
C	28.05	Na	10,506.0
Ca	409.0	O	857,113.0
Cl	18.76	S	884.5
F	1.32	Si	3.05
H	108,230.0	Sr	8.13
together with traces of Ag, Al, Ar, As, Au, Ba, Be, Bi, Cd, Ce, Co, Cr, Ci, Cu, Fe, Ge, He, Hg, I, In, Kr, Li, Mn, Mo, N, Ne, Ni, Pb, Rb, Rn, Sb, Sn, Ti, U, V, T, Xe, Zn			

Table 3a. Results: Unidirectional Bending, Air Tests - Slow (0.17Hz)

No.	$f_{\max}/f_y$	Stress Range		Endurance (cycles)
		(N/mm ²)	(psi)	
4	0.92	377	53,900	541×10^3
14	"	"	"	249
17	0.85	345	49,300	734
24	"	"	"	571
26	"	"	"	520
30	"	"	"	792
37	"	"	"	773
38	"	"	"	276
41	"	"	"	411
67	"	"	"	500
78	"	"	"	672
46	0.75	299	42,700	11,426
62	"	"	"	585
86	"	"	"	729
3	0.73	290	41,400	373
93	0.65	253	36,200	2,307

Table 3b. Results: Unidirectional Bending. Air Tests - Fast

No.	$f_{\max}/f_y$	Stress (N/mm ²)	Range (psi)	Endurance (cycles)	Frequency (Hz)
2	0.9	368	52,600	214 x 10 ³	4
6	"	"	"	218	4
42	0.85	345	49,300	256	5
47	"	"	"	346	5
48	"	"	"	250	5
64	"	"	"	342	5
66	"	"	"	331	5
68	"	"	"	289	5
58	"	"	"	406	4
70	"	"	"	619	4
71	"	"	"	347	4
59	"	"	"	909	3
65	"	"	"	417	3
69	"	"	"	289	3
72	"	"	"	315	2
73	"	"	"	334	2
51	"	"	"	495	1
53	"	"	"	420	1
52	"	"	"	398	0.75
56	"	"	"	445	0.5
5	0.8	322	46,000	380	4
7	"	"	"	293	4
54	0.79	314	44,900	295	5
74	0.75	299	42,700	563	5
76	"	"	"	558	5
10	0.74	294	42,000	1626	4
12	0.7	276	39,400	1122	4
22	0.55	207	29,600	12577	5 No failure

Table 3c. Results: Unidirectional Bending. Water Tests - Fast(5Hz)

- Short Tank

No.	$f_{\max}/f_y$	Stress Range (N/mm ²) (psi)		Endurance (cycles)	Position of Failure
80	0.85	345	49,300	194 x 10 ³	Inside tank
92	"	"	"	168	" "
96	"	"	"	192	" "
94	0.75	299	42,700	526	At end of tank
99	"	"	"	460	Inside tank
91	0.7	276	39,400	999	Outside tank
101	0.6	230	32,900	1105	Inside tank
104	0.6	"	"	2603	" "

Table 3d. Results: Unidirectional Bending. Water Tests -

Slow (0.17 Hz) - Short Tank.

No.	Intended $f_{\max}/f_y$	Intended Stress Range (N/mm ²) (psi)		Endurance (cycles)	Position of Failure (Inside or Outside Tank)
9	0.85	345	49,300	744 x 10 ³	Outside(near load point)
16	"	"	"	633	" " " "
18	"	"	"	477	" " " "
21	"	"	"	268	Outside(at end of jacket)
23	"	"	"	407	Outside(near load point)
27	"	"	"	576	Outside(in shear span)
29	"	"	"	659	Outside(near load point)
32	"	"	"	963	" " " "
35	"	"	"	193	Inside
36	"	"	"	880	Outside (in shear span)
81	"	"	"	654	Inside
83	"	"	"	618	Outside (near load point)
87	"	"	"	1209	" " " "
61	0.75	299	42,700	1299	" " " "
34	0.6	230	32,900	4269	" " " "

Table 3e. Results: Unidirectional Bending. Water Tests -

Slow (0.17 Hz) - Long Tank

No.	Intended f_{max}/f_y	Intended Stress Range (N/mm ²) (psi)		Endurance (cycles)	Position of Failure (Inside or Outside Tank)
45	0.85	345	49,300	2699 x 10 ³	Outside tank(i.e. in shear span)
49	"	"	"	2297	No failure. (Stopped for examination)
57	"	"	"	8285	Inside tank
77	"	"	"	1147	Stopped due to excessive deflection.
82	"	"	"	10,086	Inside tank
90	"	"	"	6033	No failure (Still running)
98	"	"	"	2293	Inside tank
100	0.6	230	32,900	8060	No failure(Still running)
103	0.4	138	14,800	2747	No failure (Still running)

•X• Water level at tension steel.

Table 4a. Results: Reverse Bending. Air Tests - Slow (0.17 Hz)

No.	$f_{\max}/f_y$	$f_{\min}/f_y$	Stress Range (N/mm ²) (psi)		Endurance (cycles)
97	0.61	- 0.02	290	41,400	367×10^3
110	"	"	"	"	573
112	"	"	"	"	496
113	"	"	"	"	188
119	"	"	"	"	161
121	"	"	"	"	244

Table 4b. Results: Reverse Bending. Water Tests - Slow (0.17 Hz)
(Short tank tests).

No.	$f_{\max}/f_y$	$f_{\min}/f_y$	Stress Range (N/mm ²) (psi)		Endurance (cycles)	Position of Failure w.r.to tank
107	0.61	- 0.02	290	41,400	191×10^3	Outside
111	"	"	"	"	208	"
115	"	"	"	"	161	Inside
130	"	"	"	"	184	"(long tank)
114	0.41	- 0.01	195	28,300	1930	No Failure (Test stopped) (long tank)

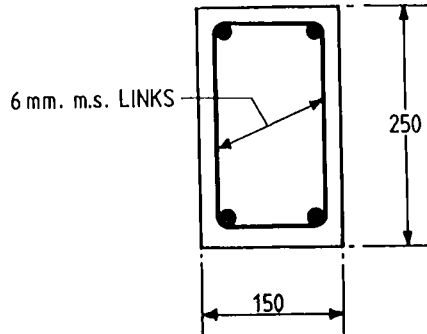


Fig. 1a--Test beam--Cross-section (dimensions in mm)

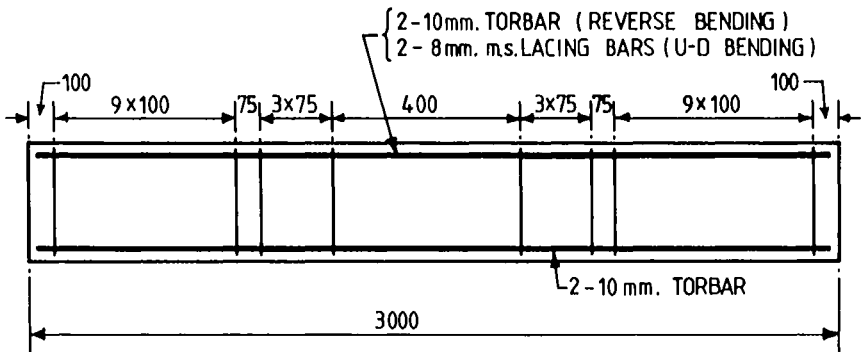


Fig. 1b--Test beam--Reinforcement details (dimensions in mm)

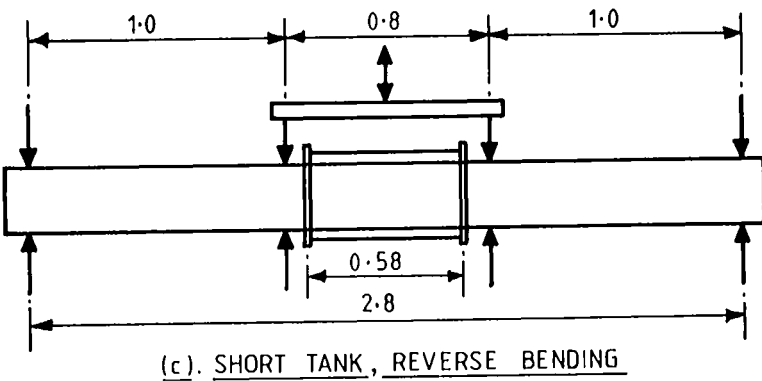
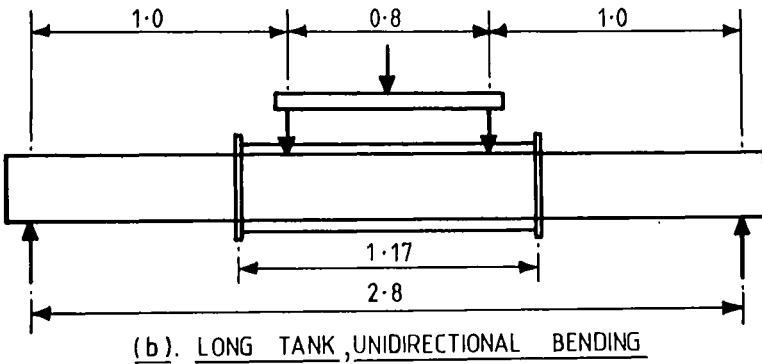
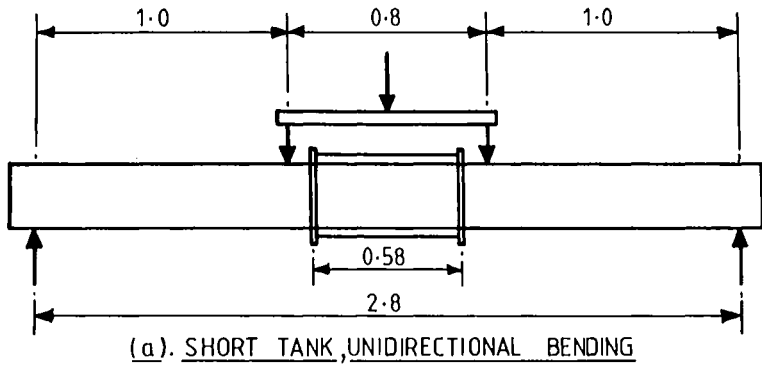


Fig. 2--Loading arrangements and tank dimensions (m)

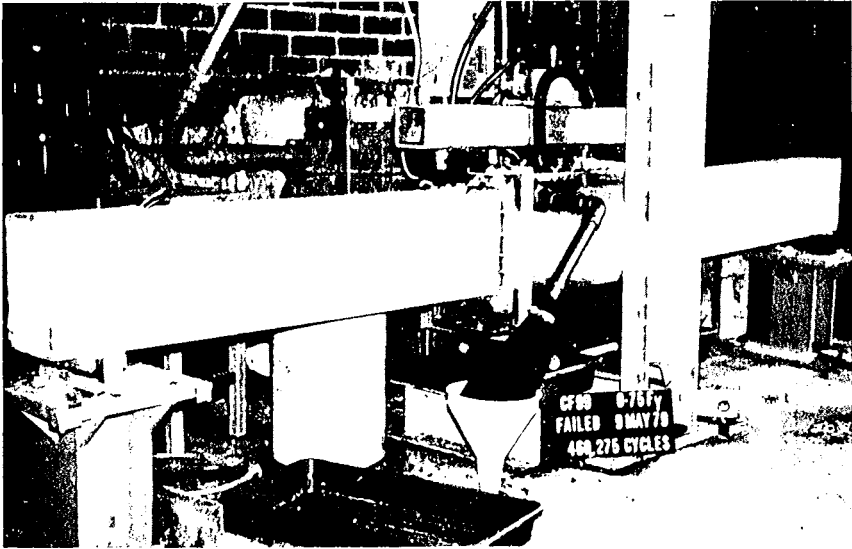


Fig. 3a--Test setup for fast tests--Short tank (unidirectional bending)

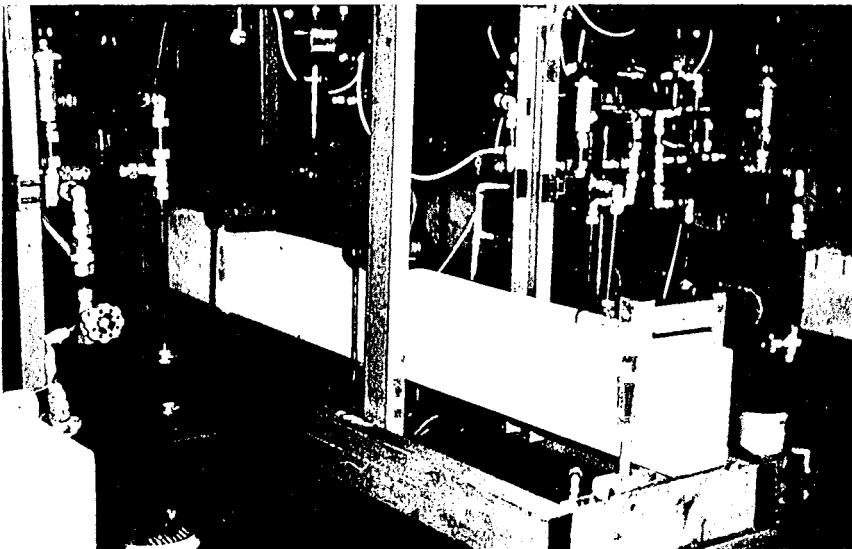


Fig. 3b--Test setup for slow tests--Air (reverse bending)

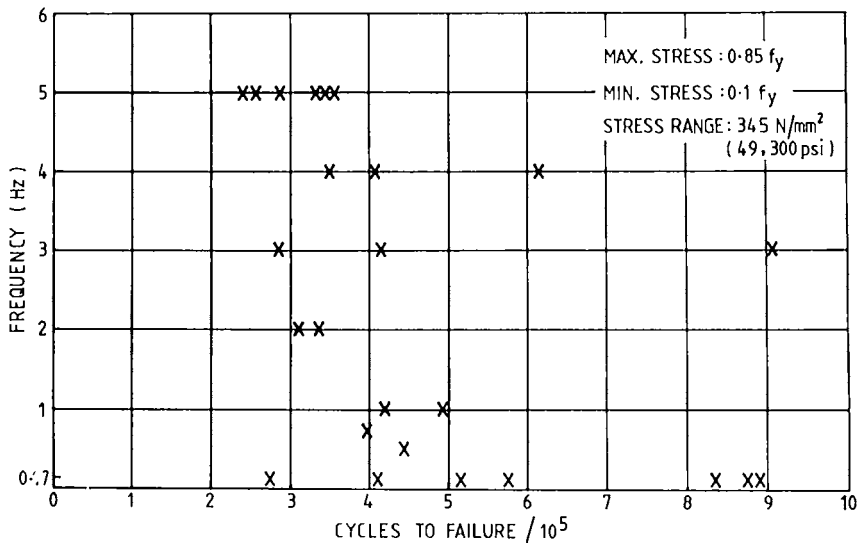


Fig. 4--Endurance versus frequency--Air tests

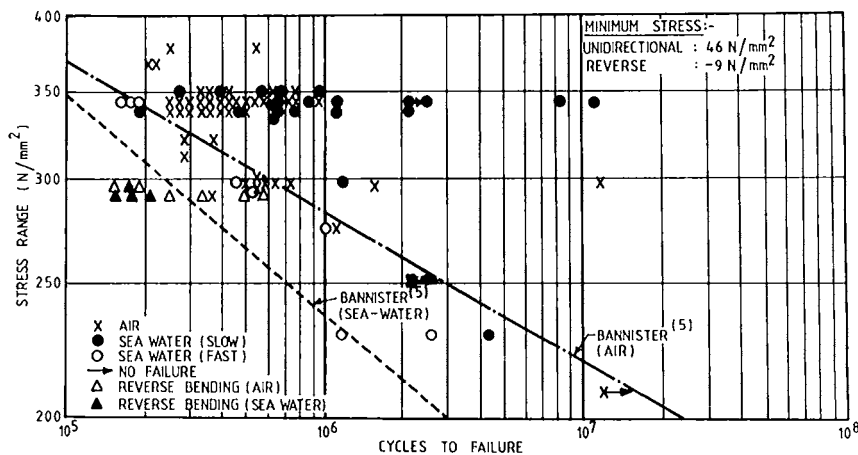


Fig. 5--Endurance versus stress range (intended)

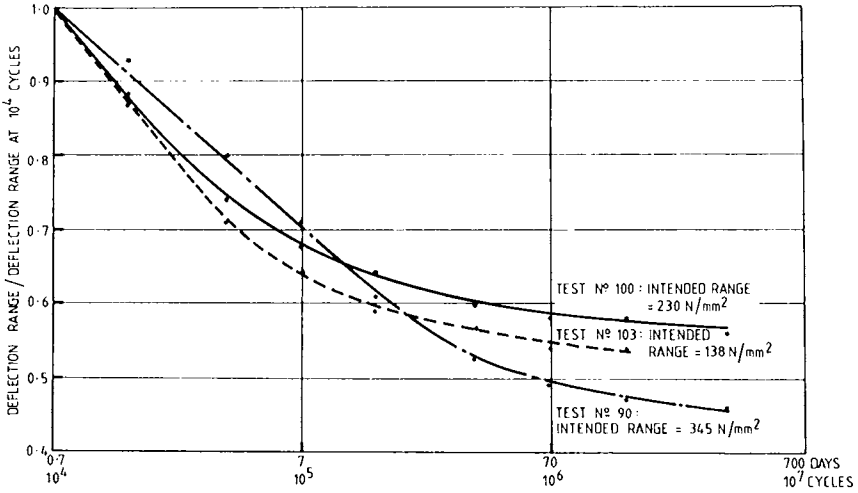


Fig. 6--Endurance versus deflection range as fraction of deflection range at 10^4 cycles

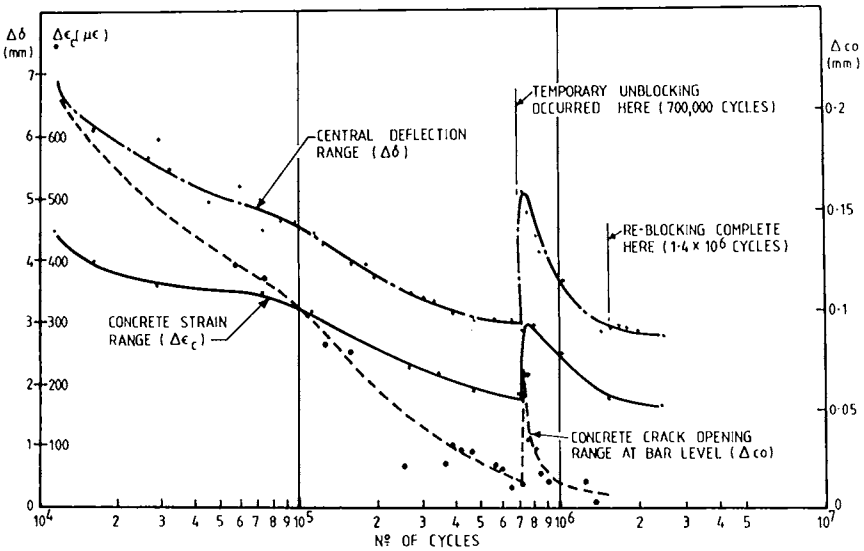


Fig. 7--Test No. 98--Number of cycles versus deflection range, crack opening range, and concrete strain range

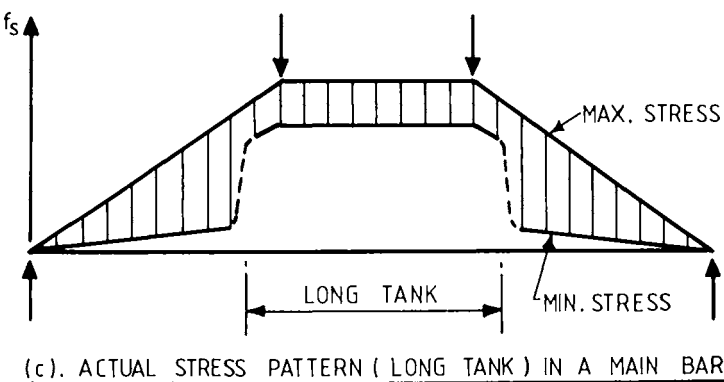
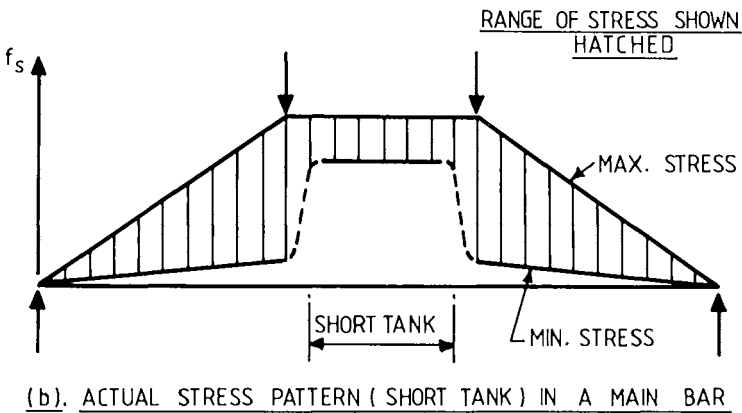
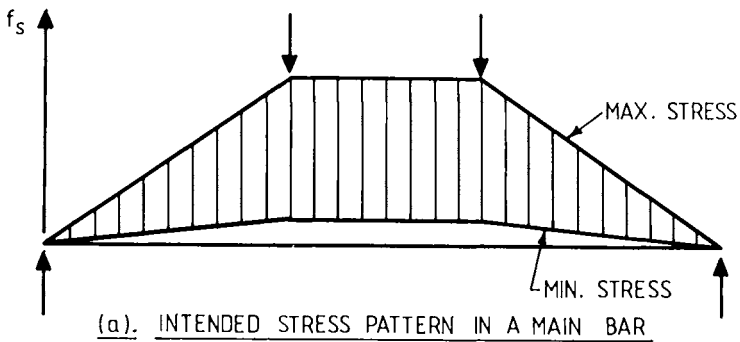


Fig. 8--Comparison of intended and actual stress distributions (slow, submerged tests, unidirectional bending)

Fatigue in Partially Prestressed Concrete Beams

By A. E. Naaman

Synopsis: Available technical information on the fatigue behavior of prestressed and partially prestressed beams is briefly reviewed. Particular characteristics of partial prestressing and its effects on the most influencing variables in fatigue are clarified. Experimental data on the fatigue life of prestressing strands are compared with data on the fatigue life of partially prestressed beams. A close correlation is observed. Global results of a reliability based models of partially prestressed beams in which serviceability and ultimate strength limit states were considered, are presented. They strongly suggest that fatigue in the reinforcement is a controlling limit state that must be seriously accounted for in design.

Keywords: beams (supports); failure; fatigue (materials); fatigue tests; partial prestressing; prestressed concrete; prestressing steels.

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INTRODUCTION

The process of progressive and irreversible deterioration in a material subjected to repetitive stresses is called fatigue. There are several reasons given by ACI Committee 215 (1) to point out that the fatigue strength of concrete members may be an important design consideration; these include:

1. Wide-spread use of ultimate strength design procedures and higher strength materials.
2. Use of concrete members in different types of applications, such as prestressed concrete railway bridges and ties, continuously reinforced concrete pavements, marine structures and the like.
3. New recognition of the effects of repeated loads on a member, which lead to increases in crack widths and deflections in comparison to identical static loading.

Fatigue is often described by a parameter termed fatigue life which essentially represents the number of cycles needed to fail the material under a given repetitive load. It is generally agreed that the relative magnitude of the stress change under load is the most important variable that influences fatigue life; the higher the stress change, the smaller the fatigue life.

Generally, in fully reinforced and fully prestressed concrete members the stress changes in the concrete and in the steel under the effect of repetitive live loads are not critical. This is particularly the case in fully prestressed concrete where the stress change in the prestressing steel due to the application of full live load, is only of the order of a few ksi.

In partially prestressed concrete beams the section is generally uncracked under the sole effect of the dead load and will crack due to the first application of the live load. Subsequent applications of live loads will lead to cracks opening at the decompression stress which is lesser than the cracking stress. First cracking or subsequent crack opening will shift upward the location of the neutral axis of bending of the section and correspondingly a higher rate of increase in the steel stress and correspondingly

in the concrete extreme fiber compressive stress (to maintain equilibrium of forces and moments in the section). These repetitive changes in stresses create fatigue damage in the corresponding materials, reduce bond properties at the interface between steel and concrete and lead to substantial increases in crack widths and deflections under service loads.

In this paper a summary is first given of the available technical information on the fatigue behavior of prestressed and partially prestressed beams. Particular characteristics of partial prestressing and its effects on the most influencing variables in fatigue are clarified. Experimental data on the fatigue life of prestressing strands (S-N curve) are correlated with data on the fatigue life of partially prestressed beams. The results of a reliability based model of partially prestressed beams where most common serviceability limit states and ultimate strength limit state are considered, are rapidly reviewed. They reinforce the belief that fatigue is a controlling limit state in the design of partially prestressed beams.

CRITICALITY OF FATIGUE: PARTIALLY PRESTRESSED VERSUS REINFORCED OR PRESTRESSED

In order to illustrate the relative importance of fatigue in partially prestressed beams a concrete beam (caption Fig. 1) was designed to achieve the same ultimate moment resistance for the following three cases: fully reinforced, fully prestressed and partially prestressed with a partial prestressing ratio $PPR = 0.72$. The beam is described in the example of Ref. 2. Where materials properties and other relevant data are given. The location of the reinforcement for the partially prestressed section is shown in Fig. 1. The partial prestressing ratio represents the ratio of ultimate moment taken by the prestressing steel to the ultimate moment taken by the total steel. The computed stress changes in the reinforcing steel and the prestressing steel for each case are plotted (Fig. 1) versus an applied load in excess of dead load whose value is varied from zero to the specified live load. It can be observed that, for the full live load, the stress change in the reinforcing steel of the fully reinforced cracked beam is about 14.5 ksi (100 MPa), the stress change in the prestressing steel of the fully prestressed uncracked beam is of the order of 4.3 ksi (31 MPa), while for the partially prestressed beam the stress changes are respectively 24.2 ksi (167 MPa) and 21.6 ksi (149 MPa) for the reinforcing steel and the prestressing steel. For the partially prestressed beam a stress jump is shown at a load corresponding to the first cracking load. Under repeated loading the stress jump disappears, and is replaced by a smoother transition; the total stress change in the steel will then depend on whether the dead load moment is smaller or larger than the decompression moment. If it is smaller the total stress change remains about the same. The stress change after pre-cracking is shown in Fig. 1 under "subsequent loadings path."

Two other figures are informative of the behavior of partially prestressed beams. Fig. 2 shows for the same beam the influence of the partial prestressing ratio on the change in steel (reinforcing or prestressing) stress due to the live load. The discontinuity in the curves corresponds to first cracking. It can be seen that higher stress changes are prevalent in partial prestressing; however, even for a partially prestressed beam, significant differences in fatigue can be expected depending on the value of PPR, that is the extent of prestressing or reinforcing. Fig. 3 shows the variation in top fiber concrete compressive stress due to live load for three different partially prestressed sections. Although in about all cases the stress change is less than the fatigue resistance of concrete ($f'_c = 5000$ psi was assumed in the analysis), substantial differences may exist depending on the type of section and the value of the partial prestressing ratio. Note that the T beam of Fig. 3 is the same as that used in Figs. 1 and 2.

It can be generally observed from (Figs. 1 to 3) that if fatigue is a problem in a fully reinforced or in a fully prestressed concrete structure, the problem can be substantially magnified for a partially prestressed concrete structure if a proper design is not achieved.

FATIGUE PROPERTIES OF STEEL AND CONCRETE

The fatigue resistance of concrete structures is directly related to the fatigue properties of their constituents, namely, steel and concrete. Structural failure will occur when the weakest link (constituent) fails.

It is generally accepted that fatigue failure due to concrete is not very likely even in partially prestressed structures. Computerized analysis of more than 200 partially prestressed test beams have indicated only three cases of slabs and hollow core slabs where fatigue of concrete may have been critical (3). It is generally observed that concrete in direct compression or tension can sustain for about ten million cycles a fluctuating stress between 0 and 50% of its static strength (4 to 8). For the purpose of design and to achieve a fatigue life of ten million cycles the stress range limit suggested by ACI Committee 215 on Fatigue can be used:

$$f_{cr} = 0.4 f'_c - \frac{f_{min}}{2} \quad (1)$$

Reinforcing steels exhibit good resistance to fatigue. Figure 4 shows curves of stress range versus number of cycles to

failure for North American bars as given in Ref. (9) where the following conclusions were drawn: the fatigue strength for at least two million loading cycles is practically independent of the grade of steel but depends on the minimum applied stress and the degree of stress concentration induced in deformed bars during the deformation process. Other factors were also found to influence the fatigue strength such as the nominal diameter and the yield strength but to a lesser extent. The following relationship was proposed for use in design and is recommended by ACI Committee 343 on bridge structures (10):

$$f_{sr} = 21 - \frac{f_{min}}{3} + 8 \frac{r}{h} \quad (2)$$

where f_{sr} = safe stress range for more than two million cycles, ksi

f_{min} = minimum stress, ksi

$\frac{r}{h}$ = ratio of base radius r to depth of rolled on deformation (a value of 0.3 may be used in absence of specific data)

Below a certain stress range, termed endurance limit, some materials do not fail up to a quasi-infinite number of cycles. Prestressing steels do not seem to have an endurance limit. At present a required fatigue life of two million cycles is considered normal for most applications. The fatigue strength of prestressing steel depends on the type of prestressing steel (wire, strand, bars), steel treatment, anchorage type, and degree of bond. It has also been shown that stress relieving increases their fatigue limit significantly. The fatigue strengths of wires for a given number of cycles are higher than those of rolled steels. But the differences are small for stress ranges expressed as percentages of the ultimate tensile strengths. In post-tensioned structures bending at an anchorage can cause higher local stresses than those assumed and reduces the fatigue resistance of prestressing steels.

Typical stress range versus number of cycles (S-N) data of prestressing strands taken from various investigators (11 to 15) are shown in Fig. 5. The data of Tide and Van Horn (12) represent averages from three U.S. manufacturers. Most data, except those of Gylltoft (13, 14) come from tests where a minimum stress of 40% to 60% of ultimate was used. A line fitting the data in a range up to two million cycles is shown and can be used to estimate an acceptable stress range at a desired number of cycles. It is important to point out that many of the observed strand failures represented in the data of Fig. 5 occurred at the grips or occurred because of fretting. Fretting between a failed wire and adjacent or center wire of a strand generates friction, stress concentrations and precipitate failure of the strand.

relation shown in Fig. 5 can be considered a lower bound design line.

As the S-N curve such as shown in Fig. 5 does not take into account the effect of minimum stress, a Goodman diagram or a Smith diagram can be used to represent the stress range at a given number of life cycles. A typical Smith diagram recommended by the FIP Commission on prestressing Steels (16) is shown in Fig. 6 for a life of two million cycles. It suggests a stress range of about $0.15 f_{pu}$ at a minimum stress of about $0.75 f_{pu}$. The author believes that such values are very optimistic and do not seem to correlate with actual test results. In comparison, the relation shown in Fig. 5 indicates a stress range of about $0.10 f_{pu}$ for a life of two million cycles and a minimum stress less than $0.6 f_{pu}$.

FATIGUE IN PRESTRESSED CONCRETE BEAMS

Much can be learned from published studies on the fatigue behavior of prestressed concrete beams and can be used to better understand the fatigue behavior of partially prestressed beams.

Ozell and Ardaman (17) conducted fatigue tests to determine the behavior of prestressed concrete beams. Eight beams, 6 in. x 8 in. x 20 ft., with a center load, were tested. The test results indicated that: (a) the failure of the beams was due to the breaking of the wires in the stands by fatigue; (b) the load deflection relationship was affected slightly during the first 30,000 cycles but greatly just prior to failure of the beam; (c) the permanent set (deflection) was also small at first but increased appreciably just before failure.

Hanson, Hulsbos, and Van Horn (18) performed fatigue tests on prestressed concrete I-beams. The tests were carried out on I-beams having an overall height of 18 in. and a web width of 3 in. The beams were prestressed with 270 ksi grade strand. Each of the six test beams was simply supported and subjected to a symmetrical two-point loading. They concluded that:

1. The flexural fatigue life of the test beams was less than expected from available information on the fatigue strength of strands (i.e. $0.12 f_{pu}$).
2. Fatigue failure will not occur in uncracked sections. However, fatigue failure may occur in a poorly proportioned section at repeated load levels below the static flexural cracking load, provided the section has been cracked by a previous overload.
3. The response of cracked sections is significantly affected by repeated loading of a magnitude sufficient to cause flexural cracks to open. This change is brought about both by progressive breakdown of the bond between

the prestressing steel and the concrete, and by changes in the stress-strain relationship of concrete. The change occurs rapidly in the early load cycles, but reaches very nearly stable values after about 30,000 cycles.

4. Concrete compression fatigue failures are possible at high load levels in under-reinforced sections, and at lower load levels in over-reinforced sections.

Rabbat, et al (20) tested in fatigue a number of full scale pretensioned prestressed concrete composite beams where the fluctuating load produced a nominal tensile stress on the bottom fiber of $6\sqrt{f'_c}$. Crack formers were used, thus essentially leading to a cracked section under maximum load. Many of their test beams for which the calculated first cycle stress range in the strands was less than 13 ksi (90 MPa) failed at about three million cycles, raising a very serious question on the fatigue properties of prestressing tendons.

Fatigue tests by Irwin (21) on full scale prestressed concrete beams have indicated a fatigue life above three million cycles. However, based on a stress range comparison, Irwin concluded that failure by fatigue of prestressing tendons could be expected in partially prestressed beams at less than three million cycles.

FATIGUE IN PARTIALLY PRESTRESSED BEAMS

There are few investigations on the behavior of partially prestressed beams subjected to repeated loads. Among existing studies, few give experimental and/or predicted values of stresses or stress changes in the materials rendering any comparative evaluation enormously difficult.

Bennett and Dave (22) tested nine partially prestressed beams under repeated loads. Each test consisted of at least three million cycles of loading in which the maximum load was the design service load and the minimum load was one-half of it. The beams were designed for an ultimate moment resistance equal to twice the service moment. Only one beam failed by fatigue of the prestressing steel at about 2.6 million cycles while all the others took more than three million cycles.

Dave and Garwood (23) tested partially prestressed post-tensioned beams under fatigue loading. The fluctuating load produced bending moments varying from about 29 percent to 58 percent of the design ultimate moment. All beams were tested to three million cycles without fatigue failure.

Although stresses and stress ranges in the reinforcement could have been predicted in the above two investigations, no

particular information was given on these values, and no experimental measurements were made to obtain them.

Fauchart, Kavyrchine and Trinh (24, 25) have tested in fatigue (constant cyclic load) a number of partially prestressed rectangular and T-beams ranging from fully reinforced to fully prestressed. The prestressing steel consisted of wires of strength $f_{pu} \approx 242$ ksi (1670 MPa) and the reinforcing steel consisted of deformed bars (Tentor) of yield strength ranging from 68 ksi (469 MPa) to 75.8 ksi (523 MPa) depending on the diameter. The minimum applied load was $0.15 M_u$ and the maximum load varied from $0.45 M_u$ to $0.75 M_u$ depending on the test series. The beams were generally precracked in the first five slow cycles of loading then fatigued. The strains in the reinforcing steel and the prestressing steel were experimentally recorded using special ring extensometers (called Tensotast) embedded in the concrete and protected by a plastic shield. The stress and stress ranges under repeated loads were calculated from both the experimentally measured strains and from equilibrium of the section. They were found comparable (less than 10% apart). By properly selecting the eccentricity of the prestressing steel, the stress range in the reinforcing steel was greatly varied. A summary of their test results is given in Table 1. The data of stress range versus number of cycles to failure are plotted in Fig. 7 where the dark filled points represent failure by the prestressing steel. Note that the stress range in the specimens that failed by the reinforcing steel is given in parentheses near each point. These are very high values and span a compression tension range. The reinforcing steel failure points are shown to indicate that the prestressing steel has survived up to the corresponding cycle. Also plotted in Fig. 7 are data from Ref. 20 on the fully prestressed beams described earlier which acted as cracked beams under maximum load. Because the observed stress varied during cycling, a small vertical line is shown to represent each specimen. The prediction relation derived from fatigue data on strands (Fig. 5) is shown on the same figure for comparison. It can be observed that the failure of beams by fatigue can be fairly well predicted from the failure of the prestressing steel alone. More research is needed to confirm such observation.

EFFECTS OF FATIGUE LOADING

There are several reasons for which premature failure may occur in prestressing tendons subjected to repeated loads. Failures due to stress concentrations at the grips are often cited thus leading to an observed fatigue life smaller than the actual fatigue life. In the case of strands twisting-untwisting during fatigue and fretting between a failed wire and adjacent wires or the center wire precipitate failure of these wires and the strand itself.

or partially prestressed, crack widths and deflections generally increase under repeated loadings. Typically, in partially prestressed beams, pronounced increases in crack widths and deflections are observed during the early stages of cyclic loading followed by a relatively stable period during which they either increase at a very slow rate or remain essentially the same. The stable period generally extends until some point where fracture of one tendon (or one individual wire of a strand) occurs releasing its share of loading to the other tendons. Further reinforcement failures lead to a rapid deterioration in behavior accompanied by a new cycle of significant increases in crack widths and deflections. Shortly thereafter failure of the member occurs.

The increase in crack widths and deflections in concrete members is mostly attributed to the cyclic creep of concrete and the bond deterioration accompanied by slip between the reinforcement and concrete on either side of existing cracks. Although the slip may be significant to produce deterioration in some tendons, general bond failure seems unlikely when prestressing strands are used. A recent study by Gylltoft (14) on the bond properties of prestressing strands subjected to fatigue loading in direct pull-out indicated a predominant fatigue failure of the strands prior to bond failure. Only in one test, where the embedment length was 40 cm (≈ 16 in.) bond failure occurred with continuously increasing slip up to about 8 million cycles.

The occurrence of significant slip in some of the tendons lead to their losing some or total tension; hence the stress ranges in the other tendons will be higher for the same applied loading and their fatigue life may be substantially reduced. This effect is particularly acute in beams with few prestressing tendons.

If the influence of slip is not significant, then the increase in crack width and deflection under repeated loading can be attributed mostly to the cyclic creep of concrete. Typical variations in crack widths and deflections with loading cycles for partially prestressed beams are given by Bennett and Dave (22). They observe that crack widths and deflections increase during early loading stages then typically stabilize up to about 90 or 95 percent of fatigue life where they start increasing at a much faster rate followed by the failure of the member. Their average ratio of deflection at the last reading prior to failure to the first cycle static deflection was 1.73 with a standard deviation of 0.21. A slightly similar result is reported by Dave and Garwood (23) where the average ratio for deflection was 1.71 with a standard deviation of 0.11. In both studies the crack width ratio was also recorded. The average ratio given in (22) was 4.39 with a standard deviation of 3.17 while in (23) the average ratio was 1.65 with a standard deviation of 0.35. The above results suggest that little correlation may exist, at least quantitatively, between the increases in crack widths and deflections under fatigue loading. Other reinforcing parameters may significantly influence crack widths.

RELIABILITY ANALYSIS OF FATIGUE

The application of reliability theory to analyze structural behavior is relatively recent. Reliability is particularly useful in modeling and understanding random phenomena with large variabilities. Fatigue, for instance, is particularly suited to such methodology.

To the author's knowledge only one recent study of the reliability of partially prestressed concrete beams is available (26). In that study a reliability based design model is developed. It is essentially based on the method call "first-order second-moment" which requires the knowledge of only the mean and variance of the distributions of the resistance and load. In this method a parameter called safety index or reliability index β is needed as an input variable specified by the code authority. However, such specified values are not available for prestressed and partially prestressed concrete beams. The predicted value of the safety index β was evaluated in (26) for typical current design situations involving precast prestressed structural elements.

Although in a number of previous studies the reliability index β has been obtained for concrete or steel structures at ultimate limit state, little has been done to evaluate β at serviceability limit states. In Ref. (26) the reliability index of partially prestressed concrete beams was evaluated at six serviceability limit states. β is directly related to the probability of failure where failure does not necessarily refer to collapse but also to unserviceability or violation of any code specified limit such as maximum stress range in the steel or maximum deflection. The six limit states considered were: maximum crack width, short-and long-term deflections and fatigue stress ranges in the concrete, the reinforcing steel and the prestressing steel.

In order to determine the β values for typical design situations, the method called Code Calibration was used. Resistance prediction equations at each serviceability limit state were developed in terms of moments. A Monte Carlo simulation model was used to compute the statistical properties of the resistance at each limit state. The statistical properties of the load were collected from available literature and used in the evaluation of the reliability index.

The effects of many parameters on the observed values of the reliability index β were analyzed. They include the effect of various types of beam cross sections, span length, magnitude of the live loads, amount of nonprestressed reinforcement and some important materials properties such as concrete compressive strength. For each case at least three reinforcing methods were considered: fully prestressed, fully reinforced, and, partially prestressed with an optimum partial prestressing ratio PPR. The optimum was defined as the minimum value for which all serviceability criteria are satisfied and one is controlled by fatigue.  Documents
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Typical variations of the reliability index β for different limit states and the three different reinforcing methods are shown in Fig. 8 for one beam. Similar results were obtained for the many other cases studied. The lower the reliability index the lower the reliability, or the higher the probability of failure. Figure 8 suggests that the fatigue limit state is among the most critical and is generally more critical for partially prestressed concrete than for either reinforced or prestressed concrete. The figure illustrates a general trend. A summary of the variation of the reliability index for each limit state and for all cases studied is illustrated in Fig. 9. The mean values represent the average of 64 data points. If we assume a normal distribution for the safety margin, then the reliability index can be easily correlated with the probability of failure. For such a case an axis was drawn in Fig. 9 showing the corresponding probability of failure. Of course, failure represents unserviceability but unserviceability can be critical in the case of fatigue. The variation of the reliability index β at the ultimate flexural strength limit state is also shown in Fig. 9 and can be compared to β values at serviceability limit states. Although the scatters can be substantial, mean values suggest that fatigue will be critical in most designs. Moreover, it can be observed that the probability of failure by lack of ultimate resistance is generally small in comparison to other serviceability limit states.

CONCLUSIONS

1. The failure of partially prestressed concrete beams subjected to repeated loads is generally due to failure of their reinforcement by fatigue. Hence for design, their fatigue life can be predicted from the smaller of the fatigue lives of the reinforcing steel or the prestressing steel.
2. Fatigue data of prestressing strands suggest a fatigue life of about two million cycles at a stress range of 10 percent of ultimate strength and a minimum stress not above 60 percent of ultimate. Their fatigue life at other stress ranges for common design variables can be predicted from the following relation $\Delta f_{ps} / f_{pu} = -0.123 \log N + 0.87$.
3. The risk of fatigue failure in partially prestressed concrete beams is greater than in fully reinforced or fully prestressed beams. However, proper design (such as the appropriate selection of the partial prestressing ratio and the relative location of the reinforcement) can substantially reduce the magnitude of the stress changes due to repeated loads and hence improve fatigue life.
4. Although fatigue failure of the prestressing tendons is often cited as a primary cause of failure, the results

of a reliability analysis of partially prestressed beams indicated that failure due to the reinforcing steel is as likely to occur.

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Table 1

Summary of Fatigue Data on Partially Prestressed
Beams (Adapted from Refs. 24 and 25)

		Pre- stressing Steel ^a	Rein- forcing Steel ^a	Δf_s ksi (MPa)	Δf_{ps} ksi (MPa)	$\frac{\Delta f_{ps}}{f_{pu}}$	N_f to failure of First Wire or Bar	Failure by:
Rectangular Section	A1	6 ϕ 7	-	-	64.7 (447)	0.267	231000.	P.S.
	A2	5 ϕ 7	3 ϕ 6	94 (647)	64.7 (446)	0.267	86000.	R.S.
	A3	4 ϕ 7	1 ϕ 6 & 2 ϕ 10	63.7 (439)	52.3 (361)	0.217	108000.	R.S.
	A3'	4 ϕ 7	1 ϕ 6 & 2 ϕ 10	57.2 (394)	37.8 (261)	0.156	88000.	R.S.
	A3''	4 ϕ 7	1 ϕ 6 & 2 ϕ 10	81 (558)	44.4 (306)	0.183	54000.	R.S.
	A4	-	3 ϕ 10 & 3 ϕ 12	40.2 (277)	-	-	420000.	R.S.
T-Section	B1	5 ϕ 7	-	-	54.9 (378)	0.227	196000.	P.S.
	B2	4 ϕ 7	2 ϕ 8	57.2 (397)	54.7 (377)	0.226	88000.	P.S.
	B3	3 ϕ 7	2 ϕ 8 & 2 ϕ 10	52 (221)	22.2 (153)	0.092	500000. + 800000.	P.S. & R.S.
	B3'	3 ϕ 7	2 ϕ 8 & 2 ϕ 10	33.8 (233)	21 (145)	0.087	First cycle (Discontinued)	P.S.
	B4	-	4 ϕ 12	46.2 (318)	-	-	170000.	R.S.
	C3	3 ϕ 7	2 ϕ 8 & 2 ϕ 10	45.9 (316)	34.2 (236)	0.141	300000.	P.S.
	D3	3 ϕ 7	2 ϕ 8 & 2 ϕ 10	19.5 (191)	16.3 (160)	0.067	830000 ^b + 450000	P.S. + R.S.

a Number and nominal diameter of wires or bars in mm

b Longitudinal cracking

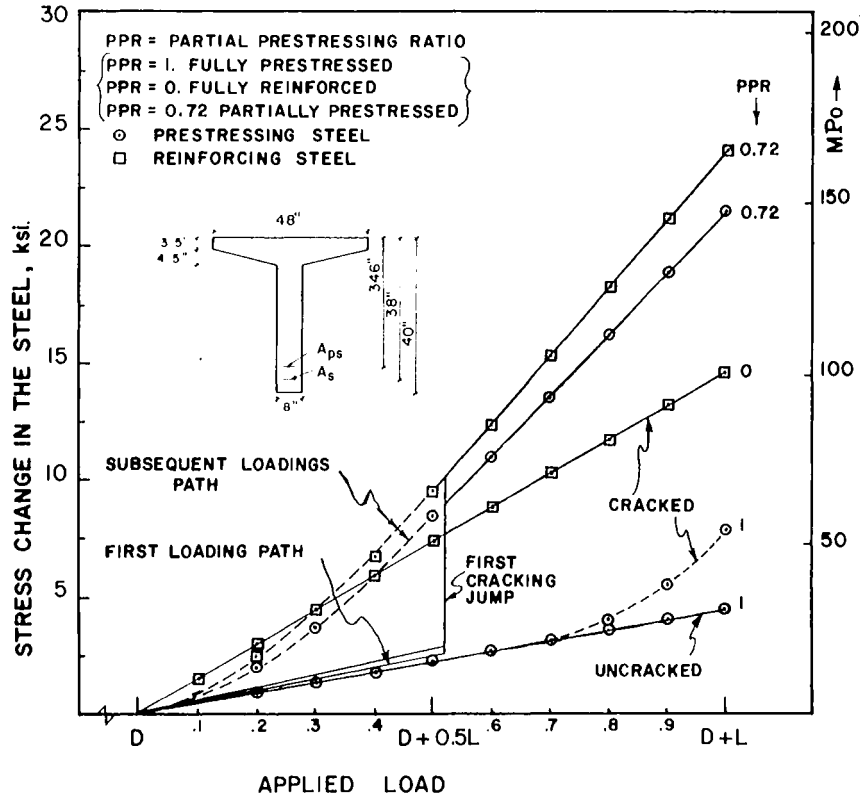


Fig. 1--Typical comparison of stress changes in the steel for reinforced, prestressed, and partially prestressed beams

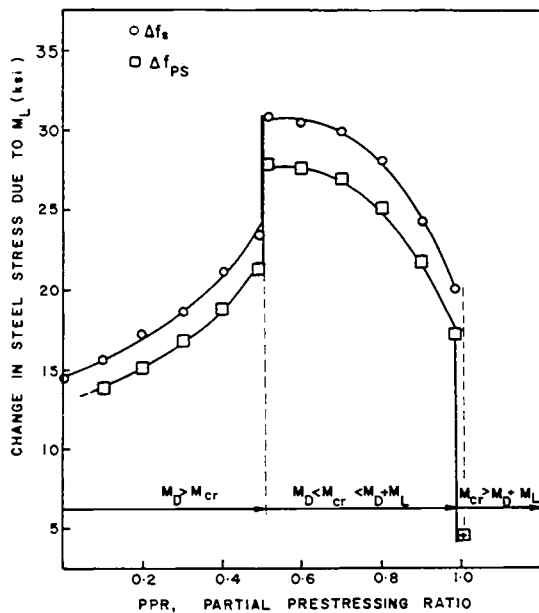


Fig. 2--Typical stress changes in the steel at different levels of prestressing

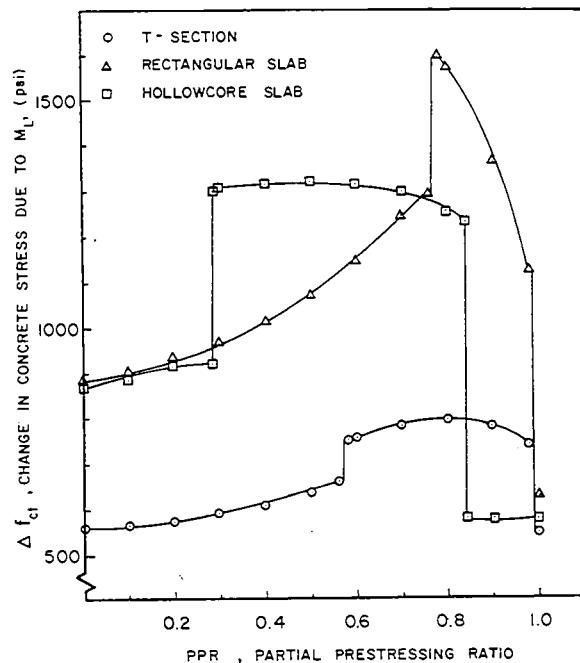


Fig. 3--Typical stress changes in concrete extreme compressive fiber at different levels of prestressing

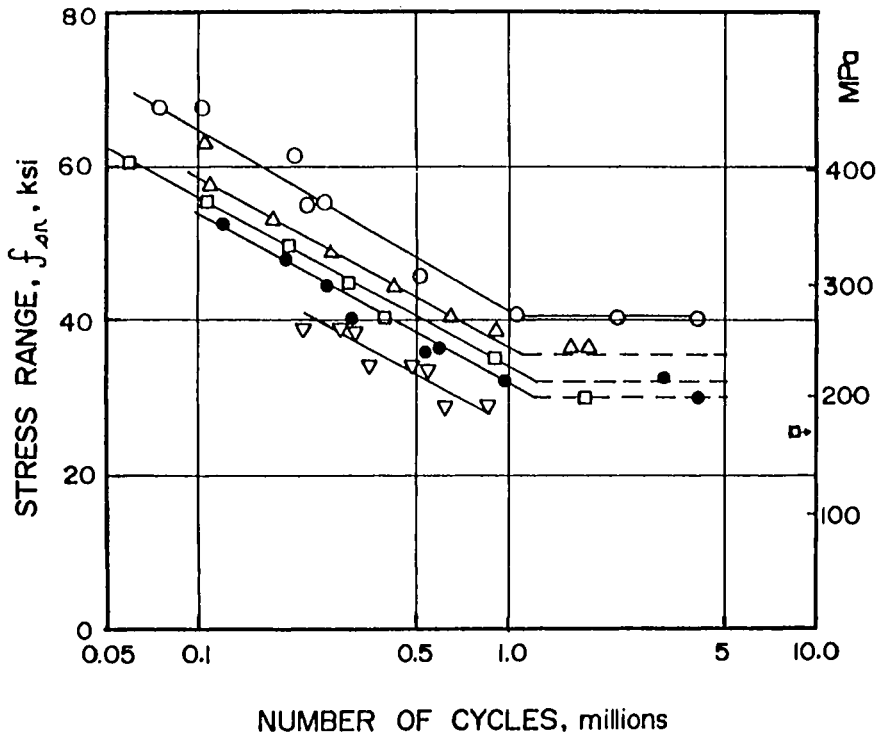


Fig. 4--Representative fatigue test results for North American reinforcing bars (Ref. 9)

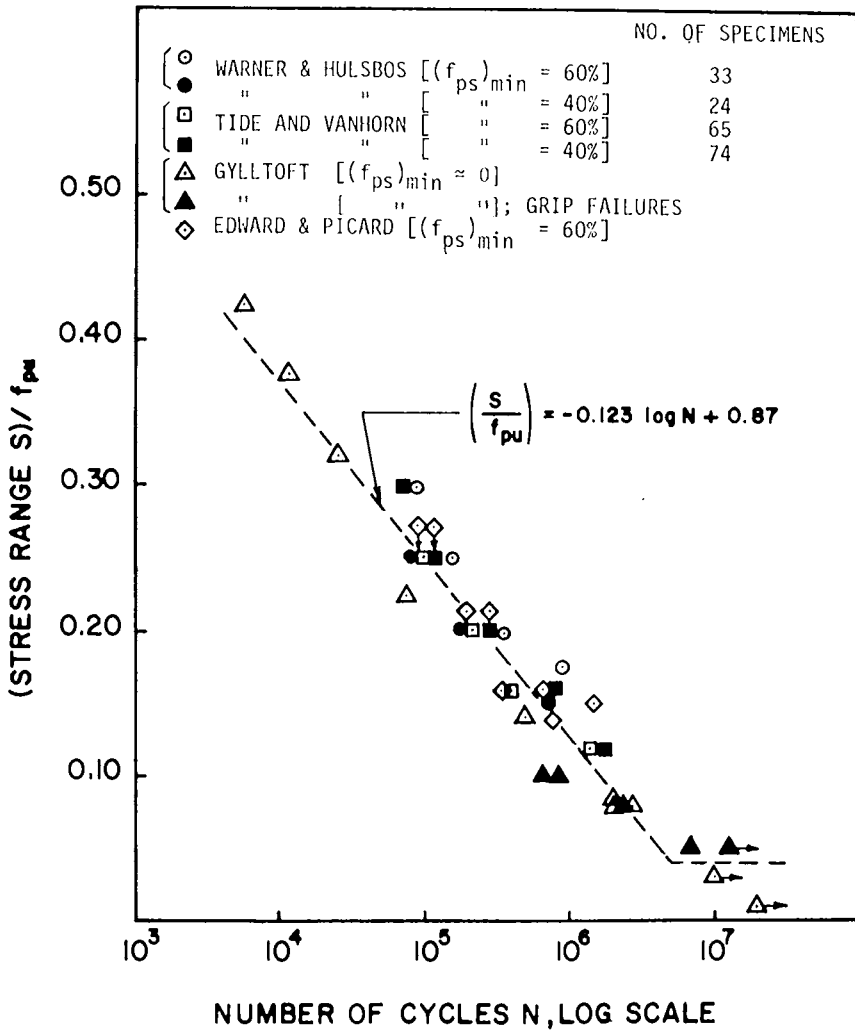


Fig. 5--Derived S-N curve for prestressing strands

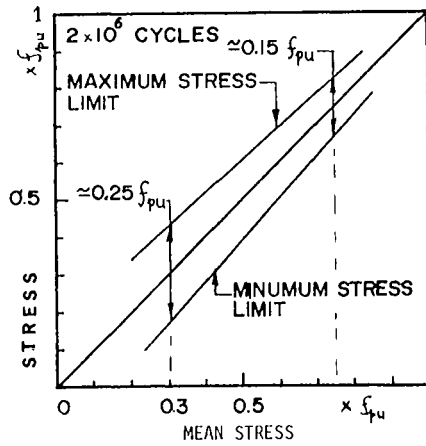


Fig. 6--Typical Smith diagram for prestressing steels (Ref. 16)

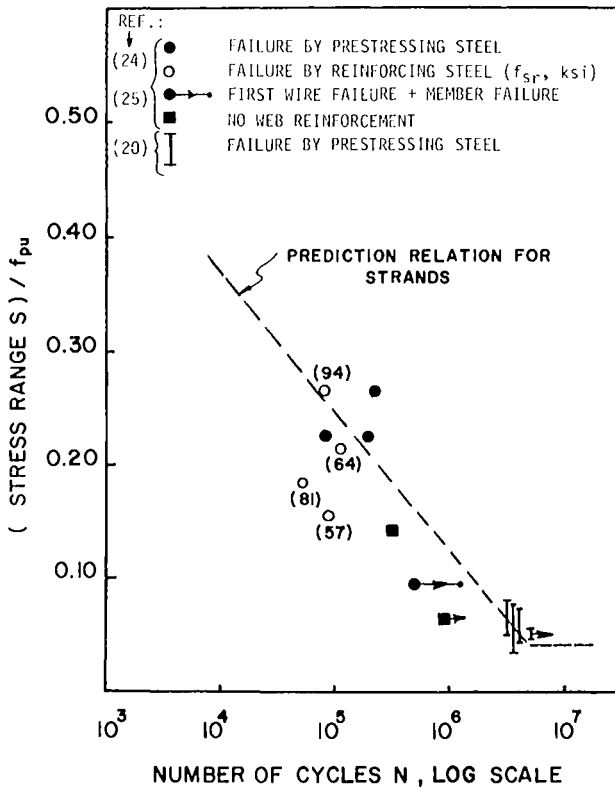


Fig. 7--Derived S-N data at failure of prestressed beams

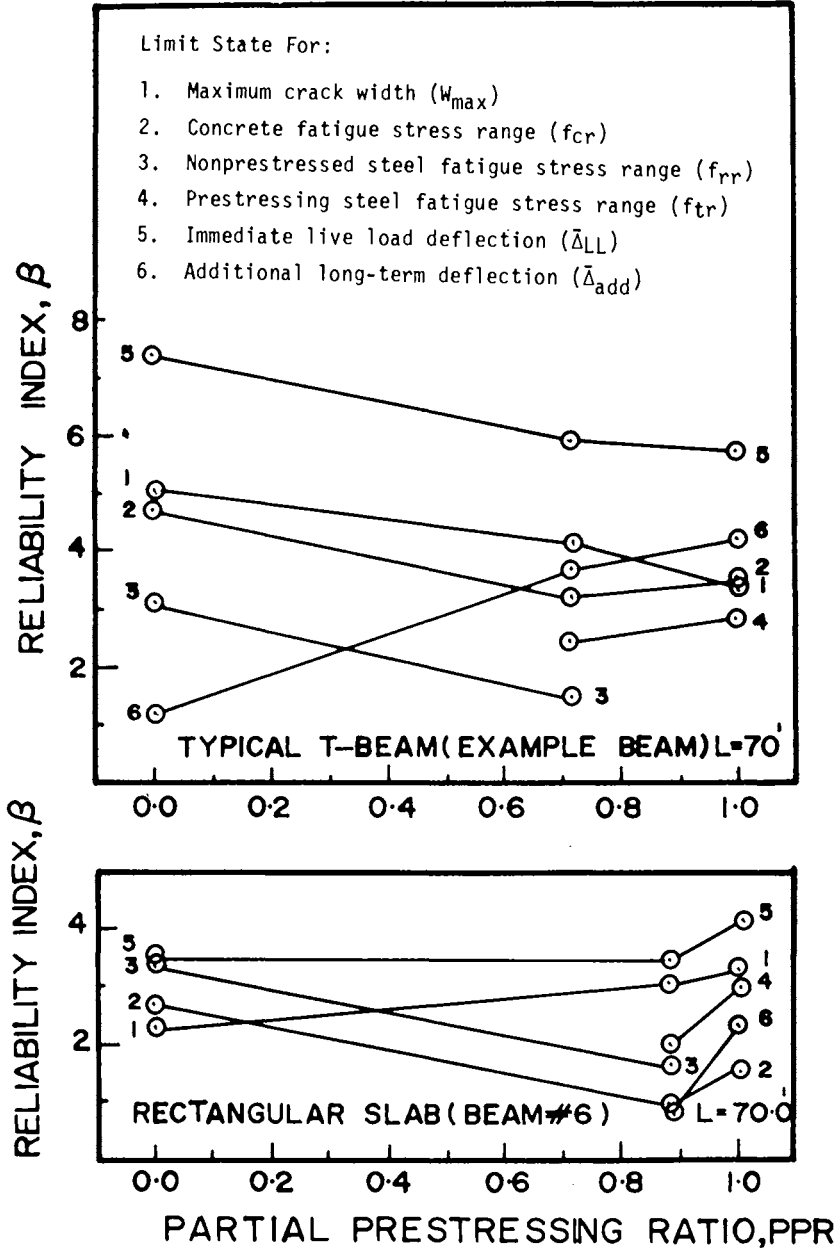


Fig. 8--Typical effect of the partial prestressing ratio on the reliability index

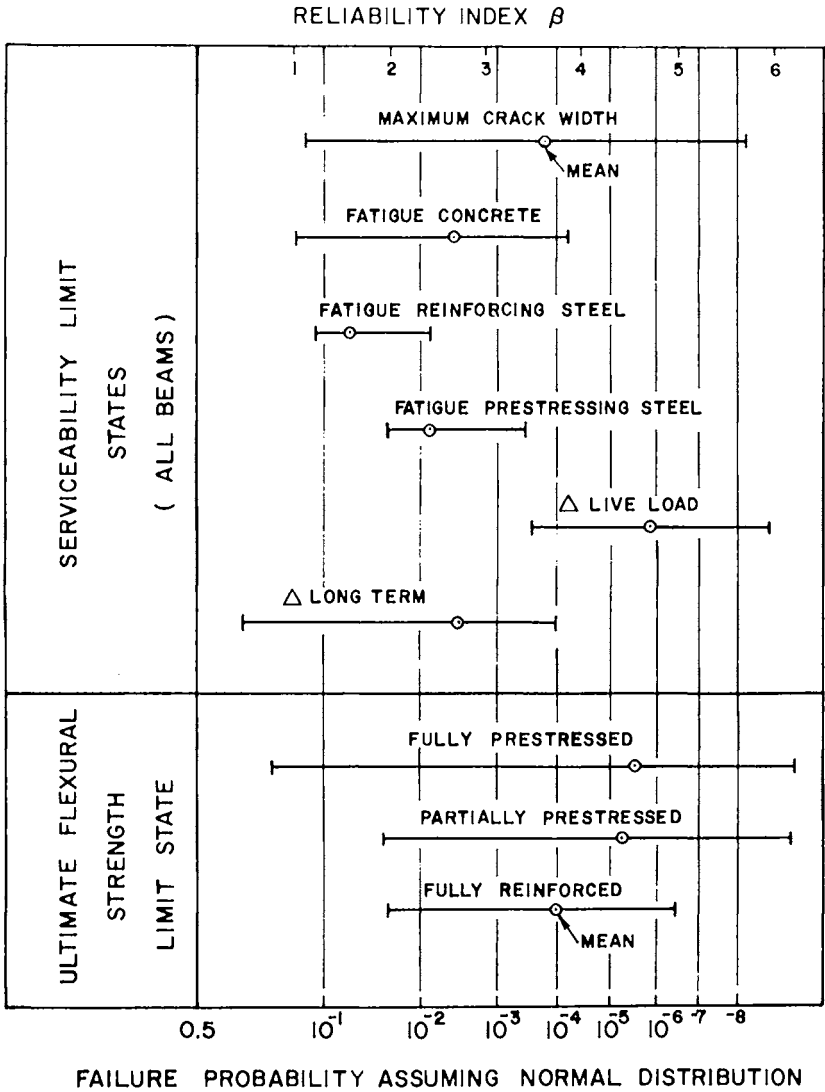


Fig. 9--Reliability index of partially prestressed beams at serviceability and ultimate limit states

Crack Growth and Fracture in Plain Concrete — Static Versus Fatigue Loading

By Stuart E. Swartz, Chen-Ming James Huang, and Kuo-Kuang Hu

Synopsis: As part of an ongoing experimental/analytical research effort to evaluate the feasibility of a test method for fracture toughness of concrete, forty-eight plain concrete beams have been tested in bending to failure. All beams were notched and then precracked to different crack length/depth ratios prior to loading to failure. The precracking was done using an electro-hydrodynamic materials testing system and displacement control.

The beams which were cracked in fatigue were subjected to one million cycles of sinusoidal loading at 4 Hz. After the cycling was complete on a beam, the crack depth was determined using a compliance calibration technique following which the beam was loaded to failure. A load versus crack-mouth-opening-displacement trace was plotted during this final load run.

For each beam tested in fatigue, a companion beam was precracked "statically" by loading in repeated cycles until the crack depth, as measured by compliance calibration, matched that of the fatigued specimen.

The studies were made on two different beam sizes in three and four-point bending with two different mix designs. Test results indicate the failure strength and associated maximum stress-intensity of the statically precracked beams to be slightly higher than those precracked in fatigue.

Keywords: beams (supports); bending; concretes; cracking (fracturing); crack propagation; failure; fatigue (materials); fatigue tests; plain concrete.

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INTRODUCTION

A number of investigations directed toward measuring the fracture toughness of cementitious materials are reported. These can be categorized as those which used notched specimens but did not effectively precrack prior to measuring fracture toughness (1, 2, 3, 4, 5) and those which did precrack the specimens before failing and determining estimates of the fracture toughness (6, 7, 8, 9, 10, 11, 12, 13).

Tests performed on limestone (14) showed that "...a crack probably cannot be adequately simulated by a sharp sawcut for the purpose of measuring fracture toughness. Even if the cut could be made sufficiently sharp so as to provide a stress singularity similar to that of a natural crack, the closure stresses would still be non-existent." The writers in performing similar experiments on concrete beams (11) also noted that in some instances, crack closure occurred.

The question of whether the fracture toughness concept is applicable to concrete is still unresolved. This effect has been shown to occur in cement paste or mortar (2, 6, 7, 8, 10, 13). For concrete, Walsh (3, 4) reports that fracture toughness for beam specimens is a viable concept if the beam depth is 10 in. (254 mm) or greater while Kitagawa and Suyama (5) indicate that this is independent of size. On the other hand, tests on plate specimens by Kesler, Naus and Lott (9) led to their concluding the concept is not applicable at all to concrete.

Earlier work presented by Naus and Lott (1) using notched beams gave "effective fracture toughness" in terms of V_{eff} .

mix design parameters. The toughness values were calculated using the Srawley formula for elastic fracture mechanics (15).

None of these investigations on concretes, mortars and pastes, other than those previously reported by the writers (11, 12) was concerned with growing cracks by fatigue prior to the final loading to failure. The only related investigation known to the writers was that reported by Schmidt (14) in which lime-stone beams were precracked in fatigue prior to loading to failure.

On the other hand, the standard method for obtaining the fracture toughness of metallic materials involves measurements on specimens which are precracked in fatigue (16).

Thus, the writers were motivated to examine the following ideas:

1. Can a crack be generated in a plain, concrete beam in a controlled manner by static loading and/or fatigue loading.
2. What difference, if any, will there be in load-carrying capacity and associated maximum stress-intensity between a beam precracked statically and one precracked in fatigue.

In order to answer these questions, a comprehensive testing program using forty-eight plain beams loaded statically and in fatigue was performed. It is intended here to summarize this program with emphasis on the aspects of fatigue testing.

TEST SPECIMENS

Two sizes of test beams were constructed. In each size a total of thirty-two beams was made with eight for compliance calibration (11, 14) and the remainder precracked statically or in fatigue. Two mixes were used and two types of load--three point and four point bending were applied. The beam and loading dimensions, nominal mix properties and other data are contained in Table 1 and Fig. 1. Further details on these are given in References 11 and 12.

All beams were tested using an electro-hydrodynamic materials testing system with a commercially available displacement transducer with sufficient sensitivity to monitor and control movement normal to the crack at its mouth, i.e., crack-mouth-opening-displacement (CMOD). Test setups for a small and large beam in four-point bending are shown in Figs. 2 and 3.

FATIGUE PRECRACKING

In precracking the beam, it is possible to apply the load in one of two ways--by load feedback control or by strain (crack-mouth-opening displacement) feedback control. The writers have used both methods. In the first instance--load control--it was found to be extremely difficult to control the extent of cracking to some desired level. A typical curve of total crack length versus load cycles reported previously (11) using load control is shown in Fig. 4.

Using strain or CMOD control, following Schmidt (14) proved to be far easier. The procedure used by the writers is outlined below.

1. Establish a calibration curve (11, 14) of beam compliance versus crack depth using two beams for each test series. A typical curve is shown in Fig. 5. Each experimental point on the curve is obtained from the inverse slope of load versus CMOD for the corresponding notch depth.
2. Using a sine function operating at 4 Hz with control on the CMOD operate the system to one million cycles. It is possible to run for fairly long periods of time without operator intervention. However, due to the decrease of load as damage increases, it becomes necessary to readjust the displacement range from time to time.
3. After the precracking is complete, the beam is loaded to failure and a trace of the load versus CMOD is obtained.
4. A companion beam is precracked statically (cf Ref. 12 for details) to the same crack length and also loaded to failure.
5. For the purpose of computing maximum stress-intensity and making comparisons on the basis of net failure stress the peak load of the failure curve was used.

A sequence of tests for three large beams in four-point bending is traced in Figures 6-8. In each of these graphs, three dependent variables are plotted versus number of cycles (log scale). The lower two traces show the upper and lower limits on CMOD. These represent the command or input signals of the sine function. The middle two traces are of the maximum and minimum load response and the upper trace shows the change in crack length.

A number of observations may be made which are common to the three figures. In all cases the initial a/W value is not zero but about 0.05. This is the depth of notch cut into the beam