Structural behaviour of layered beams with fibre-reinforced LWAC and normal density concrete

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Abstract

The hybrid concrete structures investigated in this project were beams composed of two layers of different types of concrete. Normal density concrete (NC) was used in the top layer combined with a layer of fibre-reinforced lightweight concrete (FRLWC). Hence, the beams had a low weight and the NC layer fulfilled the requirements for ductility in compression. The material properties of the FRLWC were investigated through small-scale tests, the uniaxial tension test, the 3-point bending test and a compressive test on concrete cylinders/cubes. The small-scale tests constituted the basis for obtaining design parameters used in the design of the larger hybrid beams. These beams had 0.5% and 1.0% of steel fibres, and were subjected to a 4-point bending test in order to study the performance in terms of both shear and bending actions. Fibre counting was carried out in order to relate the performance to the number of fibres crossing the critical section, which turned out to have a considerable influence on the performance of the FRLWC. In general, the types of failure were as expected. This study shows that the concept of combining NC and FRLWC in one cross-section shows promising results, and no problem with the bond between the layers of concrete was registered. Steel fibre reinforcement of the lightweight concrete increased the ductility in tension, so the amount of conventional shear reinforcement could be reduced. The concept provides a low self-weight of the structure, practical solutions in the construction phase and good premises for more efficient building.

Keywords

testing, lightweight concrete, fibres, hybrid beams, structural response

1 Introduction

In combination with ordinary reinforcement bars, concrete is one of the most common building materials in the world, and there is a wide range of areas of application with reinforced concrete, e.g. buildings, bridges, dams and retaining walls. In order to expand the possibilities in using concrete as a construction material, the development of new types of concrete and innovative structural solutions is required. Lightweight aggregate concrete (LWC) has been used as a construction material for many decades, with the main objective for using LWC normally being to reduce the dead weight of structures (ACI Committee 213 2003; Clarke 2002; EuroLightCon 1998; fib bulletin 8 2000) . Thus, with a low weight, the dimensions of the foundations in buildings can be reduced in areas with low bearing capacities, while the inertia actions are reduced in seismic regions, thereby enabling an easier handling and transportation of precast elements. Even with the major advantage of a reduced weight and the high strength-to-weight ratio of the material compared to conventional concrete, the use of LWC is still limited as a mainstream construction material in the building industry. The major disadvantage of LWC is the brittleness in compression at the material level compared to normal density concrete. However, for large and advanced structures such as high-rise buildings, bridges and offshore structures, it has been applied with great success (Haug and Fjeld 1996; Ingebrigtsen 1999; Melby 2000). Other advantages of LWAC compared to normal weight concrete are its improved durability properties, fire resistance and low thermal conductivity (Jensen et al. 1995; Lo-shu et al. 1980; Neville 2012; Zhang and Gjørv 1991)

There is also a demand for the optimization of structural elements, which leads to new combinations of materials and possibilities in application. An optimal combination of materials is situational, and one may optimize with regard to, e.g. sustainability, load capacity, weight and/or environmentally friendly aspects depending on the area of application. A variety of commercial products that combines different materials into concrete composites are available. The most common combination of materials is steel-concrete composites (Johnson 2004; Oehlers and Bradford 1999; Yam and Yam 1981). A much used and well-documented application is steel-concrete composite columns (Elnashai et al. 1990; Ranzi et al. 2013; Shams and Saadeghvaziri 1997; Shanmugam and Lakshmi 2001). These columns have advantages with respect to the fire requirements of the steel member: those of permanent formwork leads to an efficient construction phase, as well as a reduction or elimination of problems with a local buckling of the steel. Floor systems and bridge decks are another typical use of steel-concrete composites in which the sheeting of steel can be employed as both permanent formwork and external reinforcement (Batchelor et al. 1978; Mullett 1998; Schuster 1976).

Another type of composite structure is layered elements that combine different types of concrete in combination with ordinary reinforcement, fibre reinforcement or with layers of fibre-reinforced polymer composite (Iskhakov and Ribakov 2013; Keller et al. 2007; Lapko et al. 2005; Zhang et al. 2006). Such structures are often called hybrid structures, and the main challenge with this type of structure is the interaction and bond between the materials (Aydin and Saribiyik 2013; Canning et al. 1999; Santos and Júlio 2012). A much used application of hybrid structures is the combination of concrete construction methods, e.g. in-situ concrete and precast concrete, which provides simple, buildable and competitive structures (Elliott 2002; fib 2002; Glass 2005).

2 Hybrid beam, test and design methods

The motivation for investigating the performance of hybrid concrete beams in this study was to design a structural element that utilized the most beneficial properties of different types of concrete and combine them in one cross-section (Nes 2013). Hence, a two-layered cross-section was chosen with a 200mm bottom layer of fibre-reinforced lightweight aggregate concrete (FRLWC) and a 50mm top layer of normal density concrete (NC), as seen in Figure 1. By using a FRLWC with density of 1200 kg/m^3, the self-weight reduction is 42% compared to a cross section with only NC. The hybrid beams may represent one-way slab elements of which the bottom layer constitutes a precast formwork, thereby resulting in a cost-effective product with respect to manufacturing, transport/assembling of the element and load capacity. The use of lightweight concrete minimizes the self-weight, while the structural performance is taken care of by conventional longitudinal tensile reinforcement and a top layer of normal density concrete. Steel fibre was used in order to improve the performance and mechanical properties of the lightweight concrete.

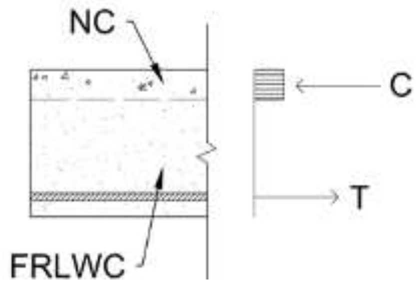

Fig. 1: Cross-section and design principle for hybrid concrete beams

Due to different mechanical properties of NC and LWAC, the shear capacity of the composed beam without fibre is reduced by 35% compared to a monolithic beam of NC (Standards Norway 2008a). The shear capacity will increase with approximately 50% and 80% for the composed beams with the introduction of 0.5% and 1.0% fibre respectively (Kanstad 2011). However, this is strongly dependent of the fibre orientation factor used in the calculation. The bending capacity is almost the same for a monolithic beam of NC compared to the composed beams in this study since the compression zone lies within the top layer of NC. By using 0.5% and 1.0% fibre in the composed beams the bending capacity will increase by approximately 10% and 15% respectively.

2.1 Uniaxial and flexural tensile strength of fibre-reinforced concrete

In order to design and validate the results from the hybrid beams, small-scale testing was required to obtain the mechanical properties. The material properties of the FRLWC were investigated through small-scale tests: the uniaxial tension test (UATT), the 3-point bending test (3PBT) and a compressive test on concrete cylinders/cubes.

The uniaxial tension test is probably the most direct method to determine the tensile properties of concrete. However, the test is hard to carry out and size, and the shape, test system, measuring techniques, curing conditions of concrete, etc. will all affect the results (Van Mier and Van Vliet 2002). The UATT was carried out as a combination of the procedures described in (SINTEF 2007; Vandewalle 2001). The specimen size was 100x100x600[mm] with a notch located at the middle of the specimen, which had a width of 4mm and a depth of 10mm on each side of the specimen. The displacement was measured using transducers located on two opposite sides of the specimen, covering a distance of 100mm. The stress-crack opening relationship was found together with the Young's modulus, which was calculated at a load level of 40% of the cracking load. A problem with this test setup is the introduction of eccentricities; therefore the specimen was clamped at both ends and was free to rotate, thus reducing this effect.

The governing design parameter for fibre-reinforced concrete is the flexural tensile strength. In this study, this was determined according to NS-EN 14651, which describes a method for casting beams with a notch subjected to 3-point bending tests (Standards Norway 2008b). The quadratic cross-section, *bxh*, of the beams is 150mm and the free span, *l*, is 500mm. To avoid multiple cracking, a centred notch is employed with a width of 5mm placed 125mm from the top of the beam. Based on the measured deflections, δ, the crack mouth opening displacement (CMOD) can be calculated with the relation δ=0.85CMOD+0.04. The corresponding residual flexural tensile strengths, $f_{R,i}$, at different load levels, F_i, can then be found as:

$$f_{R,i} = \frac{3F_i l}{2bh^2} \qquad (1)$$

In the design of fibre-reinforced concrete, a residual tensile strength corresponding to CMOD=2.5mm is used, $f_{R,3}$. However, this strength is based on a linear stress distribution. In the design for bending the tensile stress, $f_{t,res2.5}$ is assumed to be constant over 90% of the cross-section height, and is given as (Kanstad et al. 2011):

$$f_{t,res2.5} = 0.37 \cdot f_{R,3} \qquad (2)$$

2.3 Bond

To obtain monolithic behaviour in the two-layered beams, the transfer of shear stresses between the structural layers of concrete is important. This can be done by using steel reinforcement crossing the interface or simply by creating a sufficient bond between the layers. Many different tests are available for testing the bond between layers of concrete, and the bond strength is dependent on the test method used. The Bi Surface Shear Test (BSST) provides higher bond strengths than pull-off and splitting tests (Momayez et al. 2005; Silfwerbrand 2003).

The BSST was developed for evaluating the bond strength between new and existing concrete subjected to shear stresses (Momayez et al. 2004). The test setup and dimensions and typical result are shown in Figure 2. The loading is symmetrical, and steel blocks are used for distributing the load.

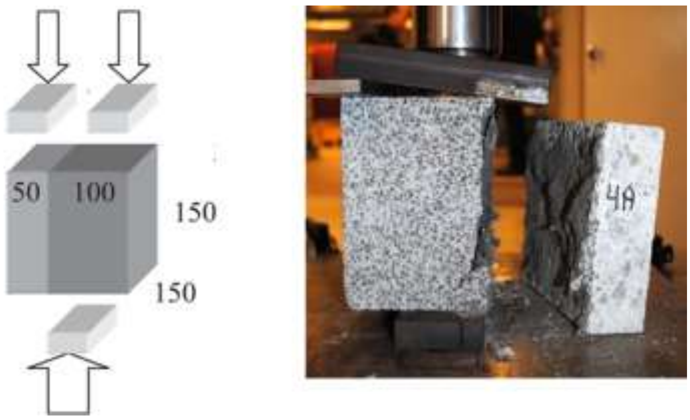

Fig. 2: Bi-surface shear test dimensions (mm) and loading (Momayez et al. 2005), and typical result

The main advantages of the BSST are the simplicity of the test, and that neither compressive nor tensile stresses are introduced during testing. It represents a state of simple shear that is typical for layered cross-sections. Thus, the BSST was used in this study to verify the bond strength between the FRLWAC and NC concrete.

2.4 Fibre distribution and orientation

One important aspect when evaluating the mechanical properties of fibre-reinforced structures is to take into account the fibre orientation and fibre distribution. The viscosity of the fresh concrete influences the fibre orientation and distribution and thereby the strength of a structure. In order to measure the fibre orientation, an orientation factor α can be determined (Soroushian and Lee 1990):

$$\alpha = \frac{n_f A_f}{v_f} \qquad (3)$$

where n_f is the number of fibres per surface unit, A_f is the cross-section area of one fibre and v_f is the measured fibre volume fraction. α = 0 or α = 1 means that fibres are uni-directed, and α = 0.5 corresponds to an isotropic fibre orientation. Due to the flow of fresh concrete, a higher orientation factor will be obtained in the flow direction, particularly in self-compacting concrete. Increased flow and a longer flow distance have been reported to lead to a better alignment of fibres (Stähli et al. 2008). However, other studies have shown no major influence of the flow length on the fibre orientation (Vidal Sarmiento et al. 2012). Near the walls of moulds the fibres are not free to rotate, and frictional restraints along the walls of the mould align the fibres parallel to the walls.

3 Experimental programme

3.1 Set-up

The test programme was designed to study the structural behaviour of hybrid concrete beams. The beams were composed of a 50mm top layer of normal density concrete and a 200mm bottom layer of fibre-reinforced lightweight concrete. Hence, the beams had a low weight and the NC layer fulfilled the requirements for ductility in compression. The fibres were added in order to improve the LWC performance.

The experimental programme consisted of 16 simply supported concrete beams, all with the same cross-section. In Programme 1, eight beams were designed for shear failure, and Programme 2 involved eight beams designed for bending failure. The beams were tested in flexure under a four-point loading system. As a result, the central part of the beam was in pure bending mode. Figure 3 shows the set-up of the test programme for Programme 2. The free span between the supports was 2.3m and 2.4m, and two concentrated loads were symmetrically applied at a distance of 0.853m and 1.546m, respectively, for beams in Programmes 1 and 2. To help investigate the effect of increasing fibre content, a different amount of fibre was employed. In Programme1, three beams were identical and had 0.5% or 1% fibre by volume of concrete, and two beams were without fibre and considered as reference beams. Four beams each had 0.5% and 1% fibre in Programme 2, and half of these of the beams had shear reinforcement with spacing 200mm to avoid a possible shear failure, thereby resulting in two identical beams.

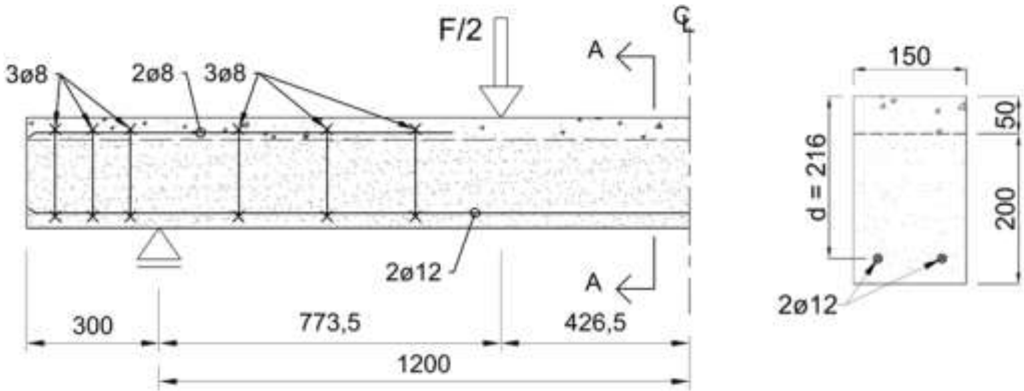

Fig. 3: Hybrid beams from Programme 2, reinforcement layout and dimensions [mm]

The beams were designed in bending to be under-reinforced to help satisfy the ductility requirements in this type of structure. In Programme 1, two deformed bars with a diameter of 16mm were provided, with two bars with a diameter of 12mm were provided in Programme 2. To ensure enough anchoring capacity a transverse horizontal bar, with a diameter of 20mm was welded onto the bottom layer of the tensile reinforcement at the ends of the beams. The beams with shear reinforcement in Program 2 also had three stirrups outside the support. The concrete cover to the stirrups was 15mm.

3.2 Materials, mix proportions and casting

To produce the lightweight concrete, foam and lightweight expanded clay aggregate, commercially known as LECA, was used to achieve the desired density of the LWAC. Table 1 gives an overview of the mix design for the FRLWC and the NC. The LECA had a particle density of 670 kg/m^3. To improve the paste/cement and fibre/concrete bonds, the mix contained 9% of microsilica by weight of the cement. Several batches were needed during the production of the beams. The wet density was measured during mixing, and foam was added until the desired density was reached for every batch. The wet density varied from 1148kg/m^3 to 1304kg/m^3. In order to achieve similar slump flow of concrete when increasing the fibre content, extra superplasticizer was added to the mix until a satisfactorily slump flow was obtained. Synthetic fibres were used in the mix to reduce plastic shrinkage. For beams with steel fibres, Dramix 65/35BN was used, which is a cold drawn wire fibre of bright steel with hooked end, a length of 35 mm and an aspect ratio of 64. The tensile strength of the fibres was 1000 MPa and the elastic modulus was 210 GPa.

Table 1: Mix design for concrete

Material	FRLWC [kg/m^3]	Material	NC [kg/m^3]
Norcem Industri	105.3	Norcem Standard FA	302
Norcem Anlegg	241.1	Sand Årdal 0-8mm	1130
Elkem Microsilica	32.5	Rock Årdal 8-16mm	760
Water	165.0	Water	198
Leca 2-4mm	220	Glenium SKY 550	3.4
Sand Lillestrøm 0-4mm	400.0		
BASF MECH PP 150	1.5		
BASF Glenium SKY 550	6.5		
BASF Glenium Steam	1.8		
Defoaming agent	0.11		
Dramix 60/35 Steel fibre	0,0.5 or 1.0%		
Foam	Varied		

The casting of the hybrid beams was done in two steps. First, the FRLWC layer was cast by pouring it from wheelbarrows into one end of the formwork for the beams and additional loads of concrete into the middle of the formwork due to an insufficient flow of concrete. The flat ladles used for the manual distribution of concrete were aligned parallel to the casting direction in order to obtain a fibre distribution and orientation governed by the flow of FRLWC. The beams were then stored in the moulds under wet burlap sacks and plastic foil for one week before casting the top layer. The surface was moist but there was no free water when the top layer of normal concrete was cast in the second step without any preparation of the FRLWC surface. A few days later the demoulding took place, and again the beams were covered with wet burlap sacks and plastic foil until testing.

3.3 Mechanical properties of NC and FRLWC

The normal density concrete obtained a mean compressive strength of 17.4 MPa and a the modulus of elasticity of 26 GPa by following the procedure in (Standards Norway 1987). To help find the mechanical properties of the fibre-reinforced lightweight concrete, uniaxial tensile tests and 3-point bending tests were carried out, with the results given in Table 2. These were used in the design and evaluation of the test results of the two-layered beams. The mean compressive strengths, f_{lcm}, found from cylinders, and the mean densities, ρ_m, obtained from different batches are also given in Table 2. Lightweight concrete without fibre was also tested and had a mean compressive strength of 20.2 MPa and a mean density of 1265 kg/m^3. The mean Young's modulus was 12 GPa based on the results from the uniaxial tensile tests. The tensile strengths given in Table 2, f_{ct}, relate to the strength at cracking for UATT, as well as to the limit of proportionality calculated at the highest load value in the CMOD interval $\leq$0.05mm for 3PBT. For the UATT, there was a strong relationship between strengths and the counted number of fibres, n_f. The residual tensile strengths, f_{R3}, obtained from the 3-point bending tests had mean values of 3.4 MPa and 3.8 MPa, and standard deviations of 1.02 MPa and 1.26 MPa for the fibre-reinforced concrete with 0.5% and 1.0% fibre, respectively. The large deviations are confirmed in Figure 4, and are quite typical when testing fibre-reinforced concrete.

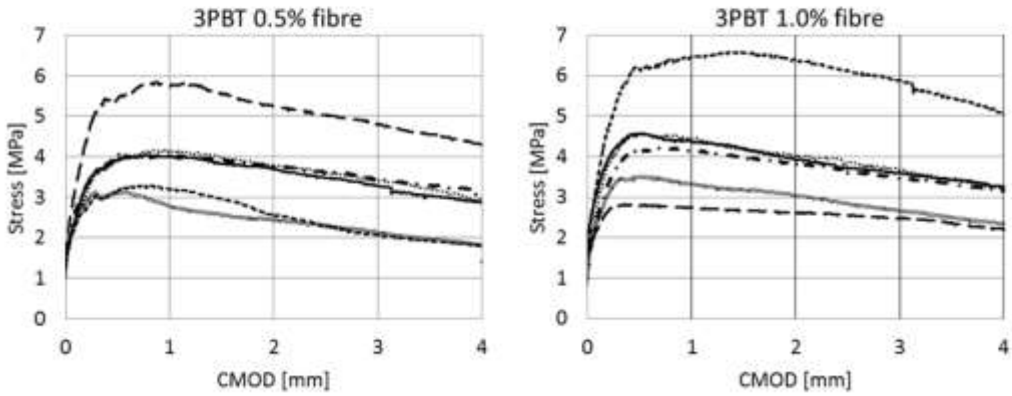

Fig. 4: Load-CMOD diagrams from 3-point bending

Table 2: Mechanical properties and fibre content of FRLWC

ID	Fibre [%]	Batch Nr	f_{lcm} [MPa]	ρ_m [kg/m^3]	f_{ct} [MPa]	f_{R3} [MPa]	n_f [pr. cm^2]	α	$f_{R3,norm}$ [MPa]
1-UATT	0.5	6	18.6	1265	1.50	-	1.8	0.855	-
2-UATT	0.5	6	18.6	1265	1.32	-	1.3	0.637	-
3-UATT	0.5	2	16.7	1170	1.29	-	1.7	0.813	-
4-UATT	0.5	3	21.9	1244	1.37	-	1.5	0.732	-
5-UATT	0.5	3	21.9	1244	1.16	-	1.2	0.561	-
6-UATT	0.5	5	22.6	1250	1.76	-	1.8	0.865	-
7-3PBT	0.5	2	16.7	1170	2.1	2.3	1.3	0.627	1.5
8-3PBT	0.5	2	16.7	1170	2.1	2.3	1.3	0.600	1.6
9-3PBT	0.5	6	19.4	1304	2.6	5.0	2.1	1.007	1.7
10-3PBT	0.5	6	19.4	1304	2.0	3.6	1.6	0.764	1.8
11-3PBT	0.5	6	19.4	1304	2.2	3.6	1.8	0.838	1.5
12-3PBT	0.5	6	19.4	1304	2.0	3.5	1.4	0.685	2.0
13-3PBT	1.0	8	17.6	1156	2.1	2.8	2.5	0.585	2.1
14-3PBT	1.0	8	17.6	1156	2.6	6.1	2.9	0.677	3.6

15-3PBT	1.0	8	17.6	1156	2.0	2.6	2.1	0.495	2.6
16-3PBT	1.0	10	16.6	1148	2.2	3.8	2.5	0.593	2.8
17-3PBT	1.0	10	16.6	1148	2.0	3.6	2.8	0.669	2.2
18-3PBT	1.0	10	16.6	1148	2.7	3.7	2.5	0.602	2.6

The prescribed casting procedure of the specimens for the 3PBT, was to pour concrete into the middle of the mould and then at the ends. Thus, at the notch there was no flow distance of concrete, and only the effect of boundary conditions (walls) and the settlement of self-consolidating concrete would align the fibres longitudinally. From Table 2 it can be seen that for specimens containing 0.5% fibres, the number of fibres per cm^2 was similar between UATT and 3PBT. The fibres were counted in sections 50mm from the notch. When the fibre content was doubled, the number of fibres obviously increased. However, the orientation factor, α, decreased when the fibre content increased. This indicates that the specimens with the lowest fibre content had more fibres aligned in the direction of the flow. Since the flexural capacity did not increase correspondingly with 1% fibre, there may exist an upper critical value for the fibre content when utilizing fibres in the longitudinal direction. This limit will be affected by the concrete mix and the ability of the concrete to flow.

3.4 Evaluation of bond

In order to study the bond between two concrete layers, a bi-surface shear test was used. The test was intended to provide information about the shear stress capacity of the interface between concrete layers in the two-layered beams. In the beams, the interface was subjected to a combination of tensile and compressive stresses, while the test represented a simple shear situation.

The casting of the first layer of FLRWC was carried out one week before the top layer of NC was cast. Two panels with dimensions of 100/600/600mm were cast of FRLWC with 0.5% and 1.0% steel fibre content, respectively. The panels were stored in the moulds and covered with soaking wet burlap sacks and plastic foil until the top layer of 50mm was cast, i.e. the same curing conditions as the hybrid beams. The surface of FRLWC was moist with no free water and was not prepared in any way before casting of the NC layer. As an indication of the roughness of the surface, Figure 5 shows the interface after casting the FRLWC. After one month of curing for the FRLWC, cubes from each panel were sawed, excluding a few cm concrete at the edges. The testing was carried out in two separate series: Series 1 - testing 36 days after casting of the FRLWC, and Series 2 - testing after one year where the specimens were stored without any cover at a temperature of approximately 20 degrees centigrade.

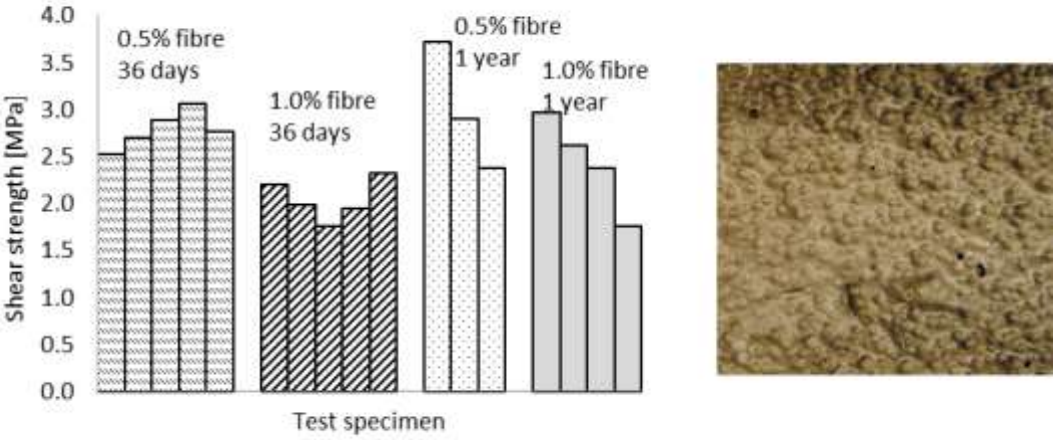

Fig. 5: Shear strengths from bi-surface shear tests and surface of FRLWC before casting of NC layer

For all specimens except one, the failure was initiated at the interface between the two different types of concrete. The typical failure mode from the test can be seen in Figure 2, while the obtained shear strength ranged from 1.76-3.72 MPa, see Figure 5. The results do not indicate any weakening of the interface between FRLWC and NC after one year under the specified conditions. The mean shear strength was lower with 1% than with 0.5% of fibre. This can be explained by a reduced workability of the mix, resulting in a bad compaction of the concrete that may cause a balling or matting of the fibres when increasing the fibre content. Some mechanical properties of fibre-reinforced concrete, like the compressive strength, reach a limit and can even be reduced when adding more fibres (Ezeldin and Balaguru 1992; Nataraja et al. 1999).

3.5 Instrumentation and test procedure

Displacement gauges were used in order to measure deflection in addition to compressive and tensile strains. All gauges were placed at the centreline of the beam, which was also centred at the width. Deflection was measured at the mid-span, while the gauges measuring compressive and tensile strains in the longitudinal direction of the beams were placed on top and under the beam, covering a length of 200mm.

The load was applied by a 250kN servo-controlled hydraulic actuator, and distributed to the LWAC beam by a steel beam (equalizer beam) with two rolled supports, see Figure 6. At an initial stage, the beams were preloaded with a very small load to remove any slack in the system. The load was then released, all instruments were zeroed and the beams were loaded at a rate of 1.0 mm/min. The loading was applied in intervals of 10 kN. At each load level there was a 5 min. break to study the formation of cracks, and after reaching the load for the spalling of the concrete cover, the beams were continuously loaded. All displacement, strain and load readings were automatically logged with a rate of 1 Hz.

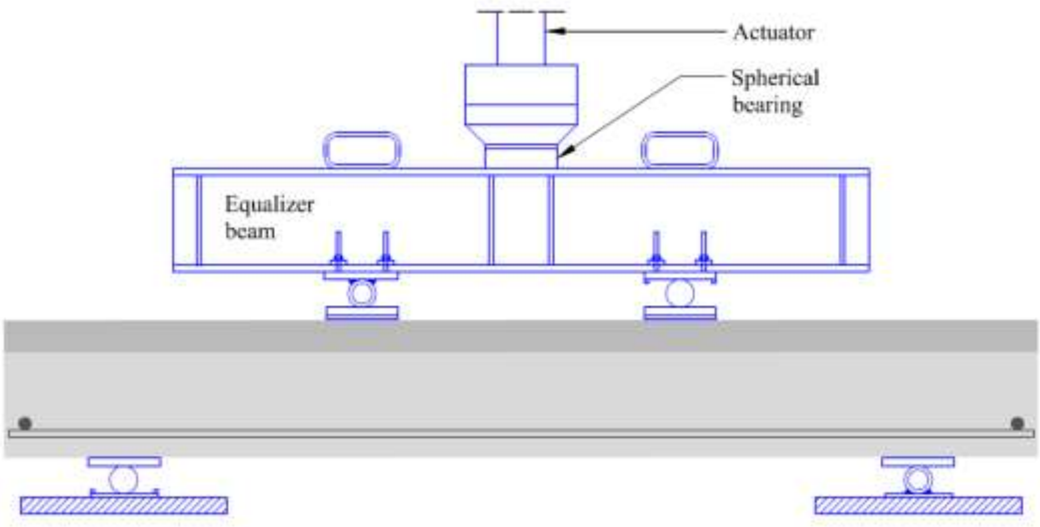

Fig. 6: Schematic illustration of the test set up

4 Results

4.1 Failure mode and ultimate strength

Beams 1-8 were designed for shear failure, which was also the governing failure mode. From Table 3 and Figure 7, it can be seen that the scatter in the results was large, but in general the results were as expected; the load level at both diagonal crack initiation, P_{diag} , and failure, P_{fail}, increased with an increasing fibre content. These results correlated well with the results from the uniaxial tension test with respect to an increase in both the direct tensile strength and post-cracking performance depending on the number of fibres crossing the crack. The behaviour of the beams was typical shear-tension failures. After the initial formation of flexural cracks, some of them propagated diagonally before the ultimate load. By introducing fibres, a much more ductile response after initiation of the diagonal cracks was achieved compared to the reference beams without fibre, which were completely brittle. The scatter in results for beams without shear reinforcement is well-known. From the data in Table 3, the results indicate that for the beams with fibre the relative variation between tests decreases with more fibres.

Table 3: Experimental results

Beam	Fibre [%]	Batch Nr	f_{lcm} [MPa]	ρ_m [kg/m^3]	Failure type	P_{fail} [kN]	P_{crack} [kN]	P_{diag} [kN]	ε_c [10^{-3}]	ε_s [10^{-3}]
1	0	1	20.2	1265	shear	43.4	10	27	-0.50	0.68
2	0	1	20.2	1265	shear	31.2	15	25	-0.53	0.70
3	0.5	6	18.6	1260	shear	70.6	28	49	-1.09	1.19
4	0.5	3	21.9	1244	shear	51.0	25	44	-0.77	1.38
5	0.5	3	21.9	1244	shear	64.1	22	46	-1.38	1.82
6	1.0	10	16.6	1148	shear	94.6	39	74	-1.66	2.26
7	1.0	9	21.2	1317	shear	87.4	33	70	-1.53	1.94
8	1.0	9	21.2	1317	shear	82.2	30	67	-1.21	1.75
9	0.5	4	20.1	1198	moment	70.5	13	-	-2.30	4.34
10	0.5	4	20.1	1198	moment	71.4	14	-	-2.17	5.01
11	1.0	7	19.4	1404	moment	73.5	15	-	-2.17	4.65
12	1.0	7	19.4	1404	moment	72.2	20	-	-2.30	3.54

13	0.5	5	22.6	1250	shear	41.2	21	41	-1.02	1.92
14	0.5	5	22.6	1250	shear	49.1	10	43	-1.09	2.17
15	1.0	8	17.6	1156	shear	70.4	24	55	-2.11	3.54
16	1.0	8	17.6	1156	shear	65.1	21	49	-2.04	3.29

Beams 9-16 were designed for bending failure, although beams13-16 experienced shear failure. The beams had the same cross-section properties, but beams 9-12 also had additional conventional shear reinforcement. Thus, a goal was to study the performance of fibres as a replacement for shear reinforcement. Figure 7 shows no large differences in the performance of the beams having bending failures, though the beams containing a 1.0% fibre reinforcement had a somewhat stiffer behaviour. The structural response for the beams with a bending failure was typical for beams designed to be under-reinforced. This was confirmed by the reinforcement strains, which were almost twice the yielding strain of steel at peak load. Close to peak loads, the flexural cracks tended to deviate from a vertical path, to an alignment more parallel with the interface between the different concretes. The difference in material properties between NC and FRLWC can help explain this behaviour. The elastic modulus of the FRLWC was only one-third of the modulus of elasticity for the normal weight concrete. Consequently, a strain in the FRLWC at an equal stress level was triple the strain value for NC. Assuming that the bond between the layers of different concrete qualities was good, the NC layer was restraining displacement of the adjoining FLRWC. When the flexural cracks were approaching this interfacial zone, they were prevented from further expansion following the original path. At the same time, the low tensile strength of the FRLWC was exceeded, thus creating energy that must be released through cracking. Hence, the cracks were forced to expand in other directions through the weakest material. The bending failures were initiated by horizontal cracks in the compressive zone, and at failure, compressive strains were approximately $-2.2\cdot10^{-3}$. This was considerably smaller than the allowed ultimate strain in the design of cross-sections for the bending of $-3.5\cdot10^{-3}$ in Eurocode 2. However, the measured strains were the average values over a length of 200mm. In addition, the layer of normal density could be considered as being subjected to approximately concentric loading, with a limited value of the compressive strain to $-2.0\cdot10^{-3}$ in Eurocode 2.

Based on the results of the beams, the fibres were not able to replace the employed conventional shear reinforcement in Program 2. The beams with 1.0% fibre reinforcement were able to carry almost as much load as the beams with shear reinforcement. However, they were not able to prevent the diagonal cracks leading to failure. For this reason, the brittle failure mode was not acceptable with regard to the ductility and yielding of the reinforcement, which is confirmed by the compressive strains in Table 3. At failure, the strains in the beams with 1% fibre were almost at the level of the beams experiencing bending failure, and therefore were at the limit between shear and bending failure.

Regarding the interface between the overlay and the FLRWC, no problems were visually observed, thereby implying that the method of casting and the curing conditions of the beams were satisfactory. However, the relative displacements between the layers were not measured and the additional shear reinforcement in Beams 9-12 ensured a combination of dowel action and shear resistance in these beams. By keeping the bottom layer of FRLWC moist during the week of curing before the next layer was cast, the FRLWC layer would not consume water from the fresh NC layer and a good bond was ensured. In addition, shrinkage was reduced. The total shrinkage was measured over a period of 18 months for the FRLWC with 1% of fibre. The shrinkage was 0.24‰, 0.57‰ and 0.98‰ after 7, 28 and 540 days. The large shrinkage can partly be explained by the use of foam in the concrete mix which replaced aggregate. The creep was also large and the shrinkage was then partly compensated in the beams by relaxation. After the top layer was cast, the normal concrete would be subjected to shrinkage, and problems may occur when this mechanism makes the concrete contract. Nonetheless, the FLRWC had a very low Young's modulus, and was most likely able to follow the movements of the NC layer without weakening the interface.

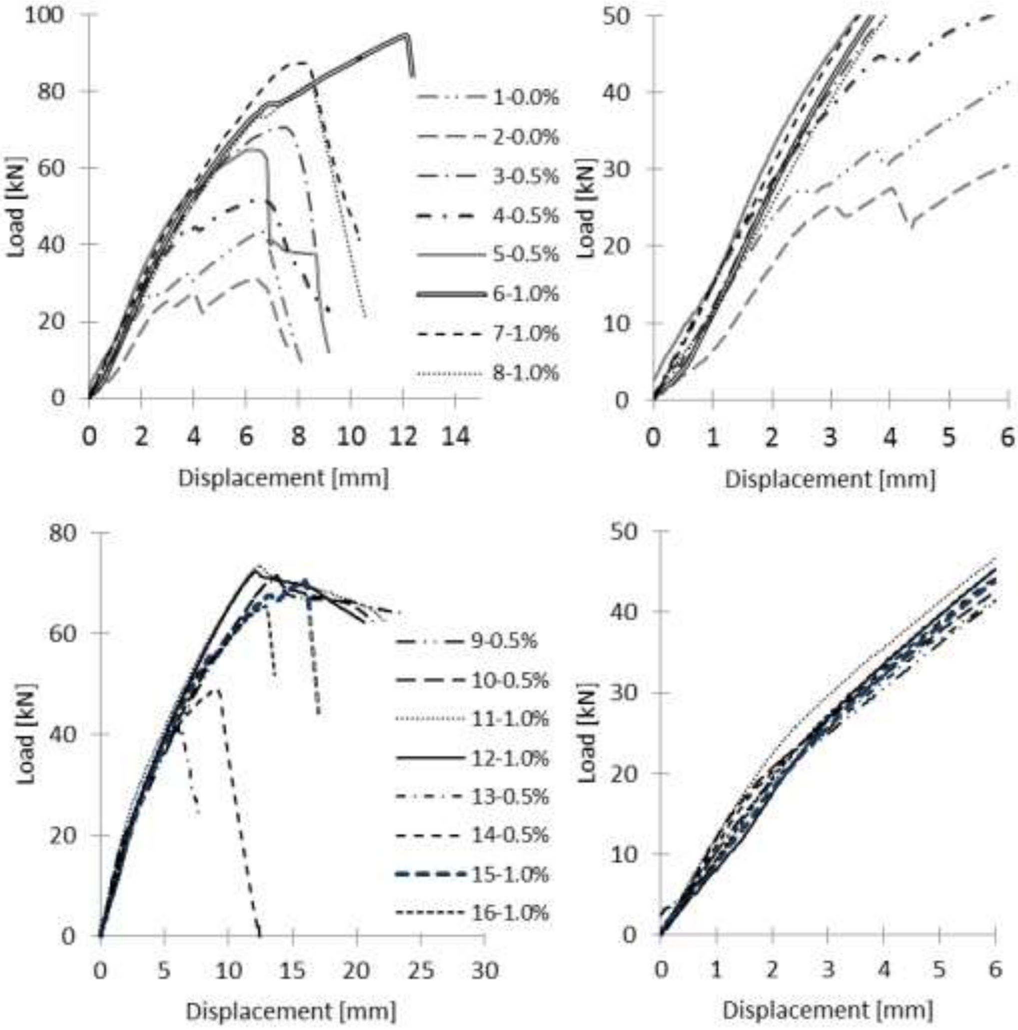

Fig. 7: Load displacement responses

4.2 Cracking behaviour

One well-known advantage of fibre reinforcement is the positive effect on crack width and crack distribution. Due to fibre bridging, each crack is able to transfer larger stresses, leading to smaller crack widths and a shorter distance between the cracks. The crack patterns for beams failing in bending with both 0.5% fibre reinforcement (top beam) and 1.0% fibre reinforcement (lowest) are shown in Figure 8 for beams failing in bending. The number of cracks was considerably higher for beams with 0.5% fibre compared to beams with 1% fibre, which was not expected. Even so, the increasing tensile strength with an increasing number of fibres, the strain hardening effect and the fibre orientation could help explain this. When the tensile strength increased, crack initiation occurred at a higher load level, see Table 3. After crack initiation, the crack development was more restricted due to the strain-hardening effect, which was more likely to occur with 1.0% fibre content than with 0.5%. As a result, the material was also able to withstand higher stresses at larger crack widths when the fibre content was high. Another important aspect was the restricted unloading of the surrounding concrete after cracking, as the fibres caused relatively stable cracking (no sudden drop in tensile capacity). Due to the casting method most of the fibres were aligned in the longitudinal direction. Nevertheless, with increasing fibre content, more fibres seemed to be aligned in other directions, leading to a more general strengthening of the material. Measuring of stains during loading showed that when a stabilized crack pattern was obtained at approximately 50 kN, the compressive strains were similar for beams with a different fibre content, whereas the tensile stains were higher for beams with the lowest content. Figure 8 shows that the cracks in the beam containing 0.5% fibres tended to split when developing and forming new deeper cracks. For the beam with 1% fibre, the fibres seemed to limit the crack development, thus leading to a somewhat stiffer behaviour and smaller displacements.

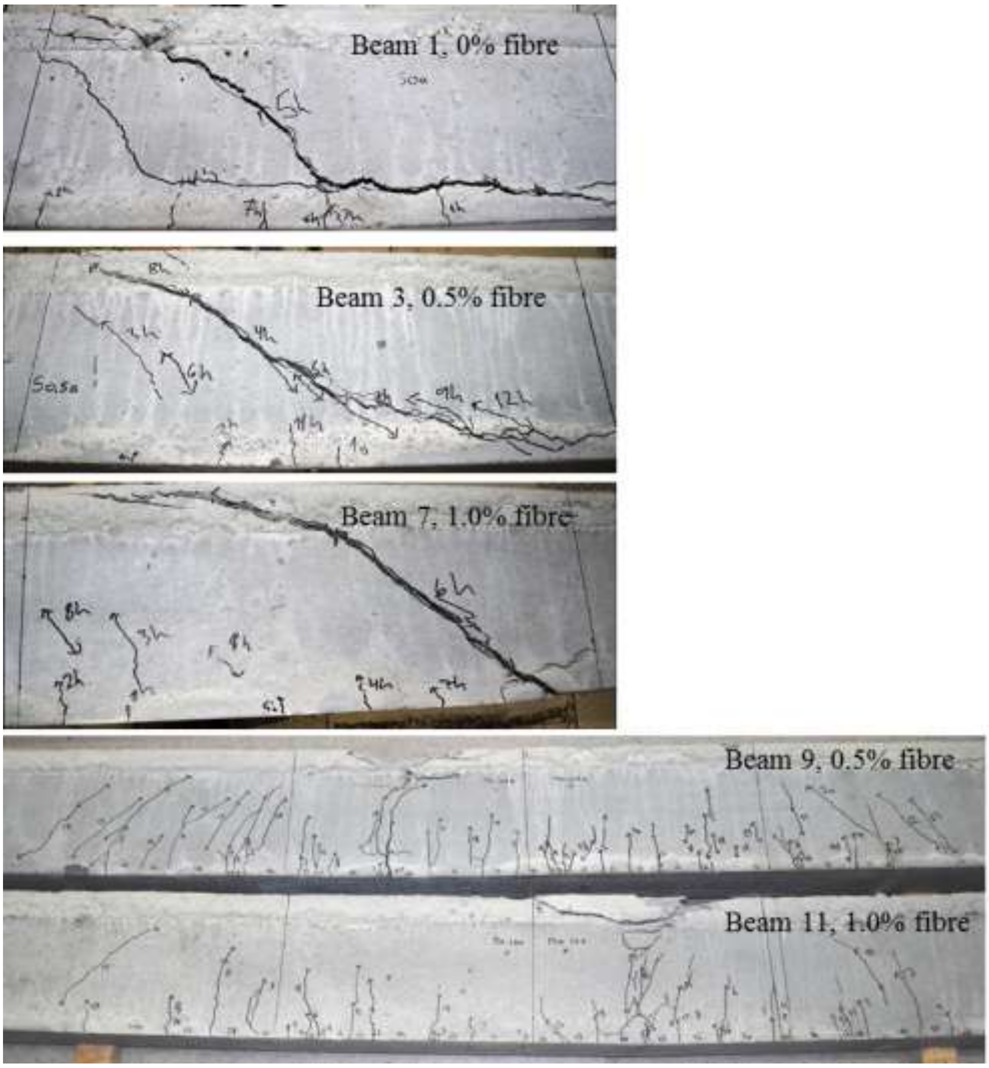

Fig. 8: Crack pattern at failure for beams 1,3 and 7 failing in shear, and beams 9 and 11 failing in bending

During the loading of the beams, an optical strain measuring device was used for the beams failing in bending, making it possible to visualize the strain development on certain areas of the beams. Figure 9 shows the contour diagrams of the major strain distribution of beams at maximum load level and at failure load for the beam with 1% of fibre failing in bending. The bright areas indicate the largest values, i.e. flexural cracks. The picture shows the splitting of the compressive zone which developed due to a high stress concentration caused by the flexural cracks. It seems like this splitting crack developed into the interface between the different types of concrete after failure was initiated, although none of the contour diagrams indicate strain concentrations in the interface before the point of failure.

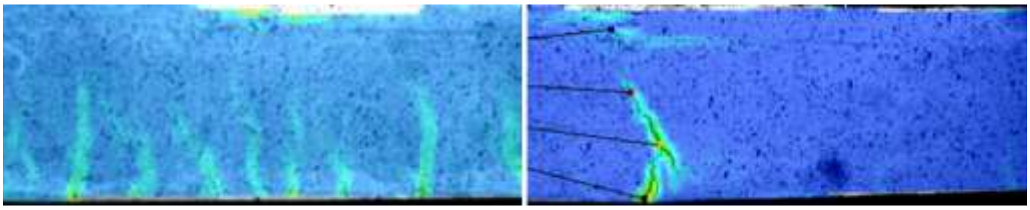

Fig. 9: Major strains at maximum load (left) and at failure (right) for beam 11

Figure 8 also gives the crack pattern for some of the beams failing in shear. The shear cracking development was as expected for this failure mode. First flexural cracks were formed. For the beams without fibre one of the flexural cracks developed into a diagonal crack towards the load. The beams with fibre had a somewhat different crack development. The shear cracks did not develop in the continuation of the flexural cracks. They formed approximately at mid height before propagating towards the loading and down to the tensile reinforcement. A general observation was that in beams with 0.5% fibre several shear cracks formed before one crack initiated the shear failure, while beams with 1% of fibre only developed one significant shear crack before failure. The visual observation of the shear cracks did not detect any major influence of the inter-layer surface. When the shear cracks reached the interface to the normal density concrete the propagation of the crack stopped for a short time before continuing into the top layer with the same angle.

4.3 Discussions of the results

For the beams that experienced shear failure, fibre counting was carried out in order to directly relate the performance to the amount of fibre reinforcement in the critical section. A specimen of with a width of 50mm was sawed from the beam, starting 100mm from where the diagonal crack crossed the interface between the NC and the FRLWC. First, the fibres crossing a section normal to the longitudinal direction were counted, n_A, section A-A in Figure 10. The small specimen was then sawed into two separate pieces, dividing the specimen at a section parallel to the diagonal crack. Even if it was only a limited area available for counting fibres, the results gave an indication on the amount of fibres crossing the diagonal shear crack, n_n. As seen from Table 4, the number of fibres was approximately doubled when the nominal fibre volume was doubled. The orientation factor normal to the crack, α_n, was considerably lower than the orientation factor in the longitudinal direction, α_A, which indicated that the flow had an effect on the orientation of the fibres in the test series. For beams 13-16, the orientation factor is almost constant normal to the crack, meaning that the flow of fresh concrete had an effect on the orientation, mostly in the longitudinal direction. When relating the number of fibres to the performance of the shear beams in terms of failure loads, P_{fail}, and load at diagonal crack initiation, P_{diag}, the best correlation was using the fibre counting normal to the crack. From Figure 11, it can be seen there is an almost linear relationship between cracking load and the number of fibres for beams 3-8.

Table 4: Results for the shear beams

Beam	n_A [pr. cm^2]	α_A	n_n [pr. cm^2]	α_n	δ_{diag} [mm]	δ_{peak} [mm]	W_{diag} [kNm]	W_{fail} [kNm]	μ	ρ_0 [10^{-3}]
1-0%	-	-	-	-	2.2	6.7	0.03	0.18	-	
2-0%	-	-	-	-	3.0	6.3	0.04	0.13	-	
3-0.5%	1.69	0.80	0.83	0.40	3.8	7.3	0.10	0.32	3.2	1.7
4-0.5%	1.29	0.61	0.50	0.24	3.7	6.4	0.09	0.23	2.6	1.3
5-0.5%	1.40	0.67	1.08	0.51	3.1	7.9	0.08	0.31	3.9	1.6
6-1.0%	2.98	0.71	2.19	0.52	6.5	12.2	0.27	0.76	2.8	2.4
7-1.0%	2.53	0.60	1.81	0.43	5.4	8.1	0.21	0.45	2.2	2.2
8-1.0%	2.64	0.63	1.35	0.32	5.6	8.5	0.19	0.42	2.2	2.1
13-0.5%	1.67	0.79	0.77	0.36	5.4	6.1	0.13	0.18	1.4	1.0
14-0.5%	1.74	0.83	0.87	0.42	5.7	9.0	0.13	0.29	2.2	1.2
15-1.0%	3.34	0.79	2.02	0.48	8.6	15.9	0.28	0.75	2.7	1.8
16-1.0%	2.05	0.49	1.71	0.41	6.7	12.9	0.19	0.55	2.9	1.6

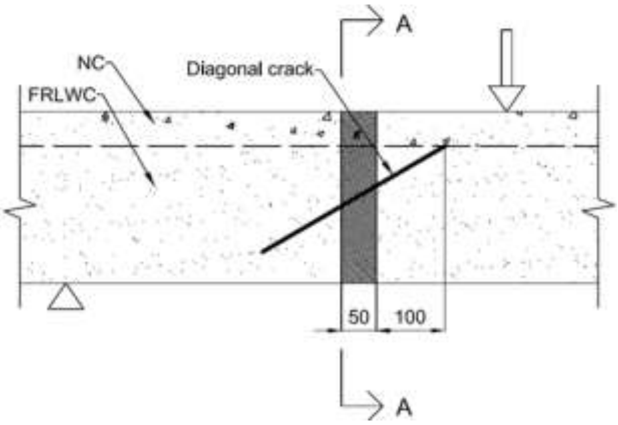

Fig. 10: Sections for fibre counting

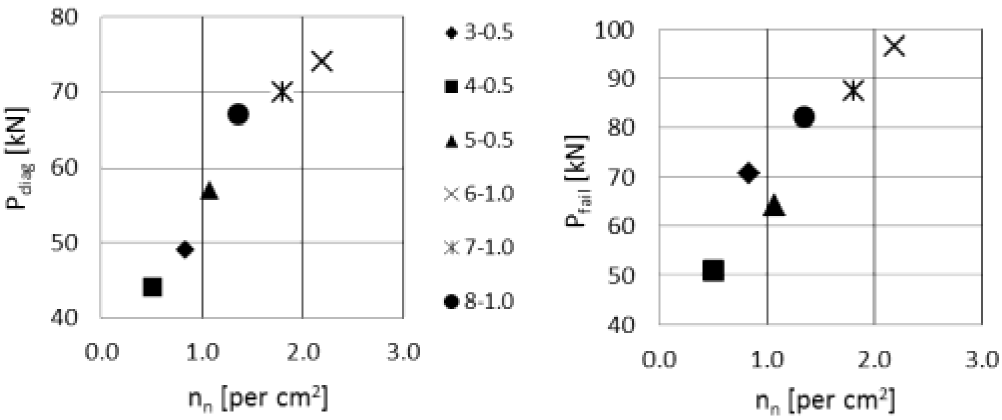

Fig. 11: Correlation between diagonal cracking load, failure load and number of fibres for beams 3-8

Ductility is the ability for a structural member to deform inelastically without any significant loss of strength. A common way of quantifying ductility in reinforced concrete is to employ a ductility index, which on member level often is defined as the ratio of displacement at a specified post-peak value to that of the yielding of the reinforcement. In this study most of the beams experienced shear mode of failure. Thus, it was more suitable to investigate ductility in terms of the consumed displacement energy in the tests. The displacement energies at initiation of the shear crack, W_{diag}, and at peak load, W_{peak}, were calculated based on the corresponding displacements, δ_{diag} and δ_{peak}. From Table 4 it can be seen that the concrete contribution to the displacement energy (beams 1-2) is at the same level as the contribution from 0.5% fibre (beams 3-5). The total consumed energy is approximately doubled when the fibre content is doubled. A ductility index, μ, is also defined in Table 4 as the ratio between W_{diag} and W_{peak}. It is not possible to fine a clear correlation between the fibre content and the ductility index. The index is roughly at the same level for all beams. One reason for this is that it is not a pure shear ductility index but based on the total shear and flexural deformations.

The main reason for using fibres in these beams was to avoid conventional shear reinforcement. A way of quantifying the effect of fibres is to calculate the required shear reinforcement for the obtained capacities in the beams. The required shear reinforcement ratios, ρ_0, are given in Table 4 and were calculated according to the truss model in Eurocode 2:

$$P_{Fail} = \rho_0 \cdot z \cdot b \cdot f_y \cdot cot\theta \qquad (4)$$

where z is the inner level arm, b the width of the cross section, f_y the yield strength of the shear reinforcement assumed to be 550 MPa, and θ the angle of the compression strut chosen to 22°. The results have a great variation with required shear reinforcement ratios of 1.0-1.7 and 1.6-2.4 for beams with 0.5% and 1.0% of fibre. However, the results give an indication of the possibility to reduce traditional shear reinforcement.

To investigate the interface between the NC and FRLWC layer, the shear force parallel to the interface can be calculated. For composite beams, a shear force K parallel to an interface between two materials in the longitudinal direction of the beam can be found as:

$$K = \frac{\sum E_i S_i'}{\sum E_i I_i} \cdot V \qquad (5)$$

where S'_i is the first moment of the area, E_i is the Young's modulus and I_i the moment of inertia, all valid for the current layer of material. The maximum shear force obtained from the testing of beams without vertical bar reinforcement was 35 kN. Using Equation (5), the shear force K at interface yields 186 N/mm calculated at the point of support. In order to compare the calculated shear force with the obtained shear resistance from the Bi-Surface Shear Test, the force is distributed over a corresponding length as the small-scale specimens (150mm). The shear stress at beam failure then becomes 1.24 MPa. The lowest obtained shear capacities from the small-scale test ranged was 1.76 MPa, indicating sufficient shear resistance. In sections closer to the point of loading, the stress states are more complex, and during the beam test the neutral axis moved towards the interface due to bending actions, indicating larger shear stresses at the interface as the maximum shear stress follows the neutral axis. However, compressive forces from the loading also act on the interface, and the direction of principal stresses and strains do not coincide with the axis of the beam. In addition, the beam was symmetrical in the longitudinal direction, thus providing fixed conditions at the mid-span. For this reason, the stress situation to the side of the loading was more favourable for the interface than close to the support.

5 Conclusions

This study shows that the concept of combining the NC and FRLWC in one cross-section shows promising results under short term loading, and no problem with the bond between the layers of concrete was registered. Durability and long term response were not considered in this work. The effect of creep stress redistribution and differential shrinkage strains between the layers are issues which must be identified in future investigations of this type of structure. Steel fibre reinforcement of the lightweight concrete increased the ductility in tension, and the amount of conventional shear reinforcement might be reduced. However, there was a lack of ductility in the beams designed for bending failure since they failed in shear when replacing shear reinforcement with fibres. Also, the bending failure of these composite beams is less ductile than the bending failure of a typical reinforced concrete beam, due to the interactions between the different parts of the system. The shear capacity increases with an increasing number of fibres but the results are strongly dependent on the distribution and orientation of the fibres. Results from fibre counting indicate that 1% of fibres give a more 3-dimensional orientation of the fibres, while 0.5% of fibres tend to orientate in the longitudinal direction of the beams. A new study currently under development is carried out to investigate the effect of dispersion and distribution of fibres on structural response, where the key issues are workability, flowability and casting methods. The concept presented in this paper provides low a self-weight of the structure, practical solutions in the construction phase and good premises for more efficient building.

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