

Replacement of minimum steel bar reinforcement with steel fibres in structural concrete members

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Abstract. Using lightweight concrete enables a decrease in dead load and thermal conductivity in the case of the manufacturing of structural concrete members. With the addition of steel fibres in concrete, its properties are altered from brittle to ductile, so that the use of additional minimum reinforcement for securing ductility and crack control can be avoided. This study is aimed at investigating the possibility of replacing conventional minimum steel bar reinforcement with steel fibre reinforcement in lightweight aggregate concrete under flexural loading. Therefore, six full-scaled beams with two different lightweight aggregate concretes (LWAC) (oven-dry densities of $<1,200 \text{ kg/m}^3$ and $<1,600 \text{ kg/m}^3$) as well as different types of reinforcement were prepared. For each LWAC, a beam with traditional steel bars, a beam with steel fibres and a beam with a combination of steel fibre reinforcement and reduced steel bar reinforcement were produced. The cracking behaviour of the lightweight concrete beams was studied in a four-point bending test. The results of this study show that it is possible to replace a high amount of the conventional mesh or bar reinforcement with steel fibres.

1. Introduction

Lightweight aggregate concrete (LWAC) permits a decrease in dead load. The lighter concrete construction results in a reduced load flow so that successive structural members can have smaller dimensions (especially foundations). Therefore, the costs of the building are reduced. In the precast concrete construction, LWAC offers a cost reduction for transportation [1]. LWAC is suitable for structural members, where the required material strength and required ratio of dead load to live load are reduced [2]. In addition, the lower density improves the thermal insulation.

Despite the low requirements on structural walls, a minimum reinforcement for securing ductility and crack control is demanded by the European Standard (DIN EN 1992-1-1 [3]). Also, further reinforcement is required in areas for openings. With the addition of steel fibres to concrete, its properties are altered from brittle to ductile [4, 5, 6], so the use of additional reinforcement could be avoided. Furthermore, the fibres could minimize the creation of cracks caused by shrinkage of LWAC [7].

This study is aimed at investigating the possibility of replacing conventional minimum steel bar reinforcement with steel fibre reinforcement in lightweight aggregate concrete under flexural loading. Therefore, six full-scaled beams with two different lightweight aggregate concretes



(LWAC) (oven-dry densities of $<1,200 \text{ kg/m}^3$ and $<1,600 \text{ kg/m}^3$) as well as different types of reinforcement were prepared. For each LWAC, a beam with traditional steel bars, a beam with steel fibres and a beam with a combination of reduced steel bar reinforcement and unmodified steel fibre reinforcement were produced. The cracking behaviour of the lightweight aggregate concrete beams was studied in a four-point bending test.

2. Experimental work

2.1. Materials

A Portland blast-furnace slag cement (conforming to DIN EN 197-1 [8] and ordinary fly ash (per DIN EN 450-1 [9]) were utilized as the cement and mineral addition.

Two grades of expanded glass fine lightweight aggregates (LWF1 and LWF2) and one grade of expanded clay coarse lightweight aggregate (LWC) (conforming to DIN EN 13055-1 [10]) are considered in this investigation. In order to achieve a good gradation, normalweight sand (NWF) was added to all mixtures. More information about the aggregates are provided in table 1.

Hooked steel fibres were utilized. They have a tensile strength of 1550 MPa, a length of 60 mm, a diameter of 0.9 mm (aspect ratio $l/d = 67$), an elastic modulus of 200 GPa and a specific gravity of 7.85 g/cm^3 .

Superplasticizer and stabilizer (corresponding to DIN EN 934-2 [11]) were selected to improve the fresh concrete properties. They enhance the workability and inhibit the floating properties of the lightweight aggregates.

Table 1. Aggregate properties

Type	Lightweight			Normal
Designation	LWF1	LWF2	LWC	NWF
Aggregate Size (mm)	0.25–0.50	1–2	2–10	0–2
Particle density (g/cm^3)	0.55	0.35	0.84	2.63
Bulk density (g/cm^3)	0.30	0.22	0.45	1.45
Water absorption (%) ^a	—	—	10	—

^a Water absorption after 60 minutes

2.2. Methods

Four concrete mixes were considered for this study. The development of the mix designs is shown in [12]. The oven-dry density of hardened concrete was targeted to be $1,200 \text{ (D1.2)}$ and $1,600 \text{ kg/m}^3 \text{ (D1.6)}$. Each concrete type was produced without steel fibres and with a steel fibre content of 1.0 vol.-% (F1.0). In this study, a flow table spread of at least 42 cm (16.5") was targeted. Superplasticizer was used at a dosage of 0.5–0.7 % (of the cement content) to achieve this workability.

The coarse lightweight aggregates were added directly (as received) to the mix. The moisture content of the expanded clay aggregates ranged from 4 to 7 % and it was considered in the mix designs.

2.2.1. Production Concretes were mixed in a 1.5 m^3 industrial ready-mixed concrete mixer with the following approach: firstly, dry materials were mixed for 30 seconds; then water was added and mixed for one more minute. Secondly, stabilizer was added and mixed for another minute. Thirdly, superplasticizer was added and mixing was continued for one minute. Finally,

steel fibres were added and mixed until the total time of 5 and a half minutes had surpassed. The concrete mix compositions are presented in table 2.

Table 2. Concrete mix composition^a

Components (kg/m ³)	D1.2	D1.2.F1.0	D1.6	D1.6.F1.0
Cement	350	350	390	390
Fly ash	80	80	80	80
Water	185	185	200	200
NWF	283	279	641	631
LWF1	49	48	—	—
LWF2	53	52	—	—
LWC	239	236	307	302
Steel fibres	—	80	—	80
Superplasticizer (%) ^b	0.50	0.50	0.70	0.50
Stabilizer (%) ^b	0.30	0.30	0.30	0.30
Total	1240	1310	1620	1684

^a Values are rounded

^b Dosage of additives in percentage of cement content

The concrete specimens were cast in plastic moulds (cubes, cylinders and small beams) and wooden formwork (full-scaled beams) and compacted with an internal vibrator. De-moulding was performed after two days. The cubic and cylindrical specimens were kept in water tanks. The small beam specimens were sealed with foil. Full-scaled beams were left in the outside until they moved to the test site. All tests were carried out after 28 days.

2.2.2. Experimental investigations For all the mixtures, fresh and hardened concrete properties were tested. In the case of the workability of fresh concrete, the flow table test (DIN EN 12350-5 [13])) was conducted. The compressive strength (DIN EN 12390-3 [14]) was tested on 150 mm cubes. The Young's modulus (DIN EN 12390-13 [15]) was conducted on cylinders having a diameter of 150 mm and a length of 300 mm. The flexural strength was carried out on 700 x 150 x 150 mm beams. They were tested using the four-point bending test according to the German Committee for Structural Concrete guideline "Steel Fiber Reinforced Concrete" [16].

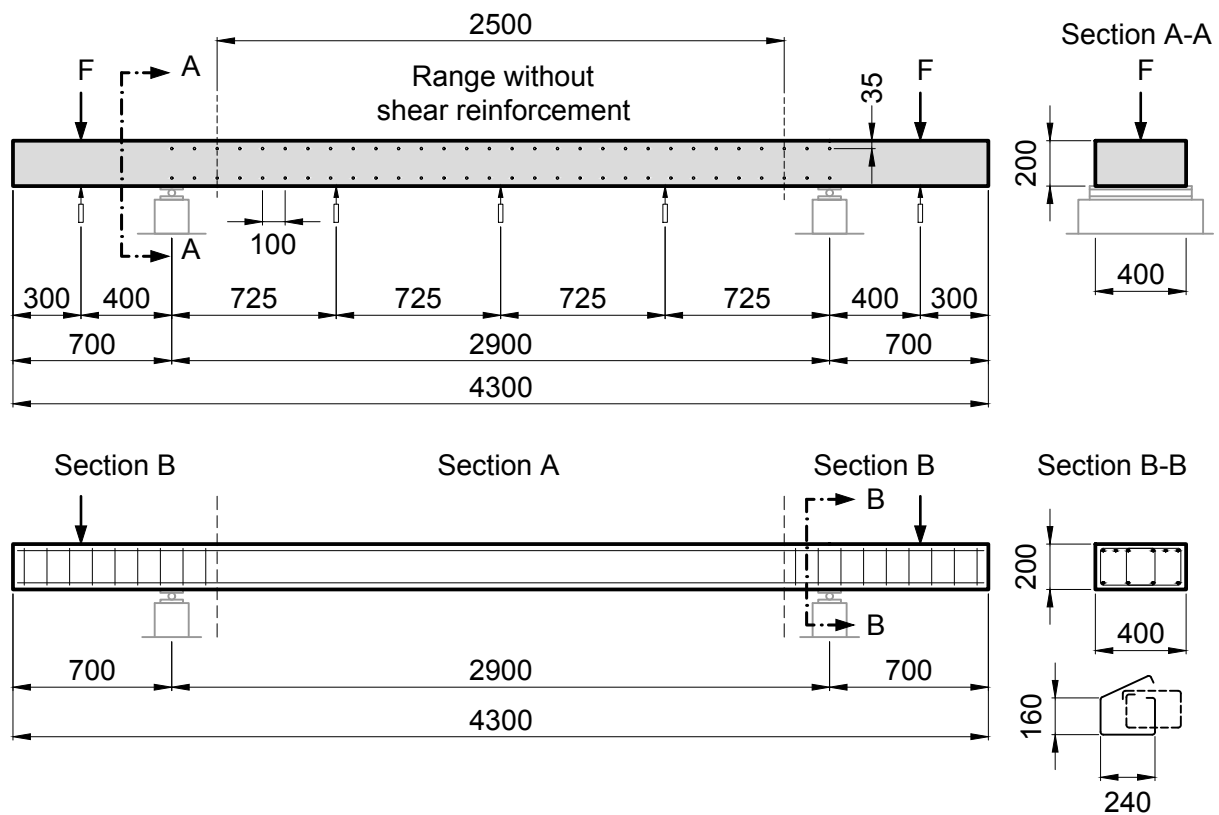
The dimensions of the full-scaled beams, the test set-up and the reinforcement drawing are displayed in figure 1. The load was applied on the two cantilevers of the beam, so that a constant flexural moment was generated between the supports. Table 3 lists the designations and the different characteristics of the specimens.

All six test beams had an rectangular cross-section with $w/h = 400/200$ mm. The concrete cover for reinforcements was 20 mm. In the 2500 mm long measuring range were no stirrups placed, so that the crack development was not disturbed by them. Shear reinforcement was only designated in the range of the cantilevers to cover the shear forces. Furthermore, two extra steel bars were added in the upper layer – from the support to the end of the cantilever – to increase the flexural strength in this area.

The quantity and the diameter of the reinforcement bars (upper & lower) in the measuring range of the beam (section A) is given in table 3. For series D1.2-S and D1.6-S, it was calculated for the minimum reinforcement for crack control ($w_{\max} = 0.3$ mm) according to Eurocode 2 [3].

Table 3. Summary of the experiments

Designation	Concrete type	Reinforcement bars Section A	Steel fibres
D1.2-S	D1.2	4 ϕ 6 mm	—
D1.2-S+F	D1.2_F1.0	2 ϕ 6 mm	1.0 vol.-%
D1.2-F	D1.2_F1.0	—	1.0 vol.-%
D1.6-S	D1.6	4 ϕ 6 mm	—
D1.6-S+F	D1.6_F1.0	2 ϕ 6 mm	1.0 vol.-%
D1.6-F	D1.6_F1.0	—	1.0 vol.-%

**Figure 1.** Test set-up and reinforcement drawing of the full-scaled beams

The steel bar reinforcement in the cantilevers (sections B) are for all beams the same:

- upper reinforcement: 6 ϕ 6 mm
- lower reinforcement: 4 ϕ 6 mm
- shear reinforcement: 9 (18) ϕ 6/100 mm.

For the testing of the beams, the load was continuously applied on the cantilevers until the initial crack formation. This also represents the first load step. Then, the load was displacement

controlled with fixed load steps on specified mid-span deflections of the beam. During the experiments, the following measurements were conducted (see also figure 2):

- the applied load on the cantilevers with two 500 kN load cells;
- the deflection at five locations with 20 mm LVDT's;
- the crack development on the top side and one side of the beam at each load step;
- the crack widths with a crack width gauge at each load step.

3. Results and discussion

3.1. Fresh and hardened concrete properties

Table 4 presents the results of fresh and hardened concrete properties. Here, the average values from three tests for compressive strength and Young's modulus as well as seven tests for the flexural and residual flexural strength are presented.

The analysis of the residual flexural strength tests is performed with the residual load at deflections of 0.5 mm (serviceability limit state) and 3.5 mm (ultimate limit state). These deflections are related to crack widths and calculated strain conditions, which are expected to occur in the serviceability and ultimate limit states. A deflection value δ of 0.5 mm corresponds to a strain value of $\varepsilon = 3.5 \text{ ‰}$. The ultimate strain ε of 25 ‰ was specified for the second deflection value $\delta = 3.5 \text{ mm}$ in case of steel fibre reinforced concrete [17].

3.2. Results from four-point bending tests

The moment–midspan deflection relationship obtained from the four-point bending tests is illustrated in figure 2. The diagrams show all three variations of the specimens — steel bar (4 \varnothing 6 mm), steel bar (2 \varnothing 6 mm) with steel fibre as well as steel fibre reinforcement.

For both types of concrete (D1.2 and D1.6), the series with steel bars and steel fibres had an improved residual load-bearing behaviour compared to the other series. The maximum flexural moment was increased by 11 % and 30.5 % for D1.2 and D1.6, respectively.

The beams with steel fibres had an improved performance up to a deflection of circa 5 mm. With further increasing of the displacement, the maximum moment could not be raised. Instead, the predominant crack was quickly opening.

3.3. Replacement of minimum reinforcement

Eurocode 2 [3] requires a minimum reinforcement for ductility and crack control.

Related to the full-scaled beam tests, an increase or stable flexural moment after initial cracking indicates that these requirements are met. This behaviour is independent of the type

Table 4. Fresh and hardened concrete properties

Concrete type	Flow table spread (mm)	Compressive strength (MPa)	Young's modulus (MPa)	Flexural strength (MPa)	Residual flexural strength L1/L2 ^a (MPa)
D1.2	530	18.3	11,233	3.2	—
D1.2.F1.0	420	21.0	15,267	3.2	5.0/3.1
D1.6	650	20.8	13,033	4.5	—
D1.6.F1.0	430	30.2	18,267	4.6	6.1/4.5

^a Performance classes L1 and L2 according to the German guideline for steel fibre reinforced concrete [16]

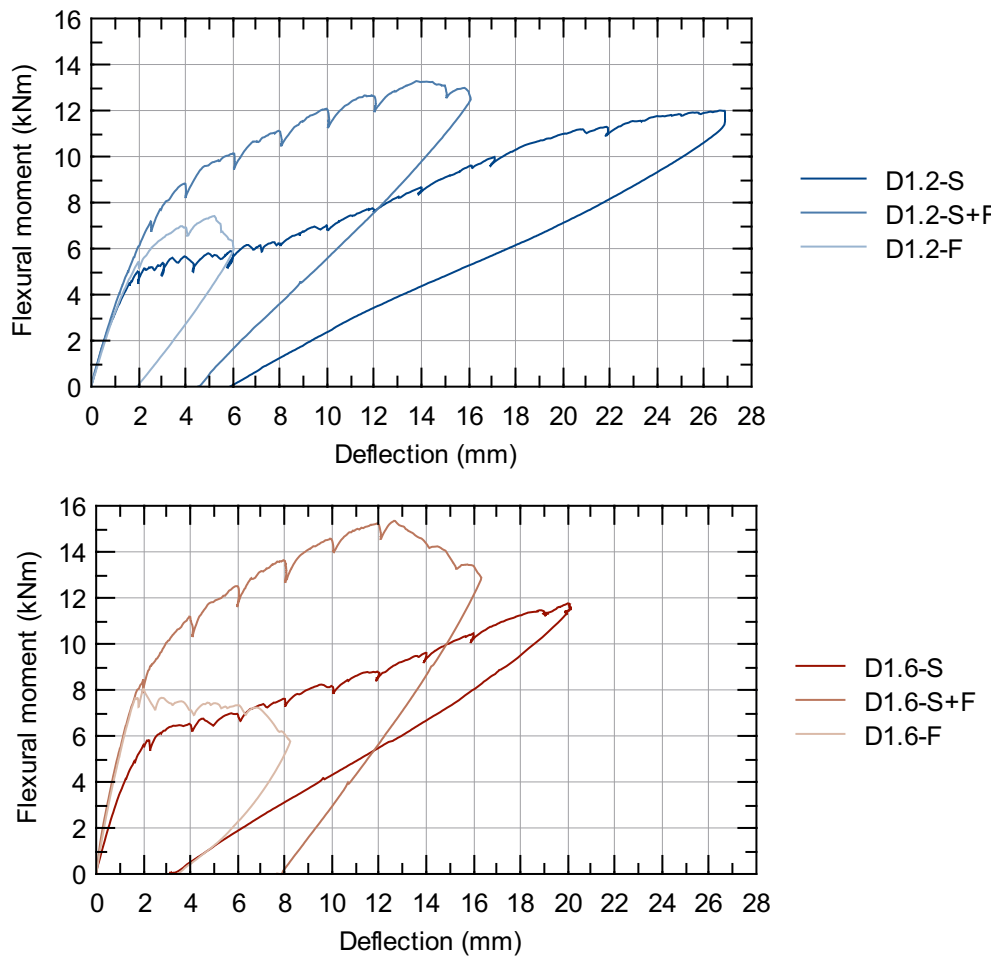


Figure 2. Moment-midspan deflection diagram for concrete series D1.2 (top) and D1.6 (bottom)

of reinforcement (steel bar or steel fibre). For the comparison of the beam tests, the cracking moment was put in relationship with two specified flexural moments. First, the flexural moment at a deflection of $l/500$ (5,8 mm) for securing member ductility and second, the moment at a crack width of 0.3 mm for crack control. If these specified moments are greater or equal to the cracking moment, the requirements for the minimum reinforcement are fulfilled.

The following approach was used for determination of the moment at a crack width of 0.3 mm. First, the measured crack widths were plotted as a function of the deflection for each load step. Second, a regression function of the data points was created for each series. Third, the deflection at a crack width of 0.3 mm of each series was determined using the regression function. And finally, the related moments were obtained using the moment-midspan deflection diagrams. Figure 3 presents the results of this approach. Additionally, the maximum crack width-deflection diagrams shows the faster crack opening with increasing deflection of the series with steel fibres (“S+F” and “F”) compared to the series “S”.

Figure 4 shows the obtained relationship for both types of concrete (D1.2 and D1.6). If the ratio is bigger than 1.0, the respective reinforcement meets the requirements of the minimum reinforcement. Nearly all series achieve a bigger ratio than 1.0. Only the series D1.6 with steel fibres (D1.6-F) have a slightly lower ratio of 0.93 and 0.96 for the ductility and crack control

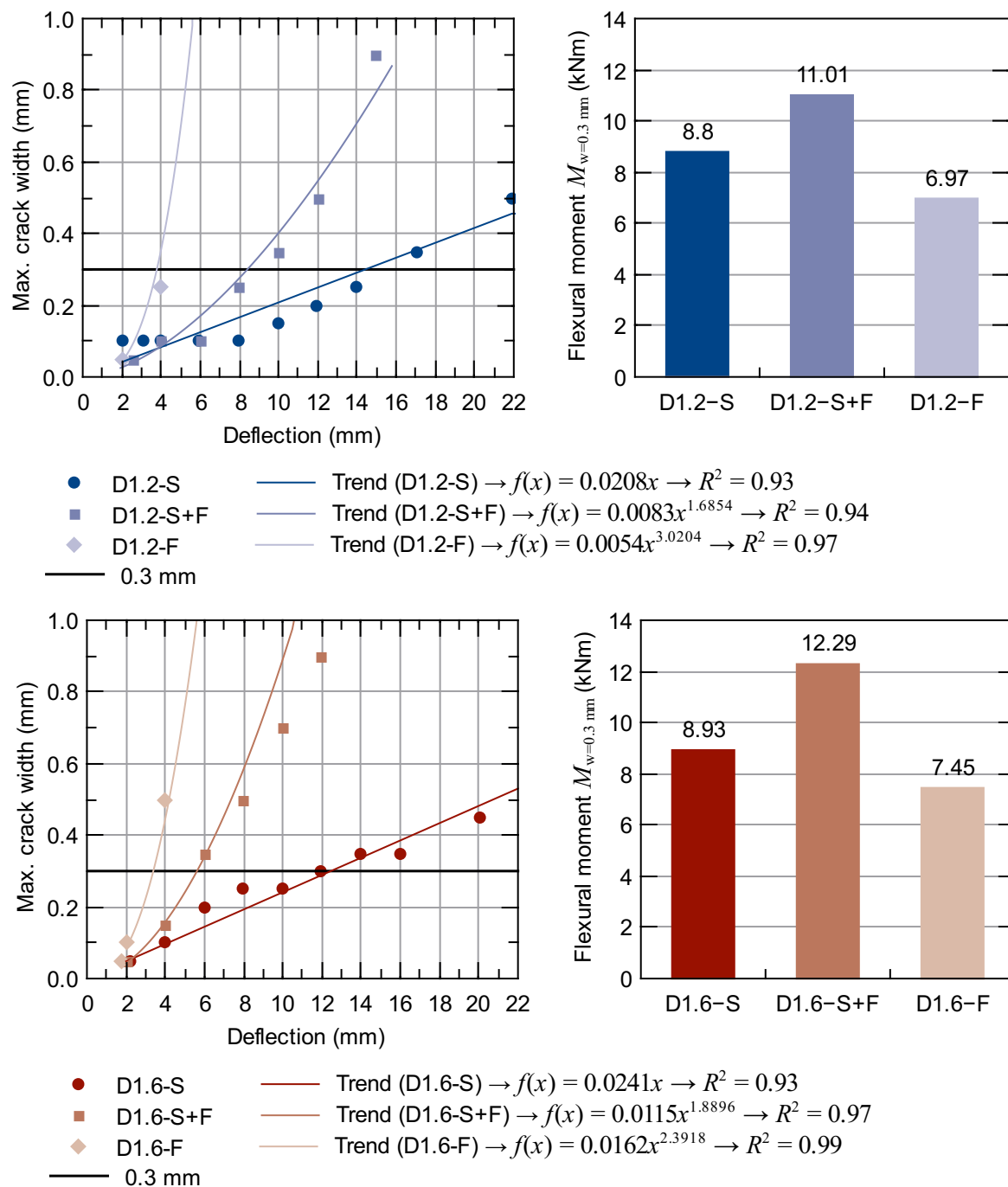


Figure 3. Determination of the flexural moment at a crack width of 0.3 mm — series D1.2 (top) and D1.6 (bottom)

criteria, respectively. The series with combined reinforcement (series “S+F” – 2 \varnothing 6 mm steel bars and fibres) show the highest ratios – except D1.6-S+F for crack control. The series D1.2-F (only steel fibres) has a sufficient load-bearing capacity for securing a ductile member behaviour and crack control and can be utilized for load-bearing walls. The series D1.6-F has a insufficient load-bearing capacity. But it could be possible to improve its performance with another steel

fibre type (e.g. a higher tensile strength or a higher aspect ratio l/d). Further tests are necessary to proof this hypothesis. For a combined reinforcement with steel bars and fibres, it is possible to reduce the steel fibre content, because of the high capacity ratios.

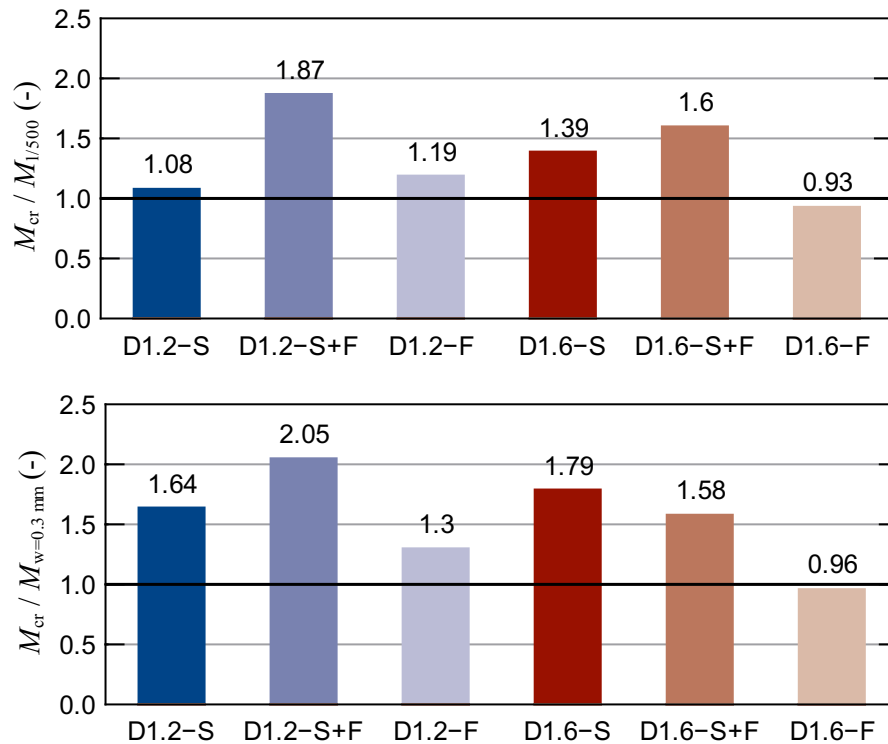


Figure 4. Comparison of the relationship between the cracking moment and the moment at $l/500$ (top) as well as the moment at a crack width of 0.3 mm (bottom)

4. Conclusion

The following conclusions can be drawn from the results of this study:

- (1) Addition of steel fibres appreciably increased the ductility of LWAC. A higher load-bearing capacity with combined reinforcement (series “S+F” – steel fibres with reduced steel bars) was conducted.
- (2) Specimens with only steel fibres (“F”) have an improved performance up to a deflection of approximately 5 mm compared to the series with only steel bars (“S”).
- (3) Specimens with steel fibres (“S+F” and “F”) have a larger crack opening with increasing deflection compared to the series “S”.
- (4) It is possible to replace the minimum reinforcement for securing ductility and crack control in load-bearing LWAC walls with steel fibres. In this study, a suitable steel fibre reinforced lightweight aggregate concrete (SFRLWAC) with a oven-dry density below $1,200 \text{ kg/m}^3$ was developed.
- (5) Another SFRLWAC with a oven-dry density below $1,600 \text{ kg/m}^3$ could be utilized for an application in load-bearing walls with an optimized steel fibre type (e.g. a higher tensile strength or aspect ratio l/d)

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