FRACTURE ENERGY AND FATIGUE STRENGTH OF UNREINFORCED CONCRETE BEAMS AT NORMAL AND LOW TEMPERATURES

ULF OHLSSON, PER ANDERS DAERGA and LENNART ELFGREN

Division of Structural Engineering, Luleå University of Technology, S-951 87 Luleå, Sweden

Abstract—Results are presented from tests where the fracture energy and the fatigue strength have been determined for unreinforced concrete beams. The tests were performed at temperatures between +20 and −35°C with concrete with compressive strength varying between 25 and 100 MPa.

INTRODUCTION

A SUBSTANTIAL part of the concrete structures in the world are built in areas with low winter temperatures. The temperature strongly influences the material properties. Experiments performed at Luleå University of Technology[1–7] have been focused on fracture mechanics properties at low temperatures. Ordinary concrete, high strength concrete, lightweight and fibre reinforced concrete have been tested.

TEST SPECIMEN

Four concrete qualities C25, C40, LC45 and C100 have been used in the investigations (Table 1). Unreinforced concrete beams and cubes have been cast in steel moulds. The beams had the dimensions 500 × 80 × 50 mm in the first two series. In the following series they had the dimensions 840 × 100 × 100 mm according to RILEM's recommendations[8]. After the casting the test specimens were covered with plastic film and kept wet. The forms were removed after three days and the specimens were then cured in water. At the end of the curing time notches were sawn in the beams (10 mm in the small beams and 50 mm in the other beams).

The tests were carried out in a climate chamber equipped with mobile test rigs (Fig. 1). Series C40 A–C were tested with a 130 kN servohydraulic Instron actuator. The other series were tested in an ordinary 200 kN hydraulic press. The midspan deflection were measured with two LVDTs (Linear Variable Differential Transducers) and the load was registered with a 20 kN loadcell.

Most of the beams were tested when they had dried in the laboratory for about a week. They then had a moisture content of about 5% (i.e. the concrete lost 5% of its weight if it spent 72 h in an oven at +105°C). In two series also wet beams were tested (i.e. the beams were taken from a water basin and were kept sealed in plastic wrappings in the climate room up to testing).

Concrete compressive strength f_{cc} and concrete tensile splitting strength f_{csp} from the tests are given in Tables 2 and 3.

FRACTURE ENERGY

Typical load–deformation diagrams for tests with C100 concrete under varying temperatures are given in Fig. 2. Fracture energy values G_{FE} derived from the experiments according to [8] are given in Fig. 3 and in Table 4.

The results show that the fracture energy G_{FE} increases considerably when concrete with some moisture in it is tested at low temperatures. This increase may be attributed to the load-carrying capacity the ice skeleton develops when the water in the pore system freezes. Similar effects have been observed in tests on the compressive and tensile strength by several investigators, see e.g. [7] and [9]. Fracture energy at low temperatures has earlier only been studied by Elices, Planas and Matarana[10] and by Körmeling[11]. Elices *et al.* found that the fracture energy increased in a

Table 1. Concrete mixes

Concrete type Reference number	Cement (kg/m ³)	$\frac{w}{c+s}$ —	$\frac{s}{c}$ —	Max aggregate size (mm)	Age at testing (days)	Notes
C25 [1]	248	0.60	0	8	28	(1)
C40 [1]	378	0.49	0	8	28	(2)
C40A [2]	372	0.48	0	16	39	(3)
B [2]	372	0.48	0	16	42	(4)
C [2]	372	0.48	0	16		(5)
LC45 [3]	375	0.35	0.10	12	44–46	(6)
C100 [4]	470	0.31	0.15	8	49	(7)

Notes: w = water, c = cement, s = silica [kg/m³]. (1) Air entrainer 74.4 g/m³, moisture content 6.4% at testing. (2) Air entrainer 113.4 g/m³, moisture content 5.2% at testing. (3) Air entrainer 50 g/m³, moisture content 5.0% at testing. (4) Air entrainer 39.1 g/m³. (5) Air entrainer 39.1 g/m³, quick hardening cement. (6) Leca 4–8 mm 300 kg/m³, Leca 8–12 mm 300 kg/m³, plasticizer 3.75 kg/m³ (Rescon P). (7) Plasticizer 10 kg/m³.

Table 2. Concrete compressive strength f_{cc} [MPa] at varying temperatures

Concrete type Reference number	T (°C)	f_{cc} (MPa)	n —	T (°C)	f_{cc} (MPa)	n —	T (°C)	f_{cc} (MPa)	n —
C25 [1]							+20	30.3	3
C40 [1]							+20	43.5	3
C40A [2]							+20	52.3	3
B [2]							+20	53.1	3
C [2]							+20	64.0	3
LC45D [3]	−30	84.1	1	−15	80.6	2	+20	69.7	1
W [3]	−30	>88.9	1	−15	79.4	2	+20	59.7	3
C100D [4]	−30	>108	1	−15	104	1	+18	103	2
W [4]	−30	97	1	−15	94	1	+18	96	2

relation 3 to 1 on lowering the temperature from 20 to −170°C. We found increases of 20–100% when we lowered the temperature from +20 to −30°C.

FATIGUE STRENGTH

Cyclic tests were performed with constant amplitudes at several load levels P_{max}/P_0 , where P_{max} is the maximum load in a cyclic test and P_0 is the peak load in a corresponding test with a monotonically increasing load. A saw-tooth type of waveform was used in the cyclic tests with a frequency of 1/15 Hz and a minimum load P_{min} of 0.1 kN. Typical results of a test are given in Fig. 4. In Table 5 all the tests are summarized. The test results are also shown in the form of Wöhler diagrams in Figs 5–7.

Table 3. Concrete tensile splitting strength f_{csp} [MPa] at varying temperatures

Concrete type Reference number	T (°C)	f_{csp} (MPa)	n —	T (°C)	f_{csp} (MPa)	n —	T (°C)	f_{csp} (MPa)	n —
C40A [2]	−24	5.4	3				+20	4.2	3
B [2]	−24	6.3	6				+20	4.0	3
C [2]	−24	—	—				+20	3.8	5
LC45D [3]	−30	6.1	3	−15	6.0	2	+20	4.8	2
W [3]	−30	7.4	2	−15	7.2	2	+20	4.9	1
C100D [4]	−30	8.4	3	−15	7.7	2	+18	5.8	6
W [4]	−30	9.0	2	−15	7.5	2	+18	7.1	5

Fig. 1. Test arrangement in climate room[2, 6].

Fig. 2. Load-deflection curves for C100 at varying temperatures[4].

Figure 6 indicates that the fatigue strength increases when the temperature is lowered. The scatter in these results is larger than the scatter in tests performed at ordinary temperature. One of the explanations to the raised fatigue strength at low temperatures can be found in the mechanisms of crack propagation during fatigue loading. A fatigue failure often shows a more extensive crack pattern than a static failure. The concrete is more “cracked” after a fatigue failure. This cracking occurs in the many imperfections, pores and interface zones of the aggregate particles. In a frozen concrete with high moisture content large parts of the pore system are filled with ice.

Fig. 3. Experimentally determined fracture energy G_{FE} as function of temperature for different concrete qualities[2, 7].

Table 4. Experimentally determined fracture energy G_{FE} [N/m] at varying temperatures

Concrete type	T	G_{FE}	s	n	T	G_{FE}	s	n	T	G_{FE}	s	n
Reference number	(°C)	(N/m)	(N/m)	—	(°C)	(N/m)	(N/m)	—	(°C)	(N/m)	(N/m)	—
C25 [1]	−35	141	15	5					+20	107	10	5
C40 [1]	−35	178	37	5					+20	100	9	5
C40A [2]	−24	200	29	5					+20	84	7	4
B [2]	−24	163	17	3					+20	94	20	3
C [2]	−24	219	22	6						—		—
LC45D [3]	−30	94	11	6	−15	54	7	3	+20	45	2	3
W [3]	−30	95	23	4	−15	55	4	4	+20	46	2	3
C100D [4]	−30	168	23	3	−15	155	2	3	+18	139	5	3
W [4]	−30	134	5	4	−15	117	17	4	+18	91	23	4

D = “dry” (about 5% moisture content).
W = “wet”.
 s = standard deviation.
 n = number of tests.

The cement paste becomes more dense and the concrete more homogeneous. In that concrete fewer stress concentration occurs. The cracking will therefore not be so extensive during fatigue loading. Another explanation is that the cracks in the ice are healed during the unloading cycles when compressive stresses are active.

For ordinary temperature the fatigue life length is sometimes written as[12, 13]

$$\log N = C \frac{1 - S_{\max}}{1 - S_{\min}/S_{\max}} \tag{1}$$

or rearranged as

$$S_{\max} = 1 - \frac{1}{C} \left(1 - \frac{S_{\min}}{S_{\max}} \right) \log N \tag{2}$$

Fig. 4. Load–deformation curves for C40 at varying temperatures[2, 6].

Table 5. Fatigue tests

Test No.	Batch No.	− 24°C			Batch No.	+ 20°C		
		P_{max} (N)	P_{max}/P_0	N		P_{max} (N)	P_{max}/P_0	N
1	A	1412	85	120	A	808	92	12
2	B	1412	85	50,150	A	803	92	26
3	A	1412	85	> 25,000	B	750	86	89
4	B	1412	85	16,707	A	750	86	102
5	A	1578	95	12,244	B	745	85	> 55
6	B	1661	100	1	B	745	85	81
7	A	1661	100	33	A	745	85	32
8	B	1578	95	> 22,345	B	723	83	293
9	B	1661	100	> 15,384	B	709	81	638
10	B	1661	100	1	A	704	80	530
11	A	1661	100	1				
12	B	1628	98	108				
13	C	1618	90	1,919				
14	C	1618	90	101				
15	C	1618	90	> 215				
16	C	1618	90	4				
17	C	1618	90	4				
18	C	1618	90	2,317				
19	C	1618	90	401				

N = number of cycles before failure.

where

- N = number of loadings to failure,
- C = coefficients depending on environment and stress conditions. Normal values range between 10 and 20,
- $S_{max} = \sigma_{max} / (k_1 f_{ck})$,
- $S_{min} = \sigma_{min} / (k_1 f_{ck})$,
- σ_{max} = maximum stress,
- σ_{min} = minimum stress,
- f_{ck} = concrete strength,
- k_1 = reduction coefficient to be used in design codes.

For concrete in tension or compression or subjected to stress reversals at normal temperatures Tepfers has proposed $C = 14.6$ ($1/C = 0.0685$)[12, 14]. Using this value and P_{max}/P_0 ,

Fig. 5. Cyclic beam tests at +20°C[2].

Fig. 6. Cyclic beam tests at -24°C [2].

$P_{\min}/P_0$ instead of $S_{\max}$ and $S_{\min}$ we obtain with eq. (2) the following expression when $P_{\min}/P_{\max} = 0.1 \text{ kN}/0.8 \text{ kN} = 0.125$:

$$P_{\max}/P_0 = 1 - 0.0685(1 - 0.125)\log N = 1 - 0.060 \log N. \quad (3)$$

This line has been drawn in Fig. 5 and it gives a fairly good correspondence with the tests at $+20^{\circ}\text{C}$.

For the tests at -24°C we obtain in the same way with $P_{\min}/P_{\max} = 0.1 \text{ kN}/1.6 \text{ kN} = 0.0625$ that

$$P_{\max}/P_0 = 1 - 0.0685(1 - 0.0625)\log N = 1 - 0.064 \log N. \quad (4)$$

This line has been drawn in Fig. 6. However it seems to be too conservative. A higher value of C is called for. With $C = 20$ we obtain

$$P_{\max}/P_0 = 1 - \frac{1}{20}(1 - 0.0625)\log N = 1 - 0.047 \log N. \quad (5)$$

This line has also been drawn in Fig. 6 and it gives a better correspondence with the test results.

Fig. 7. Cyclic beam tests at $+20^{\circ}\text{C}$ and -24°C referred to static peak load P_0 at $+20^{\circ}\text{C}$ [2].

DISCUSSION AND CONCLUSIONS

For concretes with a high moisture content both the experimentally determined fracture energy G_{FE} and the fatigue strength are increased when the temperature is lowered from $+20$ to -30°C . This is probably due to a contribution from the ice that is formed in the concrete. The positive effect may be reduced by repeated freeze-thaw cycles and the combined effect of fatigue loading and freeze-thaw cycles ought to be studied.

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