EFFECT OF STEEL FIBRE ON CRACKING STRENGTH OF REINFORCED CONCRETE STRUCTURES

Pekka Sipari
M.Sc.(Tech.), Research Scientist
VTT Building Technology
P. O. Box 1804 (Lämpömiehenkuja 3, Espoo)
FIN-02044 VTT, Finland
Tel +358 0 4561
Fax +358 0 456 4709

Matti Lanu
M.Sc.(Tech.), Research Scientist
VTT Building Technology
P. O. Box 1803 (Kivimiehentie 4, Espoo)
FIN-02044 VTT, Finland
Tel +358 0 4561
Fax +358 0 456 4815

Seppo Huovinen
Dr.Tech., Associate Professor
Helsinki University of Technology
Rakentajanaukio 4
FIN-02150 ESPOO, Finland
Tel +358 0 451 3718
Fax +358 0 451 3724

ABSTRACT

Five specimens were tested under tensile load and five specimens under bending load. All the specimens were reinforced with 12 mm deformed bars. The concrete mix used in the specimens contained no fibre, 1.0 volume-% of steel fibre or hybrid fibre (0.5 volume-% steel fibre and 0.5 volume-% polyacrylonitrile fibre). During the hardening phase and loading tests the cracking behaviour of the specimens was studied.

The test results showed that the fibre can bridge the cracks to a certain extent and thus limit the crack width, the number of cracks and the depth of the cracks. It was also noticed that in the conventionally reinforced members there are obviously shrinkage stresses which relax due to the cracking, and thus open the cracks wider in the specimens without added fibre.

key words: Fibre-reinforced concrete, Crack width, Crack space

1. INTRODUCTION

Fibres are utilised in practical applications such as ground floors, highways etc. In structures such as reservoirs and containers the crack width and the crack spacing are usually controlled by adding auxiliary reinforcing bars. This might lead to such tight reinforcement that the casting and compacting of the concrete is difficult. Here it is supposed that fibres also have good possibilities.

In the literature there are a lot of results concerning fibre-reinforced concrete. There is also a lot of knowledge of the basic properties of fibre-reinforced concrete, but more advanced design methods for the fibre-concrete structures are needed.

Especially the cracking behaviour of the fibre concrete structures of practical size, with normal reinforcement, designed with national codes and manufactured with commonly used concrete types used in Finland was noticed to be inadequately researched.

2. TEST PROCEDURE

2.1 Materials and test specimens

The properties of basic mix are reported in /1/, except mix C.

The fibre types and reinforcement bars used were:

steel: Dramix ZP30/.05 by Bekaert; length 30 mm, thickness 0.5 mm.
PAN: Dolanit T11 by Hoechst, length 12 mm, thickness 0.104 mm.
rebars: type A 500 HW, yielding strength 500 MPa, E-modulus 200 000 MPa.

The proportioning of the concrete mixes is presented in Table 1. The basic mix was proportioned with a rather low water / cement ratio to give a strength of about 70 MPa. The specimens are illustrated in Fig. 1.

Table 1 Concrete mixes and their use in test specimens.

	Mix A	Mix B	Mix C
Tensile test specimens	AV1, AV2	BV1, BV2	CV
Bending test specimens	A1, A2	B1, B2	C
Constituents	kg/m^3	kg/m^3	kg/m^3
Portland cement	350	350	350
Silica	35	35	35
Water	130	130	130
Super plasticer	23.5	23.5	23.5
Dramix 30mm	-	78 (1vol%)	39 (0.5vol%)
Dolanit 12mm	-	-	5.9 (0.5vol%)
Filler	320	320	320
0.1-0.6 mm	195	195	195
0.5-1.2 mm	213	213	213
1-2 mm	142	142	142
2-3 mm	320	320	320
3-5 mm	354	354	354
5-10 mm	391	391	391
Aggregate total	1 935	1 935	1 935

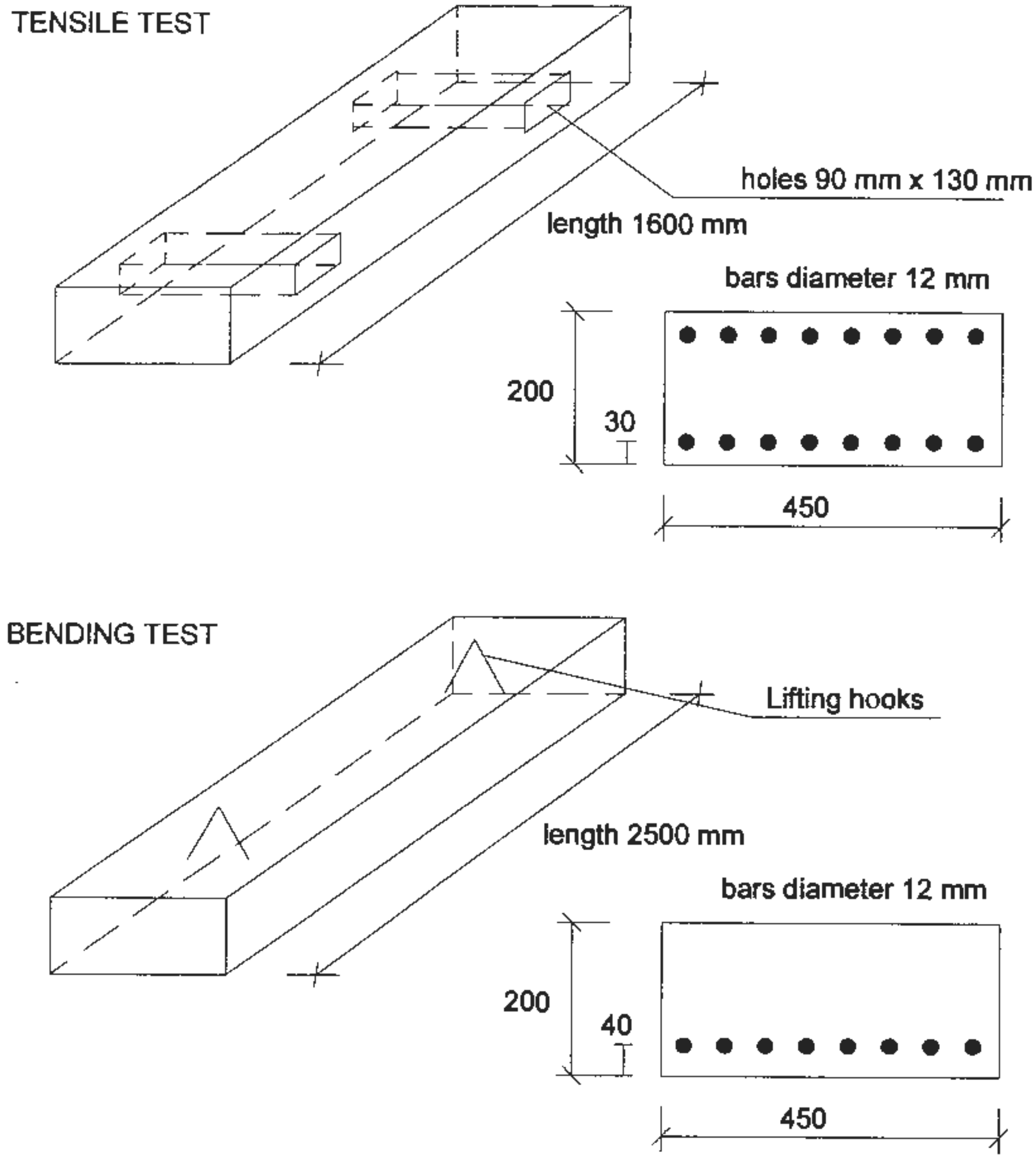


Fig. 1. Test specimens.

The concrete was mixed with a paddle mixer. The fibre was added after five minutes of mixing followed by continued mixing for another five minutes. The mix was cast into plywood moulds and compacted using rod vibration. The surface was finished with a steel trowel. The specimens were stored for three days in the laboratory (60 % RH, 20 °C) and covered with a plastic sheet, except two bending test specimens (A1 and B1) which were uncovered. After three days the specimens were demoulded and transferred into the testing hall (50 % RH, 20 °C) and stored uncovered.

2.2 Tests and arrangements

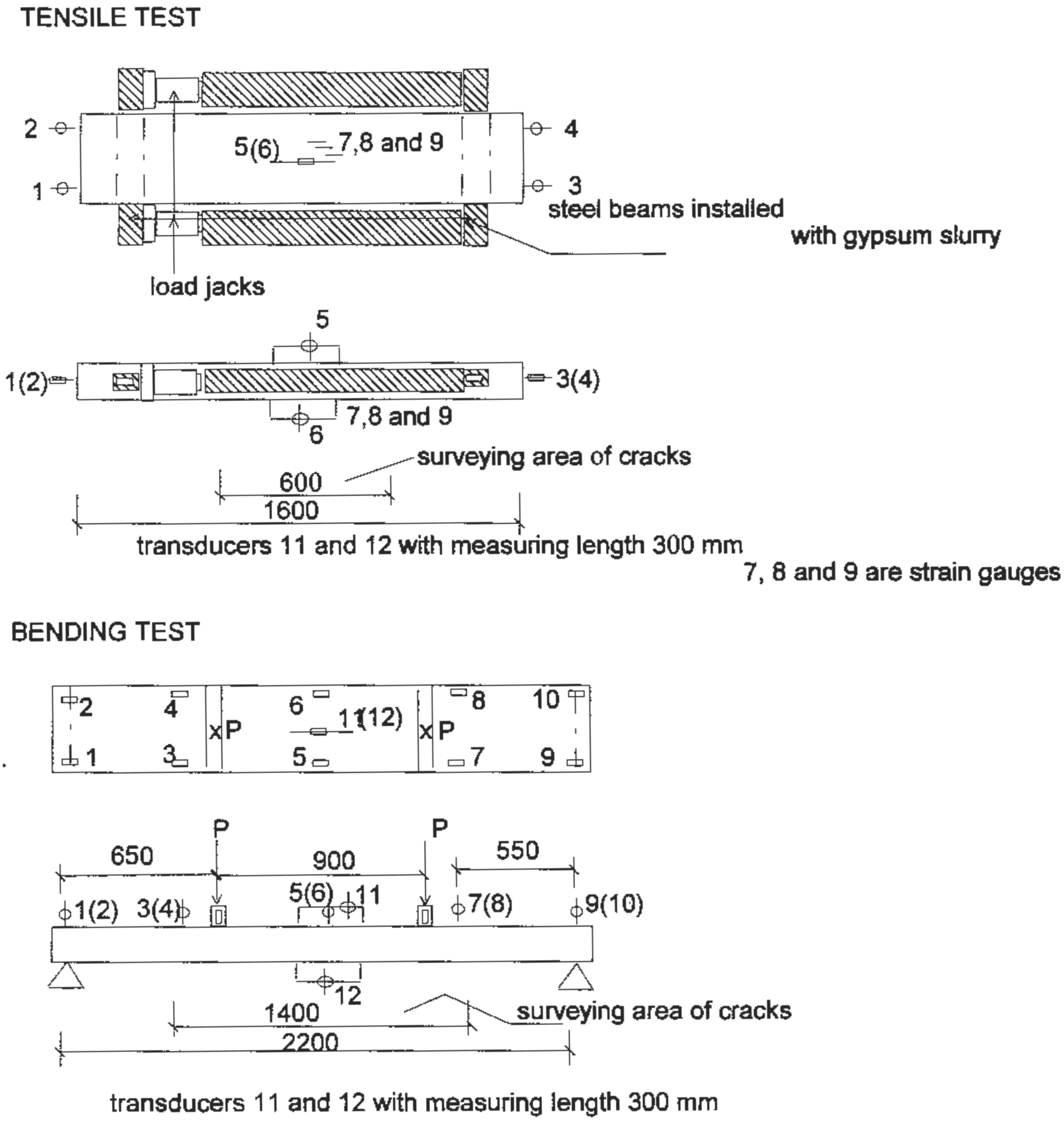
The centric tensile load was applied to the specimen by the help of I-sectioned steel beams in holes and by hydraulic jacks, see Fig. 2.

The load was applied step-wise. The displacements and the strains were recorded at each load step. The strain on one side of each specimen was measured with three Omega-type strain gauges with a measuring length of 50 mm. The gauges were placed along a line after another. At each load step the appearance of the cracks was surveyed visually.

The span of the <u>bending test</u> specimens were 2.2 m. Hence there was a 0.9 m long area of constant bending moment in the middle of the beam, see Fig. 2.

The load was applied step-wise. The deflections at the ends, at the mid-span and at 500 mm apart from the supports as well as the strains on both sides (in the middle) of the specimen were recorded at each load step.

At each load step the appearance of the cracks and their depths were surveyed. Almost at each load step the widths of the eight chosen cracks were measured.



transducers 11 and 12 with measuring length 300 mm

7, 8 and 9 are strain gauges

transducers 11 and 12 with measuring length 300 mm

Fig. 2. Test arrangements

3. RESULTS

3.1 Notes on mixing and on workability of concrete

There were no difficulties in the mixing process. The concrete without fibre (mix A) had the highest slump, but there was still no noticeable segregation of water.

In the case of fibre-concretes, the moulds were filled with spades while the casting of the plain concrete could be done from the skip. There were some difficulties in making the fibre-concrete properly fill the places under the holes due to the thick consistency and setting of the masses in the early state. The main results from the tests with concrete masses are presented in Table 2.

Table 2. Properties of fresh concrete.

METHOD	PROPERTY	mix A no fibres	mix B 1.0%	mix C 0.5+0.5%
SFS 5288 (ISO 6276)	Density of fresh concrete (kg/m3)	2439	2329	2319
SFS 5287 (ISO 4848)	Air content (%)	3.2	7.0	6.0
SFS 5285 (ISO 4110)	Vebe test (sVb)	1.1	5.9	5.5
SFS 5284 (ISO 4109)	Slump (mm)	170	45	40

3.2 Basic properties of hardened concrete and shrinkage behaviour

The basic properties of the hardened concrete are presented in the next table.

Table 3. Properties of hardened concrete.

METHOD	PROPERTY	mix A no fibres	mix B 1.0%	mix C 0.5+0.5%
SFS 5442 (ISO 6275)	Density of concrete (kg/m3)	2440	2410	2410
SFS 5444 (ISO 4013)	Flexural strength (MPa)	9.0	12.6	7.1
SFS 5444 (ISO 4013)	Compressive strength (MPa) [1]	71	68	79
SFS 4474 (ISO 4012)	Compressive strength (MPa)	75	65	
SFS 5450 (ISO 6784)	Elasticity modulus (MPa)	34200	31500	
SFS 5443 (ISO 4108)	Splitting strength (MPa)	5.1	4.8	

For all the concrete mixes a free drying shrinkage value was 0.4...0.45 mm/m after 8 weeks. (RILEM CPC 9 [with the exception of RH 40% when drying the specimens]). The results from the restrained shrinkage tests with mix A and mix B showed that there was no visible cracking in the steel fibre-reinforced concrete while the concrete without fibres cracked /1/.

3.3 Cracking of the specimens due to shrinkage

A summary of observed cracking during hardening is given in table 4. These initial cracks observed are not included when the cracking properties are analysed further.

---

[1] Compressive tests were performed on specimens sawed from the ends of prisms used in flexural test

Table 4. Observed crack widths (mm) in test specimen during hardening.

Fibre content	0%		1%		0.5+0.5%	0%		1%		0.5+0.5%
specimen	A1	A2	B1	B2	C	AV1	AV2	BV1	BV2	CV
Age	(uc)		(uc)							
4 hours RH 60% 20 °C	-	-	-	-	-	-	-	-	-	-
3 days plastic film except (uc) RH 60% 20 °C	0.02 to 0.04	-	-	-	-	-	-	-	-	-
1 month RH 50% 20 °C	0.04 + 0.08	0.02 + 0.14	0.06	-	0.04	0.1	-	0.06	-	-

(uc) means that specimens were stored uncovered during the first three days.

3.4 Tensile test

In Fig. 3 the average strains at the mid-span are presented. The measuring length of elongation was 300 mm, thus the strain also includes the effect of the cracks between the measuring points.

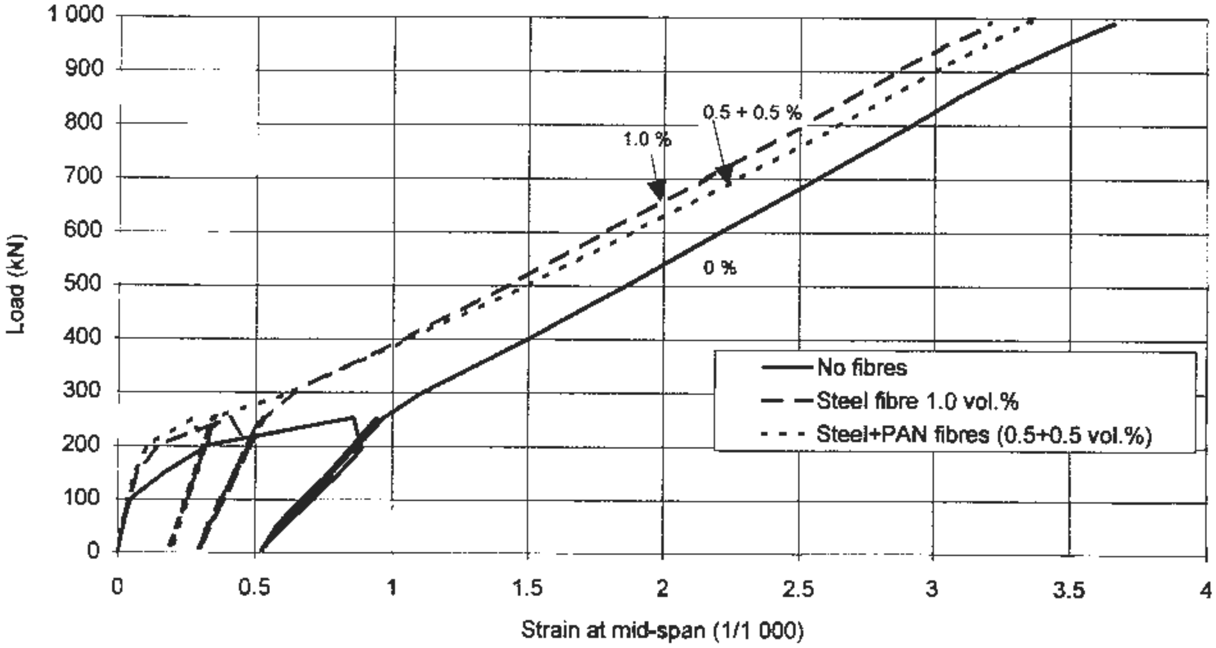


Fig. 3. The load - average strain curves of tensile specimens.

The first cracks in the specimens without fibre (AV1, AV2) or with 1.0% steel fibre (BV1, BV2) were observed at load 150 kN. The specimen with hybrid fibre (CV) cracked at load 250 kN.

The average number of the cracks is presented in Fig. 4 as a function of load. This has been calculated by dividing the number of the observed cracks by the distance they covered. The average widths of the cracks are presented in Fig. 5 (the first load cycles are not included).

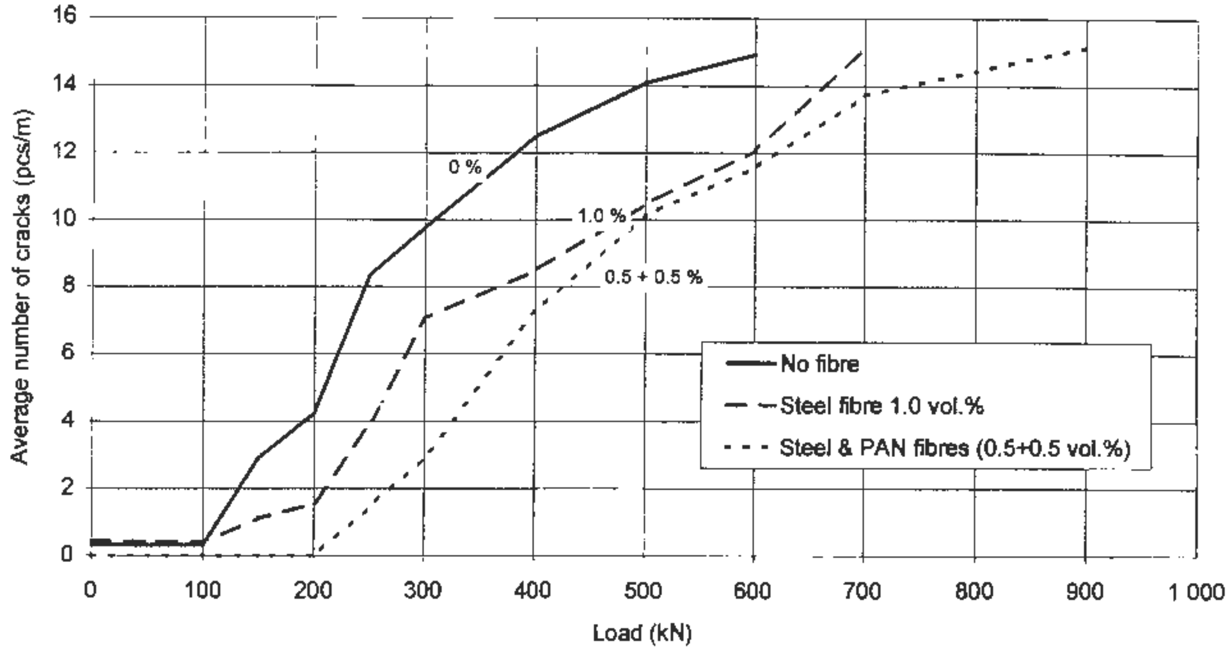


Fig. 4. Average number of cracks on one side of specimen as a function of load.

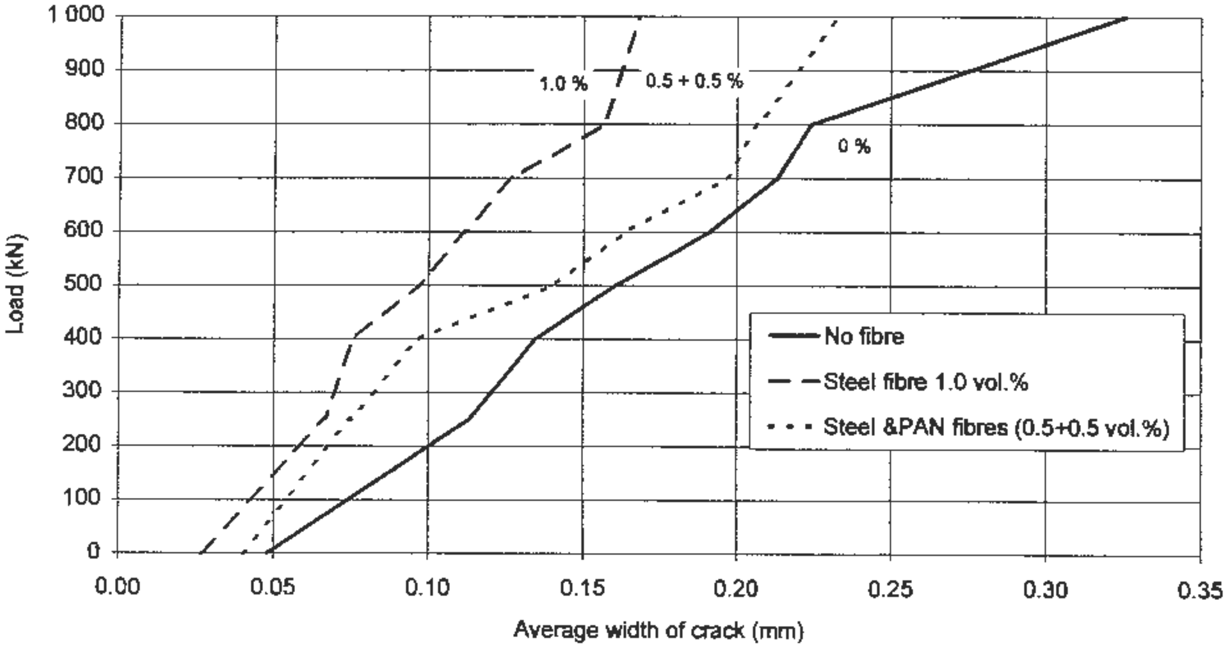


Fig. 5. The load as a function of an average width of crack.

3.5 Bending test

For the specimens with no fibre (A1, A2) and specimens with hybrid fibre (C) the load deflection curve was linear up to 115 kN/jack. The deflection in the middle was about 18 mm. After this, the reinforcing steel bars began to yield, the deflection increased rapidly and the depth of the neutral axis rose step by step. The failure was classified as a yielding failure.

The specimens containing 1 vol. % steel fibre (B1, B2) acted linearly up to 125 kN/jack and 18 mm deflection. After this the behaviour differed from previously. Because of the fibre acting on the tensile side the load increased to a higher level also causing the stresses in the compression zone to be higher. Due to the higher stresses and the possible slight decrease in the strength of the compression zone, both beams failed after a limited yielding of reinforcing bars by the failure at the compression zone. One can classify these as yielding failures. The load deflection curves are shown in the Fig 6.

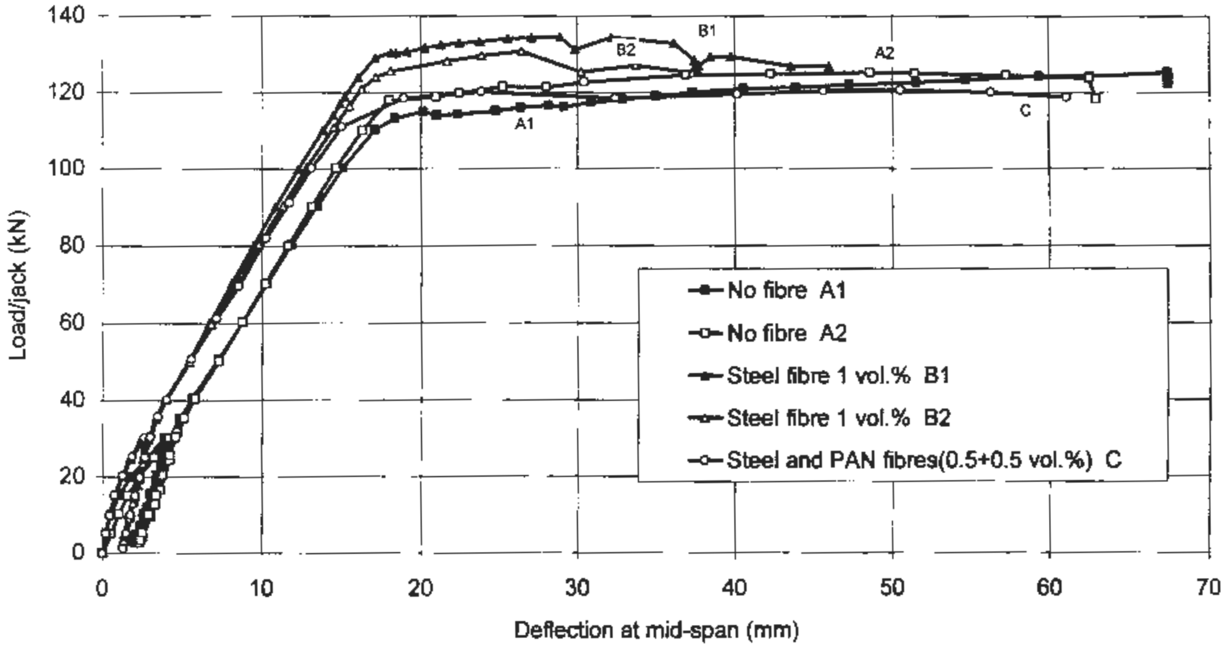


Fig. 6. Deflection of beams at mid-span as a function of load.

The strain in the mid-span was measured by the use of transducers with measuring lengths of 300 mm. Thus the strain contains the effect of cracks within the measuring length, see Figs. 7 and 8.

The first cracks appeared in specimens under loads 10...20 kN/jack when they were loaded for the first time up to the service load. If cracks with a measurable depth are considered as the first cracks it can be concluded that the cracking load of specimens with fibre was about 20 kN/jack giving about 13 kNm for cracking moment. The cracking load of the specimens without fibre was about 15 kN/jack giving about 10 kNm for cracking moment.

The results for the average number of cracks is presented in Fig. 9, the average crack depth in Fig. 10 and the average crack width in Fig. 11 as a function of bending moment.

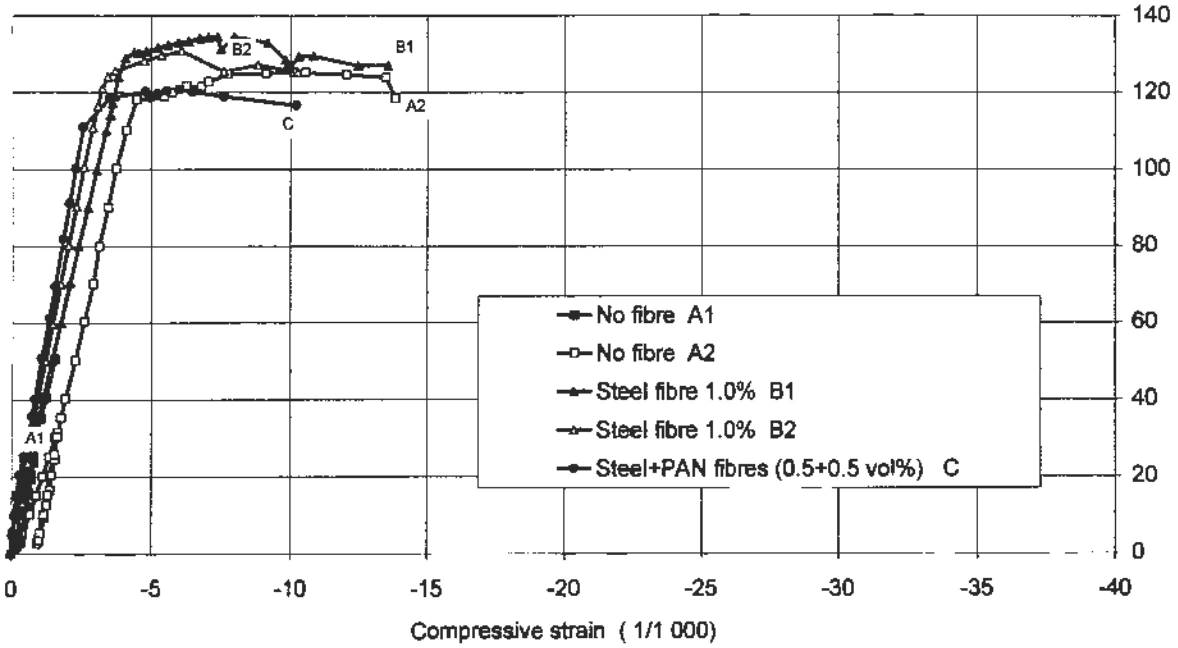


Fig. 7. Compressive strains of beams at mid-span as a function of load.

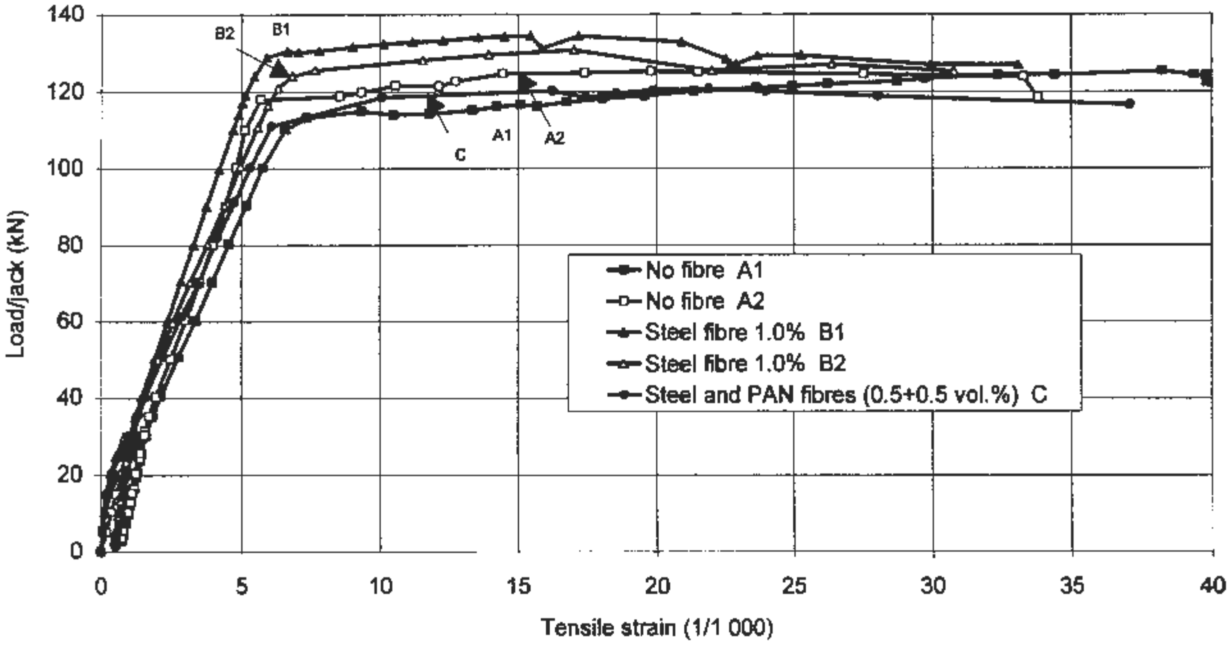


Fig. 8. Tensile strains of beams at mid-span as a function of load.

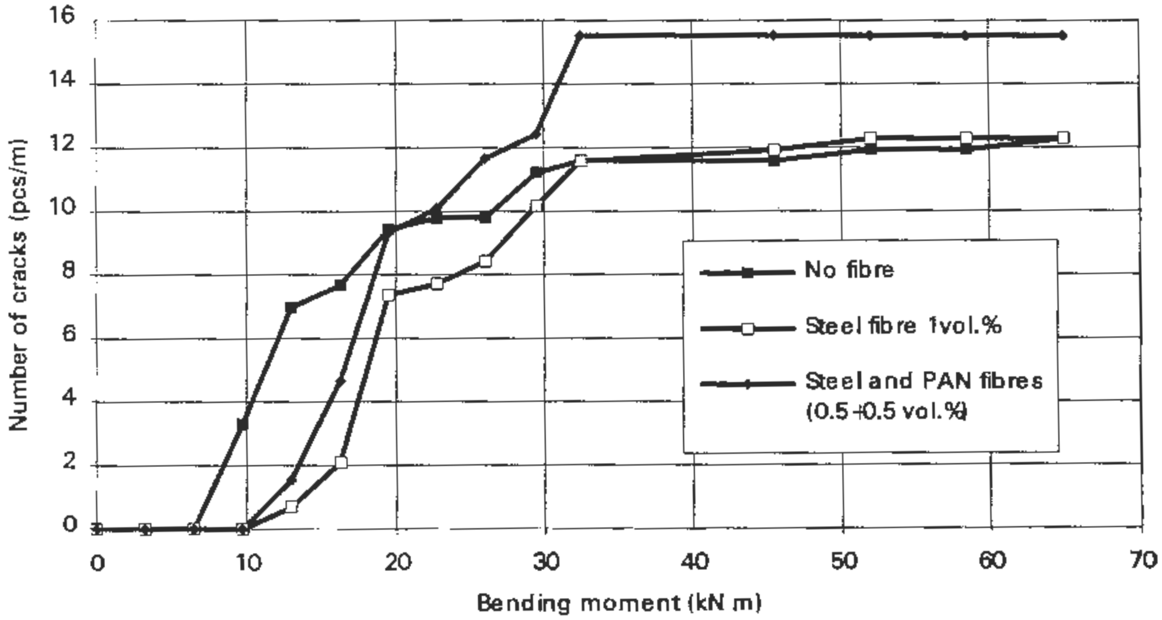


Fig. 9. Average number of cracks in beams as a function of bending moment.

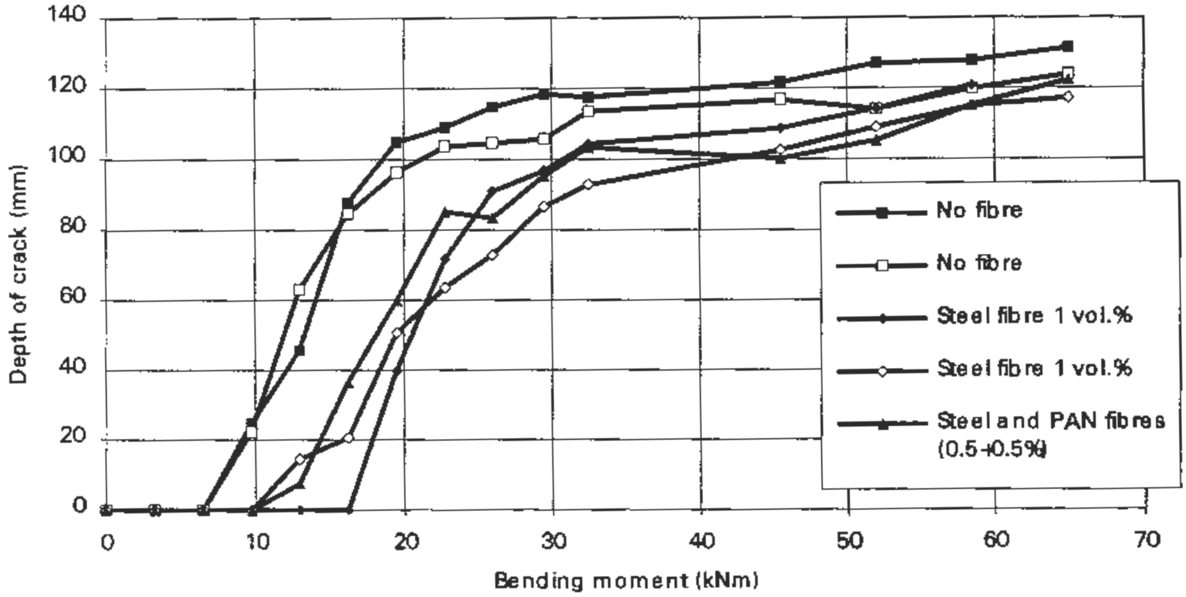


Fig. 10. Average depth of cracks as a function of bending moment.

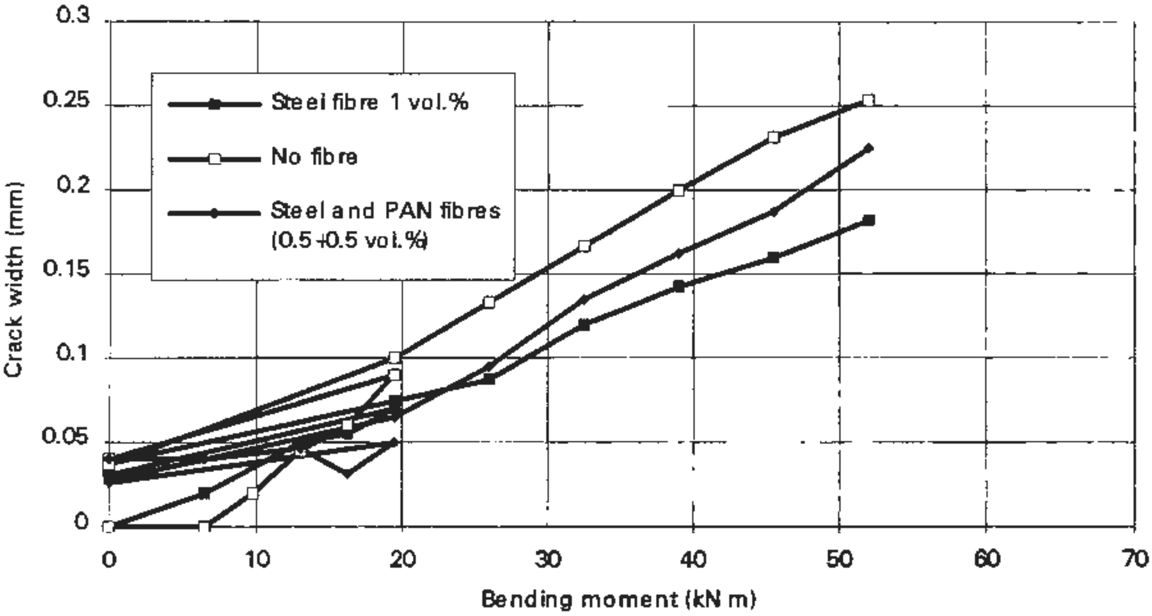


Fig. 11. Mean crack width in bending tests as a function of bending moment.

4. DISCUSSION

4.1 Tensile tests

The fibre has effectively limited the number of the cracks in tension. At a service load of 250 kN, the number of cracks was limited to 20 - 40 % and at a load of 500 kN (about half of the failure load) to 70% from that of non-fibrous specimens.

The crack widths of specimens with steel fibre were on average 40% narrower than those observed in the specimens with no fibre during the loading time. The crack widths of specimens with hybrid fibres were 10-30% narrower.

The measurement using strain gauges clearly shows that the tensile strains (most of the gauges) began to grow strongly at load levels of 200-300 kN, which indicates cracking of the concrete matrix. After some increase in the load, cracks appeared within the measuring length of most of the strain gauges, and the gauges showed tensile strains. With three strain gauges the measuring length stayed uncracked. Two of them in specimens without fibre showed compressive strains of the order of 0.0015-0.002, while one gauge in one of the specimens with 1.0 vol. % steel fibre showed tensile strain.

Based on these measurements, it is evident that there were initial tensile stresses and strains due to shrinkage in the conventionally reinforced concrete specimens. These stresses were released by crack formation process, which leads to wider cracks. It is obvious that the fibre had bridged the cracks and transferred tensile stresses. Consequently fibre prevents, to some extent, the cracks from opening.

The effect of the released shrinkage stresses and strains could explain the observed differences in the permanent deflection or strain after cyclic loads. It also partly explains the differences in the crack strengths of the concrete matrix between practical size specimens and small laboratory specimens. For instance, it can be calculated from the tensile loads of 150...250 kN, that the cracking stress of the tensile specimens were 1.7...2.8 MPa, which are rather low values for concrete with a compressive strength of 60-70 MPa. According to the Finnish codes /2/ the value of the characteristic tensile strength, f_{ctk}, for concrete grade K 60 is estimated by the following equation

$$f_{ctk} = \alpha \ K^{2/3} = 0.2 \cdot 60^{2/3} = 3.1 \text{ MPa} \tag{1}$$

So the characteristic value of the cracking tensile load capacity should be:.

$$N_r = f_{ctk} A_c = 3.1 \text{ MPa} \cdot 90000 \text{ mm}^2 = 280 \text{ kN} \tag{2}$$

where A_c is the cross-sectional area of the concrete.

In all cases the observed cracking load was lower than this value.

4.2 Bending tests

The load-deflection behaviour was in all bending specimens, nearly the same. Above the service load state, the fibre limited the crack widths by roughly 20-30 %.

The fibre limited both the depth of the cracks and the number of cracks. The effect of the fibre was greatest above the estimated service load and near it, and the effectiveness decreased gradually as the load level increased.

All the observed cracking moments were 10-13 kNm. If calculated according to the Finnish codes /2/ the value of the cracking moment capacity M_r should be about:

$$M_r = 1.7 \cdot W_{ce} f_{ctk} = 1.7 \cdot \frac{450 \text{ mm} \cdot 200^2 \text{ mm}^2}{6} \cdot 3.1 \text{ MPa} = 15.7 \text{ kNm} \tag{3}$$

Here W_{ce} is the elastic section modulus at the uncracked stage. The factor 1.7 represents the difference between the concrete strength under flexural loading and tensile loading.

The characteristic crack width, w_k, - in plain concrete - is calculated according to the Finnish code with the formula:

$$w_k = \varepsilon_s \left(3.5 \cdot c + k_w \cdot \frac{\phi}{\rho_r} \right) \tag{4}$$

$$\rho_r = \frac{A_s}{A_{ce}} \tag{5}$$

where A_s is the cross-sectional area of main rebars. A_{ce} is determined according to following figure.

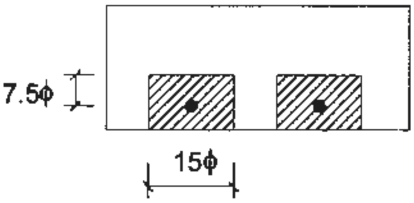


Fig. 12. Determining the area A_{ce} according to the Finnish code.

k_w is a coefficient, depending on the surface of the steel bar (here $k_w = 0.085$), and ϕ is the bar diameter (here 12 mm), and c is concrete cover (here 40 mm). The steel strain ε_s is calculated according to the equation:

$$\varepsilon_s = \frac{\sigma_s}{E_s} \left[1 - \frac{1}{25 k_w} \left(\frac{\sigma_s}{\sigma_{sr}} \right)^2 \right] \tag{6}$$

The steel stress at the cracked stage σ_s and the steel stress just when cracking occurs σ_{sr} are

$$\sigma_s = \frac{M_d}{z_1 A_s} \quad \sigma_{sr} = \frac{M_r}{z_2 A_s} \tag{7}$$

where M_d is bending moment and $z_1 = z_2$ is internal moment arm.

The contribution of fibres could be taken into account by lowering the stress in main reinforcement. The following equation is suggested here to estimate the steel stress $\sigma_{s,fc}$ in the case of fibre-reinforced concrete.

$$\sigma_{s,fc} = \sigma_s - \frac{A_{ce}}{A_s} A V_f f_\tau \frac{l_f}{d_f} \tag{8}$$

A is an efficiency factor (here 0.4).

In Figure 13 the maximum crack widths observed in the bending tests are presented, and they are compared to those calculated using Finnish codes. Also the suggested theoretical values according to the previous equation are presented with the notation F().

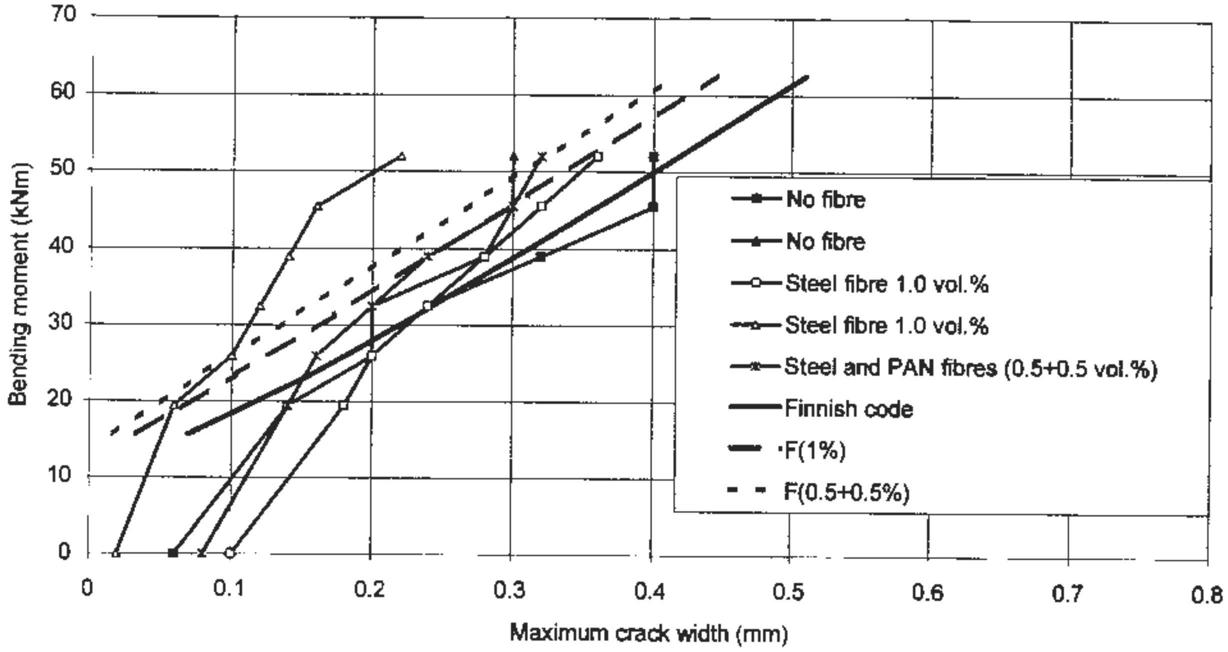


Fig. 13. Maximum crack widths and a theoretical value according to Finnish codes.

In the equations above:

ρ_r	Reinforcement ratio
ϕ	Bar diameter
M_d	Bending moment
A_s	Amount of reinforcement steel in cross section
$z_1=z_2$	Internal moment arm

It can be seen that the formula above represents the short-term maximum crack widths quite well for plain concrete.

5. CONCLUSIONS

Based on test results it can be stated that the fibre transfers stresses over cracks. At the same load-level the cracks of specimens with fibre were narrower, the number of the cracks fewer and the depth of cracks of the bending specimens with fibre were lower compared to specimens without fibre.

Test results give some evidence that shrinkage stresses remaining in the reinforced concrete may have a role in the crack formation and widening process. The fibre limited the number of cracks under external loading, but they had no effect on the spacing of the cracks near the failure loads.

The test specimens used had a low reinforcement ratio. The effect of fibre on the cracking properties should be examined more in the future with steel ratios near the balanced steel ratio. A preliminary consideration is that the effect of fibre (content 1.0 vol. %) decreases when the reinforcement ratio increases in structures under bending loads.Thus, it can be concluded that the fibre may be most effective in cases when the reinforcing amount of structure is small (floors for instance).

It is also widely known that steel fibre reduces cracking due to shrinkage and that concrete floors with fibre are not so vulnerable to lift-up effects in the floor corners for instance.

On the above basis it can be concluded that fibres have beneficial effects on the ductility of concrete structures. This is especially true in the case of floors which in many cases need small amounts of conventional reinforcement.

A practical design tool is needed to estimate first the maximum crack width and then probably the crack spacing in a full-scale structure. The information available to the designer is usually only the concrete grade, structural dimensions, type and dimensions of rebars, type and content of fibres and external loadings.

6. ACKNOWLEDGEMENTS

This research was funded by the Technical Research Centre of Finland. The structural and material tests and analysis, as well as the manufacturing of the test structures were carried out in VTT Building Technology.

7. REFERENCES

/1/ Vares, Sirje. Kuitubetonien muodonmuutoskäyttäytyminen [Dimension stability of fibre reinforced concrete]. Espoo 1995, Valtion teknillinen tutkimuskeskus, VTT Tiedotteita - Meddelanden - Research Notes 1645. 33 p. + app 9 p. (in Finnish)

/2/ Betonirakenteet. Ohjeet 1990. [Concrete Structures. Codes 1990] Helsinki 1990, Ympäristöministeriö, Suomen rakentamismääräyskokoelma, Osa B 4. 80 pp.