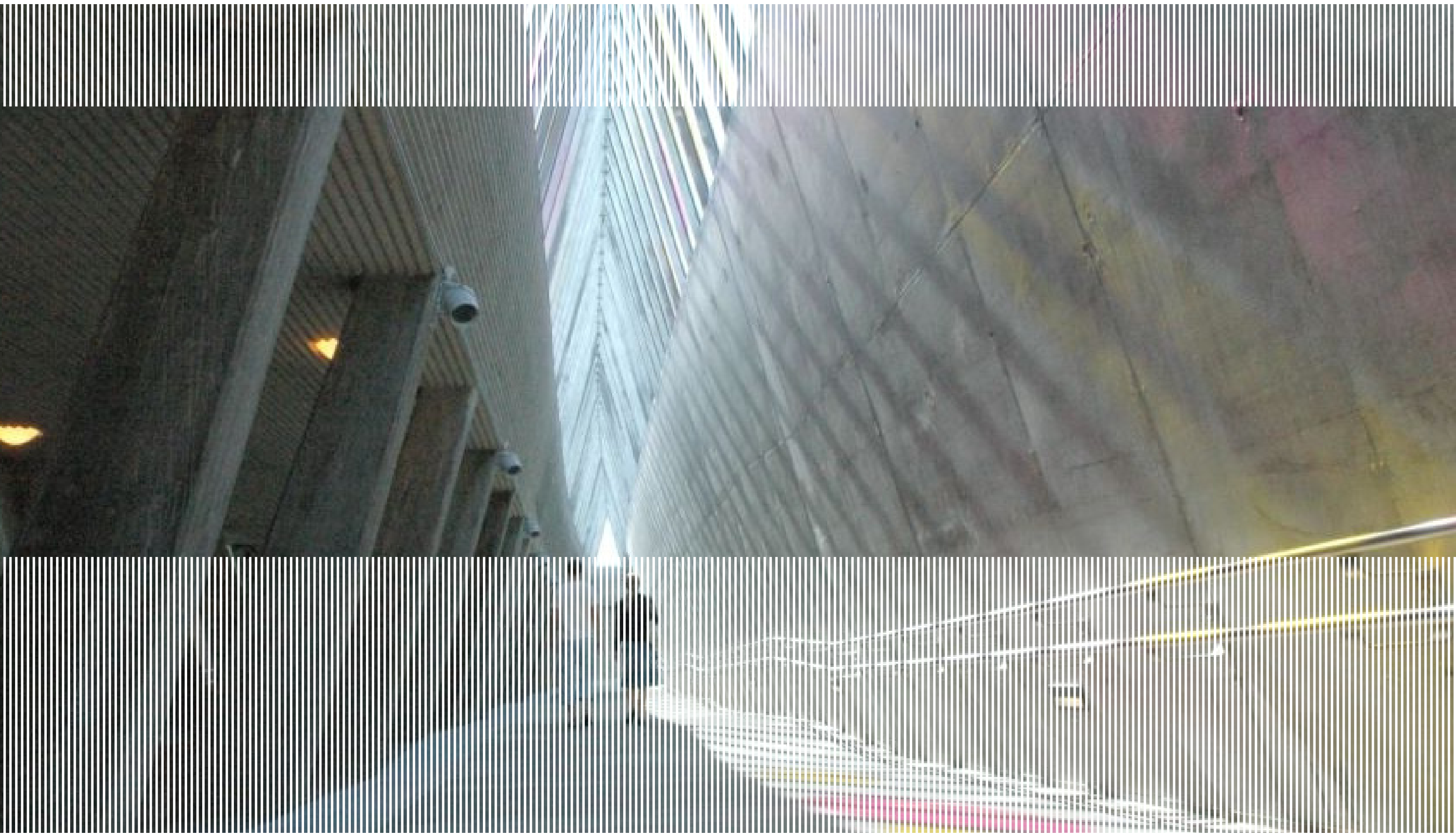
SINTEF Building and Infrastructure Terje Kanstad (NTNU)

Fibre Reinforced Superlight Concrete: Testing of Materials and Full Scale Beams

COIN Project report 15 - 2009



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COIN P 3 Innovative construction concepts

SP 3.1 Fibre reinforced concrete

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Preface

This study has been carried out within COIN - Concrete Innovation Centre - one of presently 14 Centres for Research based Innovation (CRI), which is an initiative by the Research Council of Norway. The main objective for the CRIs is to enhance the capability of the business sector to innovate by focusing on long-term research based on forging close alliances between research-intensive enterprises and prominent research groups.

The vision of COIN is creation of more attractive concrete buildings and constructions. Attractiveness implies aesthetics, functionality, sustainability, energy efficiency, indoor climate, industrialized construction, improved work environment, and cost efficiency during the whole service life. The primary goal is to fulfil this vision by bringing the development a major leap forward by more fundamental understanding of the mechanisms in order to develop advanced materials, efficient construction techniques and new design concepts combined with more environmentally friendly material production.

The corporate partners are leading multinational companies in the cement and building industry and the aim of COIN is to increase their value creation and strengthen their research activities in Norway. Our over-all ambition is to establish COIN as the display window for concrete innovation in Europe.

About 25 researchers from SINTEF (host), the Norwegian University of Science and Technology - NTNU (research partner) and industry partners, 15 - 20 PhD-students, 5 - 10 MSc-students every year and a number of international guest researchers, work on presently 5 projects:

- Advanced cementing materials and admixtures
- Improved construction techniques
- Innovative construction concepts
- Operational service life design
- Energy efficiency and comfort of concrete structures

COIN has presently a budget of NOK 200 mill over 8 years (from 2007), and is financed by the Research Council of Norway (approx. 40 %), industrial partners (approx 45 %) and by SINTEF Building and Infrastructure and NTNU (in all approx 15 %).

For more information, see www.coinweb.no

Tor Arne Hammer
Centre Manager

Summary

Background

A test programme with recently developed prefabricated lightweight concrete beams (1100-1200 kg/m^3) for use in masonry structures has been carried out. The programme includes comprehensive material testing, and the effects of different fibre reinforcement types as synthetic micro fibres (fibremesh), synthetic macro fibres and steel fibres are investigated. The test program is a cooperation between Saint Gobain Weber Federals, SINTEF and NTNU, and is partly financed by COIN.

The beams are produced with lengths 1,5 or 3 meter, L-shaped cross section with height 390mm, net web width 45mm and ordinary reinforcement steel in the length and transversal direction. The planned self weight is 80 kg to satisfy HES requirements so that two persons can carry the beam at building sites. This requirement is the main reason for using lightweight concrete. The beam will be used over window and door openings within the”Leca Isoblokk 35 cm building system”. In the finished wall two beams are placed with 150 mm EPS- or Rockwool-isolation between, and the opening between the isolation and the beam webs will be filled with ordinary concrete. Thus the compressive zone is strengthened in the finished state. An important objective by the project is to contribute towards optimum performance and verify that the beam has sufficient stiffness and strength. Furthermore to show that there is reasonable agreement between the structural behaviour of the beam and the calculation methods.

The lightweight concrete which is developed by Saint Gobain Weber is a rather new concept, and contains Leca aggregate and foam to achieve satisfactory low weight. The self weight lies within the range 11-1200 kg/m^3, and the desired compressive cylinder strength is 20 N/mm^2. The content of the foam is restricted, but the behaviour and appearance can be compared to shaving foam. In general six types of macro fibres were used, five types of steel fibres and one type synthetic fibres. In addition were synthetic microfibres (fibremesh) used in some of the mixes.

The programme, carried out in the period September -08 till –august 09, comprises strength testing, creep and shrinkage, pullout of single fibres, uniaxial tensile testing, and standard 4-points beam testing in addition to the full scale beams. This is much more comprehensive than necessary for verification of the structural load test behaviour of the beams because it is a relevant part of COIN’s general fibre concrete research activity.

Compressive strength

Four slightly different receipts have been used, and the strength variation might seem too large. The average compressive strength is approximately 15 N/mm^2, and variation range is between 10 and 22 N/mm^2. Obviously this influences the evaluation of the test results, and makes it difficult to sort the effect of certain parameters.

Shrinkage- and creep properties

The applied lightweight concrete has a considerably large drying shrinkage which due to restraint from the longitudinal reinforcement resulted in shrinkage cracks in all the full scale beams. The fibre reinforcement reduced the crack width and the number of cracks. The concrete has also a relatively high creep ratio, however, this has no important consequences because a large part of the compressive forces are carried by the additional B20 compressive zone.

Pullout testing of single fibres

Compared to previous test results with ordinary concrete, the pullout testing for 50-60 mm long fibres in the lightweight concrete gave surprisingly large capacity considering the low tensile strength of this concrete. For 35 mm long fibres is, however, the ductility considerably lower because of occurrence of conical failure surfaces in the concrete surrounding the fibre. This is due to the low tensile strength of the lightweight concrete. Addition of 0,3% (2,7kg/m^3) microfibres compensates partly for this and gives a clear improvement of the ductility of the pullout behaviour of the shortest fibres.

Standard 4-points beam testing

This is the main standard test method in the proposal for Norwegian guidelines for design and execution of fibre reinforced concrete where beams of dimension b/h/L=150/150/600 are used. Storage in climatic room gave in most cases a considerably lower residual strength than storage in water (from 0 to 50% reduction). The effect is larger than for normal concrete, and is due to the large shrinkage. For fibre volume between 0,5 and 1,0%, a relatively ductile behavior is achieved for all fibre types. Maximum capacity in general occurs a while after the first crack, and at a deformation of L/200 the load is reduced by 25-40%.

Full scale beams

Most of the beams were tested as net lightweight concrete beams, and the most interesting observations are:

- In the beams without fibres, the failure started as an anchorage failure with a major horizontal crack in the interface between the flange and the web which developed further as a diagonal crack.
- All fibre types improved the bond and the tensile robustness so that this type of failure did not occur in fibre reinforced beams. The final failure then occurred in the compressive zone due to insufficient moment capacity. The failure came after comprehensive diagonal cracking.
- The beams with 35 mm end hooked steel fibres gave in general largest capacity. The compressive strength of the concrete is however lower than prescribed, and its variation is too large, which makes it difficult to range the effect of other fibre types.
- Five fullscale beams with additional cast-in-place concrete was also tested, and all of them had sufficient capacity, ranging from 1,74 to 2,5 times the prescribed design capacity. The beam with 0,5% end hooked steel fibres had the largest capacity, and this probably means that the design capacity can be increased considerably (roughly estimated to 75%). Alternatively can the amount of shear and longitudinal reinforcement be reduced. However, more tests have to be carried out to verify this.

Table of contents

1 Introduction

This chapter gives a short presentation of the prefabricated Light Weight Aggregate Concrete (LWAC) beam, which is the origin of the present test programme.

The beam has L-shaped cross-section, length of either 1,5 or 3 m, and geometry and reinforcement layout as shown in figures 1 and 2. The longitudinal reinforcement is 2ø12 in the bottom and 1ø8 in the top, while the transversal shear reinforcement is single Ø6 spacing 100 mm. The reinforcement quality is B500C.

The beam's planned self weight is 80 kg to satisfy health and security requirements so that two persons may carry and place it in the right position without crane-equipment. This is the main reason for using LWAC. It is meant for use above windows- and door-openings within maxit's "Leca Isoblokk 35 cm" building-system (see figure 1b). In the finished wall structure two beams are placed towards each other with 150 mm EPS isolation between. The openings between the isolation and the beam-webs are filled with ordinary concrete of quality B20 to strengthen the beam.

Design values for the capacities in the Ultimate Limit States for each beam including the B20 concrete, is in maxit's product-catalogue given as:

Moment capacity:	21,0 kNm
Shear capacity:	22,0 kN

An important objective by this project is to contribute towards optimum performance, and verify that the beam satisfies these requirements.

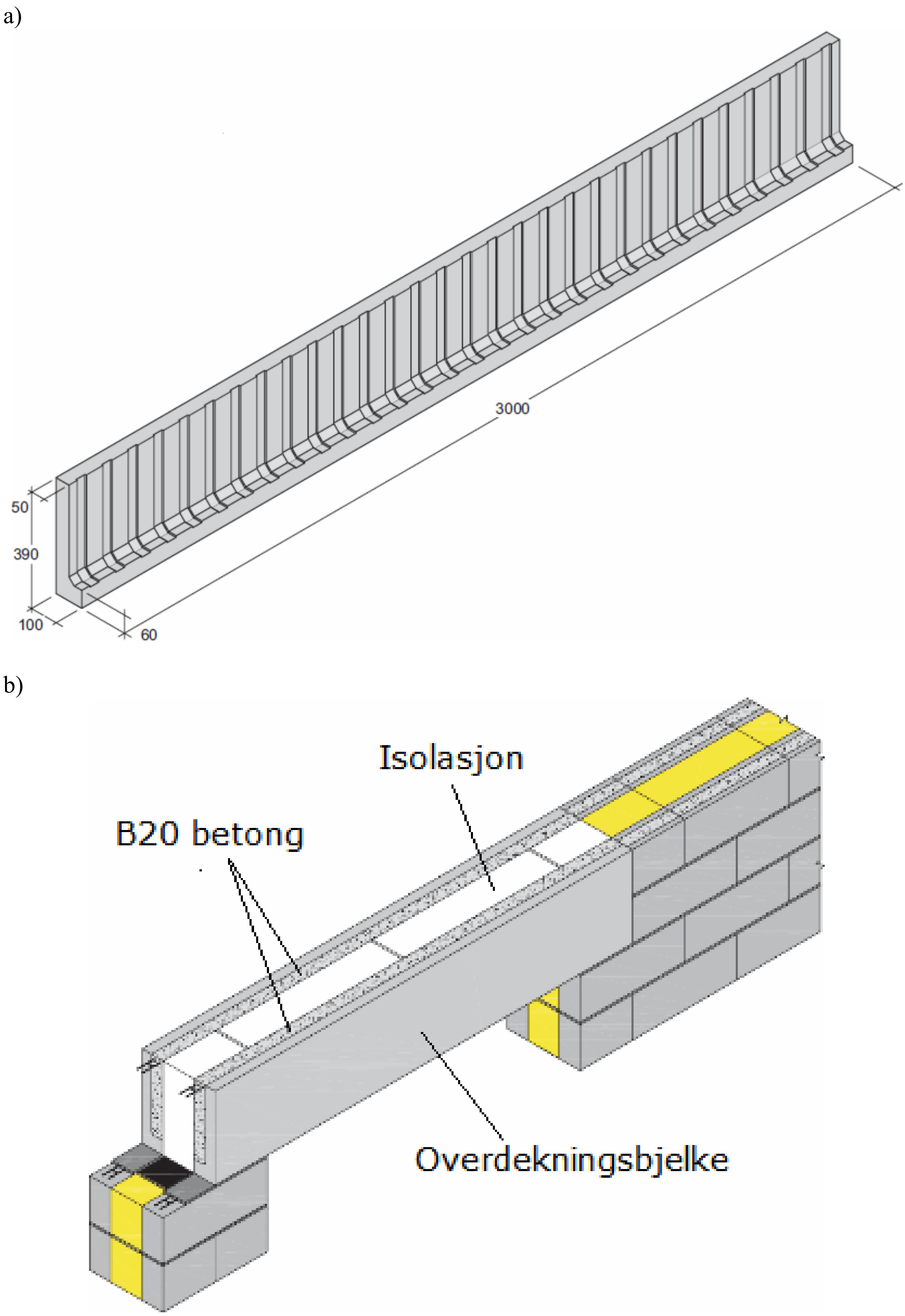


Figure 1. Geometry, layout and performance of maxit's prefabricated beam

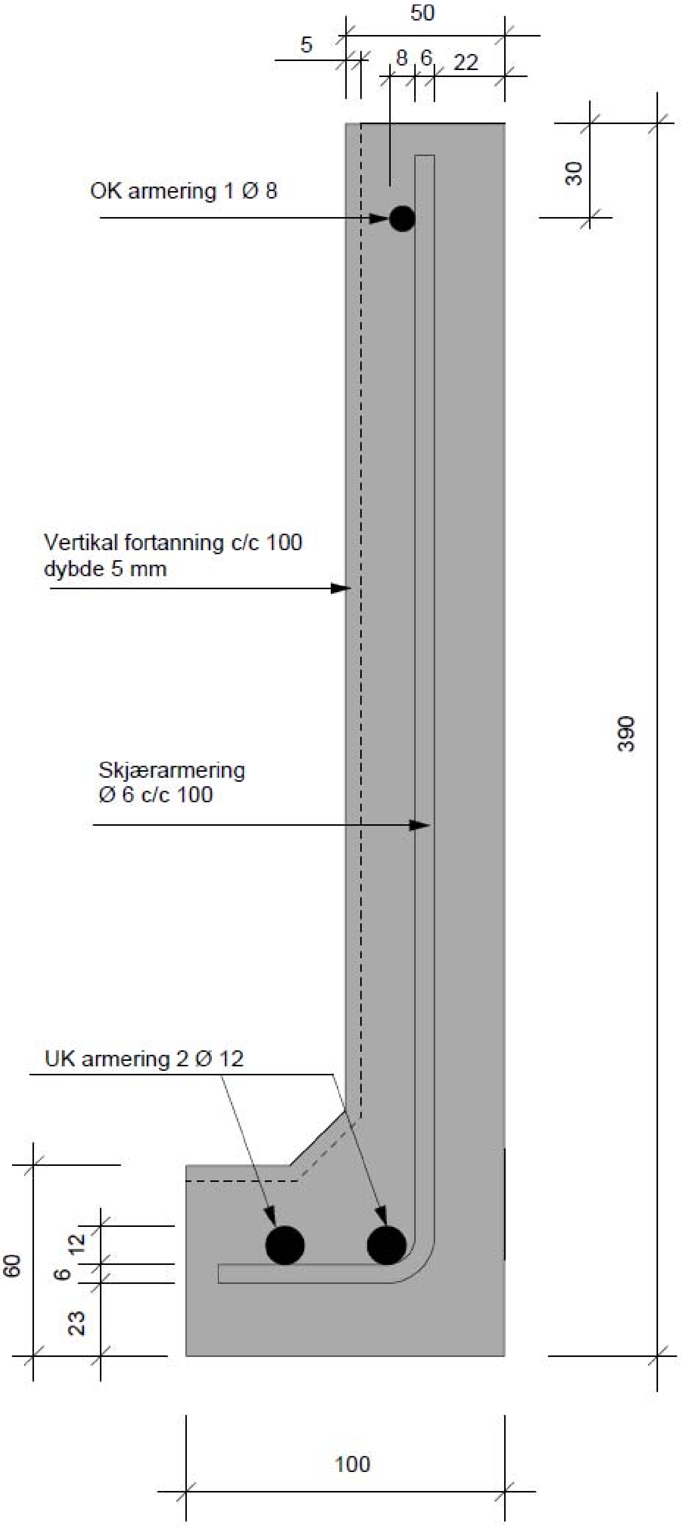


Figure 2. Cross section of the prefabricated beam

2 Experimental programme

2.1 Compressive Strength Testing

Cylinders for compressive strength testing were cast from most of the concrete mixes used within the project. Due to the relatively small concrete mixer at maxit Lillestrøm, and therefore a large number of similar mixes, some of the cylinders are made as an average of two or more mixes. The purpose by the compressive testing was to verify that the concrete reached the prescribed strength, determine material parameters for the calculation methods, and to use the strength data for evaluation of the results in general. Due to several reasons, as explained in the subsequent section 3.2.2 dealing with the concrete production, 5 different receipts denoted M1-M5 were used throughout the test programme.

The compressive strength testing is carried out in accordance with the standard test method at NTNU.

2.2 Fibre Pullout Tests

Pullout tests of single fibres from the LWAC were carried out in several series as continuation of previous work for mapping of the behaviour and effectivity of different fibre types in various concrete mixes [Sandbakk et al 2009]. The test method, see figure 3, is described in detail by [Døssland 2008] and [Sandbakk et al 2009]. Several types of fibres were included, and reference tests were conducted for comparison with previous tests. Synthetic microfibres (fibremesh) were also added to the concrete in some of the pullout tests to investigate if this could have a significant positive effect on the pullout properties of the macro fibres.

Furthermore were pullout tests of some fibre types from ordinary concrete (B35) conducted due to interesting findings in the LWAC tests. Table 1 gives an overview over the test specimens of LWAC, and table 2 correspondingly for ordinary concrete. In general 5 parallels are carried out for each varying parameter. All LWAC-specimens were cast with receipt M1 as defined in section 3.2.2.

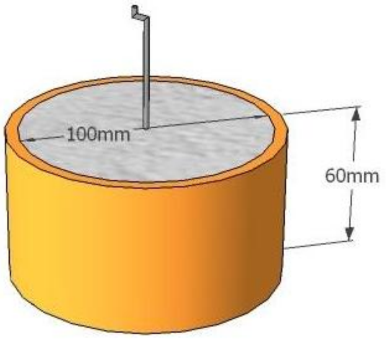
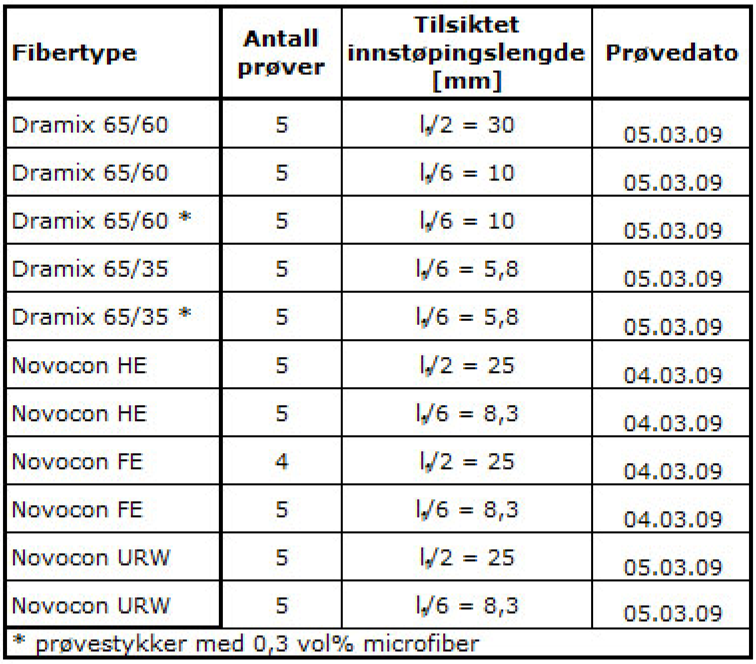


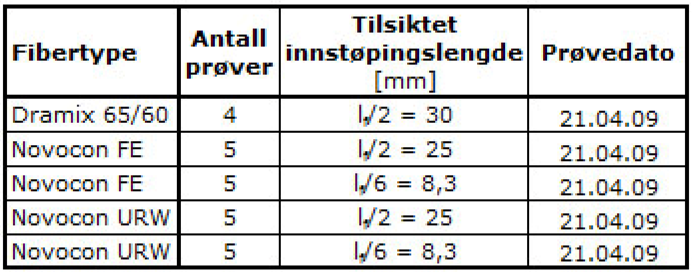
Figure 3. Test specimen for pullout of single fibres

Table 1 – Overview of pullout test specimens in LWAC presenting fibre type, number of parallels, anchor length, and test date. Receipt M1 was used in all the specimens.



Fibertype	Antall prøver	Tilsiktet innstøpingslengde [mm]	Prøvedato
Dramix 65/60	5	$l_f/2 = 30$	05.03.09
Dramix 65/60	5	$l_f/6 = 10$	05.03.09
Dramix 65/60 *	5	$l_f/6 = 10$	05.03.09
Dramix 65/35	5	$l_f/6 = 5,8$	05.03.09
Dramix 65/35 *	5	$l_f/6 = 5,8$	05.03.09
Novocon HE	5	$l_f/2 = 25$	04.03.09
Novocon HE	5	$l_f/6 = 8,3$	04.03.09
Novocon FE	4	$l_f/2 = 25$	04.03.09
Novocon FE	5	$l_f/6 = 8,3$	04.03.09
Novocon URW	5	$l_f/2 = 25$	05.03.09
Novocon URW	5	$l_f/6 = 8,3$	05.03.09
* prøvestykker med 0,3 vol% microfiber			

Table 2 – Overview of pullout test specimens in ordinary concrete presenting fibre type, number of parallels, anchor length, and test date.



Fibertype	Antall prøver	Tilsiktet innstøpingslengde [mm]	Prøvedato
Dramix 65/60	4	$l_f/2 = 30$	21.04.09
Novocon FE	5	$l_f/2 = 25$	21.04.09
Novocon FE	5	$l_f/6 = 8,3$	21.04.09
Novocon URW	5	$l_f/2 = 25$	21.04.09
Novocon URW	5	$l_f/6 = 8,3$	21.04.09

The results give anchorage capacity and load-displacement relation for single fibres. This is basic information for the calculation models, and indicates which fibres are most interesting to use in tensile- and bending tests.

2.3 Uniaxial Tensile Test

In total 34 uniaxial tensile test specimens with dimension 100x100x600 mm were cast, in three casting series. The results are most relevant for the calculation methods used in the service limit states, and the rules for minimum reinforcement.

Table 3 shows an overview of the specimens, and in addition to fibre type and –amount, two storage types are used (climatic conditions (T=20^{o}C, RH=50%) and in water). The first and the third casting were carried out at maxit Lillestrøm, while the second was carried out in the concrete laboratory at NTNU. In addition to the fibres, some of the specimens contained longitudinal reinforcement, and reference specimens without fibres were also cast. The specimens were tested by uniaxial tension in the test rig shown in figure 4, which was not used for FRC earlier.

The purpose with these tests was to determine a possible increased tensile strength due to fibre addition, and to see if it is possible to obtain strain hardening without conventional reinforcement for this particular FRC, and thereby fulfill the minimum reinforcement requirement with fibres only.

The test method is described in detail by [Strandgård 2008], while the results are presented in Chapter 4.3.

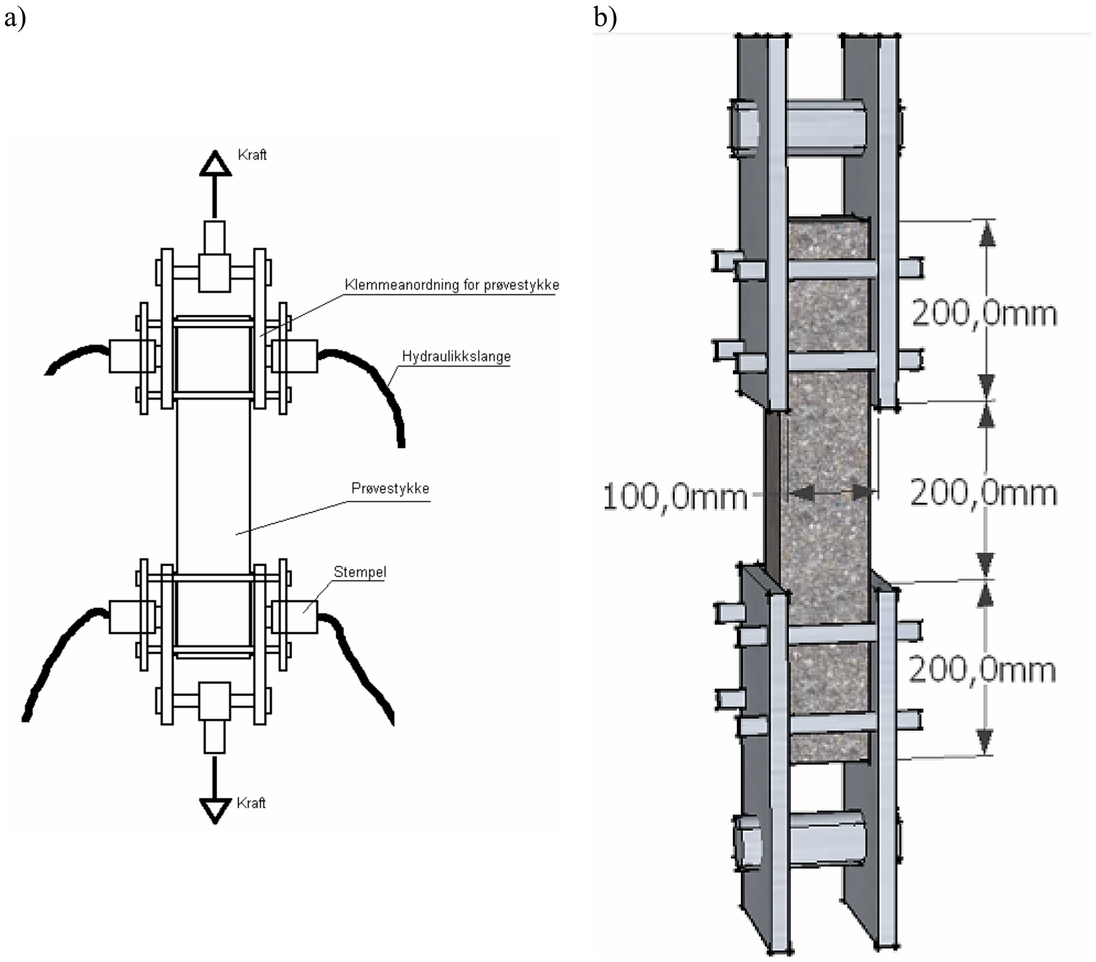
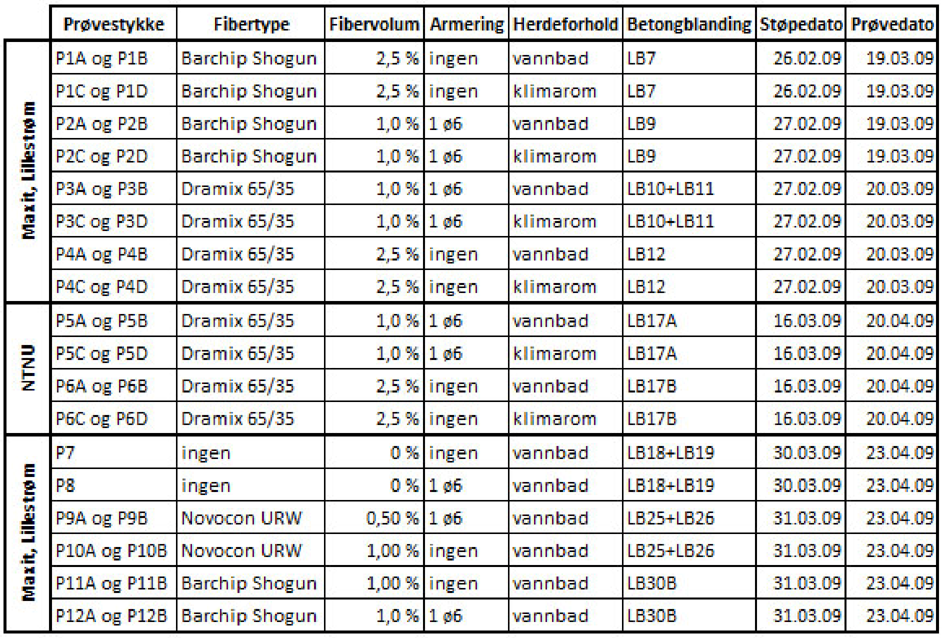


Figure 4. – Schematic sketches of the uniaxial tensile test rig. [Myhre 2008].

Table 3 – Overview of the uniaxial tensile test specimens, presenting production place, fibre type and –volume, bar-reinforcement, storage conditions, name of concrete mix, casting and test date and receipt name.



	Prøvestykke	Fibertype	Fibervolum	Armering	Herdeforhold	Betongblanding	Støpedato	Prøvedato	Receipt
Maxit, Lillestrøm	P1A og P1B	Barchip Shogun	2,5 %	ingen	vannbad	LB7	26.02.09	19.03.09	M2
	P1C og P1D	Barchip Shogun	2,5 %	ingen	klimarom	LB7	26.02.09	19.03.09	
	P2A og P2B	Barchip Shogun	1,0 %	1 ø6	vannbad	LB9	27.02.09	19.03.09	M3
	P2C og P2D	Barchip Shogun	1,0 %	1 ø6	klimarom	LB9	27.02.09	19.03.09	
	P3A og P3B	Dramix 65/35	1,0 %	1 ø6	vannbad	LB10+LB11	27.02.09	20.03.09	
	P3C og P3D	Dramix 65/35	1,0 %	1 ø6	klimarom	LB10+LB11	27.02.09	20.03.09	
	P4A og P4B	Dramix 65/35	2,5 %	ingen	vannbad	LB12	27.02.09	20.03.09	
	P4C og P4D	Dramix 65/35	2,5 %	ingen	klimarom	LB12	27.02.09	20.03.09	
NTNU	P5A og P5B	Dramix 65/35	1,0 %	1 ø6	vannbad	LB17A	16.03.09	20.04.09	M1
	P5C og P5D	Dramix 65/35	1,0 %	1 ø6	klimarom	LB17A	16.03.09	20.04.09	
	P6A og P6B	Dramix 65/35	2,5 %	ingen	vannbad	LB17B	16.03.09	20.04.09	
	P6C og P6D	Dramix 65/35	2,5 %	ingen	klimarom	LB17B	16.03.09	20.04.09	
Maxit, Lillestrøm	P7	ingen	0 %	ingen	vannbad	LB18+LB19	30.03.09	23.04.09	M4
	P8	ingen	0 %	1 ø6	vannbad	LB18+LB19	30.03.09	23.04.09	
	P9A og P9B	Novocon URW	0,50 %	1 ø6	vannbad	LB25+LB26	31.03.09	23.04.09	
	P10A og P10B	Novocon URW	1,00 %	ingen	vannbad	LB25+LB26	31.03.09	23.04.09	
	P11A og P11B	Barchip Shogun	1,00 %	ingen	vannbad	LB30B	31.03.09	23.04.09	
	P12A og P12B	Barchip Shogun	1,0 %	1 ø6	vannbad	LB30B	31.03.09	23.04.09	

2.4 Standard Testing: Beams Sawn from Slabs (150x600x600)

Bending of small beams is often used as the standardized test method for determination of the residual tensile strength of FRC which is the main model parameter in capacity calculations. The objective by these tests is further to determine and compare the residual tensile strength for different fibre types and fibre volumes in the actual LWAC.

2.4.1 Casting

The concrete was cast horizontally mainly from one of the edges in forms with dimensions shown in figure 5. It was experienced that concrete with 1 vol-% Barchip-fibres (synthetic) had better workability than concrete with the same amount steel fibres (Dramix 65/35), but poorer workability than concrete with 0,5 vol-% URW(steel). The forms were not vibrated during the casting process.

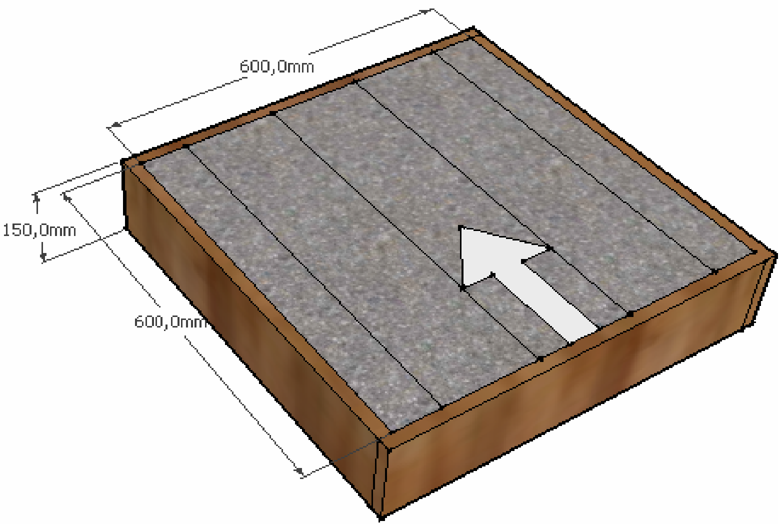


Figure 5 – Casting direction for Standard beams

2.4.2 Hardening

The moulds were removed after 1-2 days and transported covered with plastic sheets by car from Lillestrøm to NTNU in Trondheim, and thereafter stored either isolated (inside wet jute sacks and plastic) or in constant climatic conditions (RH=50, T=20) until testing. An overview is presented in table 4.

Before testing, the slabs were sawed to 3 beams with quadratic cross section 150x150mm and length 600mm as shown in figure 6. The beams are parallel to the concrete flow direction.

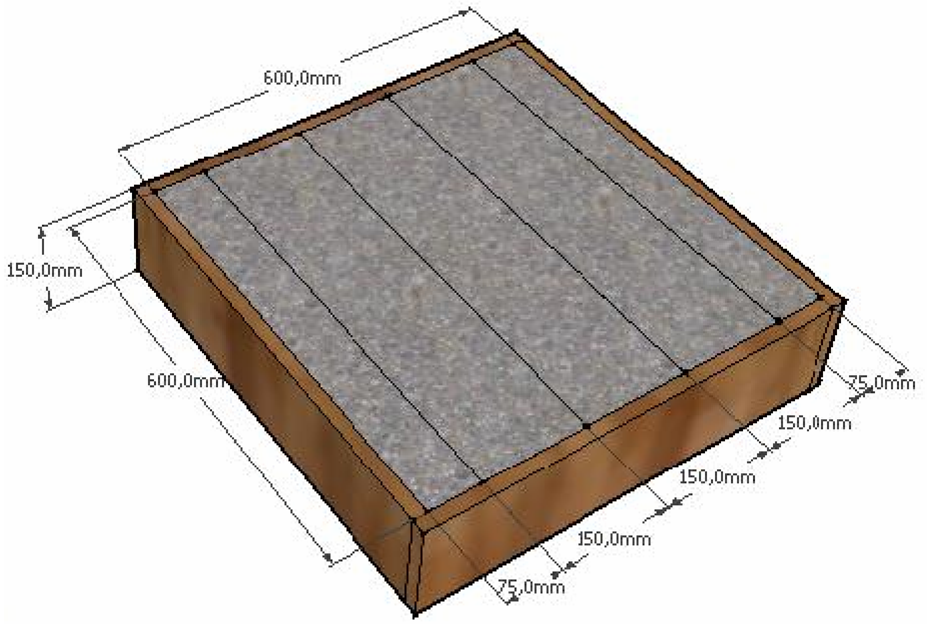


Figure 6 - Sawing pattern for Standard beams

2.4.3 Testing

The testing was carried out according to the Norwegian guidelines [Thorenfeldt & Fjeld et. al, 2006] with two exceptions:

(1) Displacement velocity 0,5mm/min instead of 0,1mm/min.
(2) The testing was continued after the prescribed limit given as 3,0mm to get a more complete picture of the load deformation relation.

The tests were run until final failure so that the number of fibres crossing the fracture surface could be counted, and it could be studied how the fibres crossing the failure zone was influenced by the loading.

The test rig with twin loads in the third points and displacement recorders at the mid section is shown in figure 7.

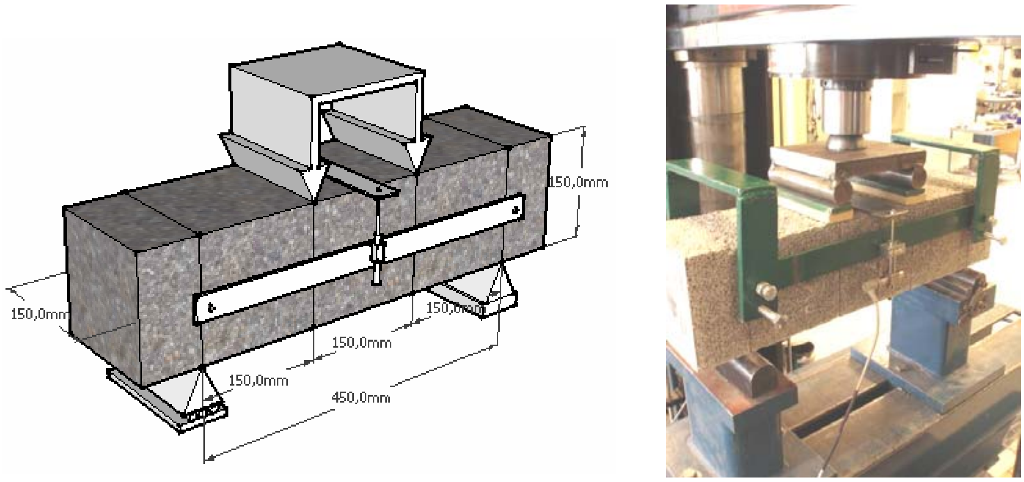


Figure 7 – Test rig for Standard beams

The test results are the load-displacement relations for the beams, and each of the beams are evaluated according to [Thorenfeldt & Fjeld et. al, 2006], where the force development in the displacement range 0,50mm - 2,50mm is basis for the calculations. From this range of the load displacement relation, the equivalent bending residual tensile strength is calculated.

Table 4 – Overview of tested standard beams, including fibre type and –volume, storage conditions, name of concrete mix, casting date, test date and receipt no.

Prøvestykke	Fibertype	Fibervolum	Mesh	Herdeforhold	Betongblanding	Støpedato	Prøvedato	Receipt
B1A til B1C	Dramix 65/35	0,5 %	-	Vannbad	LB2+LB3	26.02.09	Langtid	M2
B2A til B2C	Barchip Shogun	1,0 %	-	Vannbad	LB8	26.02.09	Langtid	
B3A til B3C	Dramix 65/35	0,5 %	0,30 %	Vannbad	LB13	27.02.09	23.03.09	M3
B3D til B3F	Dramix 65/35	0,5 %	0,30 %	Klimarom	LB13	27.02.09	24.03.09	
B4A til B4C	Dramix 65/35	1,0 %	0,30 %	Vannbad	LB14	27.02.09	Forkastet	
B4D til B4F	Dramix 65/35	1,0 %	0,30 %	Klimarom	LB14	27.02.09	Forkastet	
B5A til B5C	Barchip Shogun	0,5 %	0,30 %	Vannbad	LB15	27.02.09	23.03.09	
B5D til B5F	Barchip Shogun	0,5 %	0,30 %	Klimarom	LB15	27.02.09	24.03.09	
B6A til B6C	Barchip Shogun	1,0 %	0,30 %	Vannbad	LB16	27.02.09	23.03.09	
B6D til B6F	Barchip Shogun	1,0 %	0,30 %	Klimarom	LB16	27.02.09	24.03.09	
B7A til B7C	Dramix 65/35	0,5 %	0,17 %	Vannbad	LB20	30.03.09	Langtid	M4
B8A til B8C	Dramix 65/35	0,5 %	0,30 %	Vannbad	LB23A	30.03.09	04.05.09	
B8D til B8F	Dramix 65/35	0,5 %	0,30 %	Klimarom	LB23A	30.03.09	05.05.09	
B9A til B9C	Dramix 65/35	1,0 %	0,30 %	Vannbad	LB23B	30.03.09	04.05.09	
B9D til B9F	Dramix 65/35	1,0 %	0,30 %	Klimarom	LB23B	30.03.09	05.05.09	
B10A til B10C	Novocon URW	0,5 %	0,30 %	Vannbad	LB24	30.03.09	04.05.09	
B10D til B10F	Novocon URW	0,5 %	0,30 %	Klimarom	LB24	30.03.09	05.05.09	
B11A til B11C	Barchip Shogun	1,0 %	0,17 %	Vannbad	LB30B	31.03.09	Langtid	
B12A til B12C	Barchip Shogun	1,0 %	0,30 %	Vannbad	LB30C	31.03.09	04.05.09	
B12D til B12F	Barchip Shogun	1,0 %	0,30 %	Klimarom	LB30C	31.03.09	05.05.09	

2.5 Four point Bending of Full Scale Beams

2.5.1 Casting and Storage

The beams previously described in chapter 1, were cast in the factory of maxit at Lillestrøm. The L-shaped moulds which could be vibrated, were turned upside-down with the tensile zone up during casting. Due to the slender web and the dense reinforcing net, the pouring situation was critical due to separation.

The slender cross section with bar reinforcement made casting with 1 vol% Dramix 65/35 rather difficult, and the fibres had a tendency to bundle around the ordinary reinforcement.

The beams were demoulded after approximately 20 hours, and stored in the production area before they were sent to NTNU in Trondheim. In total 36 beams were cast, with varying parameters as shown in table 5, and the production was carried out in two series with approximately one month interval. Fibre type and fibre volume were varied, and in addition were also reference beams without fibres produced. Some of the beams were performed with reduced amount of longitudinal or transversal reinforcement to provoke moment tensile- or shear failure. The age at testing was approximately 1 month for series 1 and 2, while it was around 5 months for series 3 as shown in table 5.

2.5.2 Testing

The tests were displacement controlled with 1mm/min as velocity. The deflection was recorded by LVDT's below both loads and at the middle section as shown in figure 8 below. The strains in the longitudinal direction were recorded by a LVDT at the height of the tensile reinforcement, and a strain gauge at the top of the cross section.

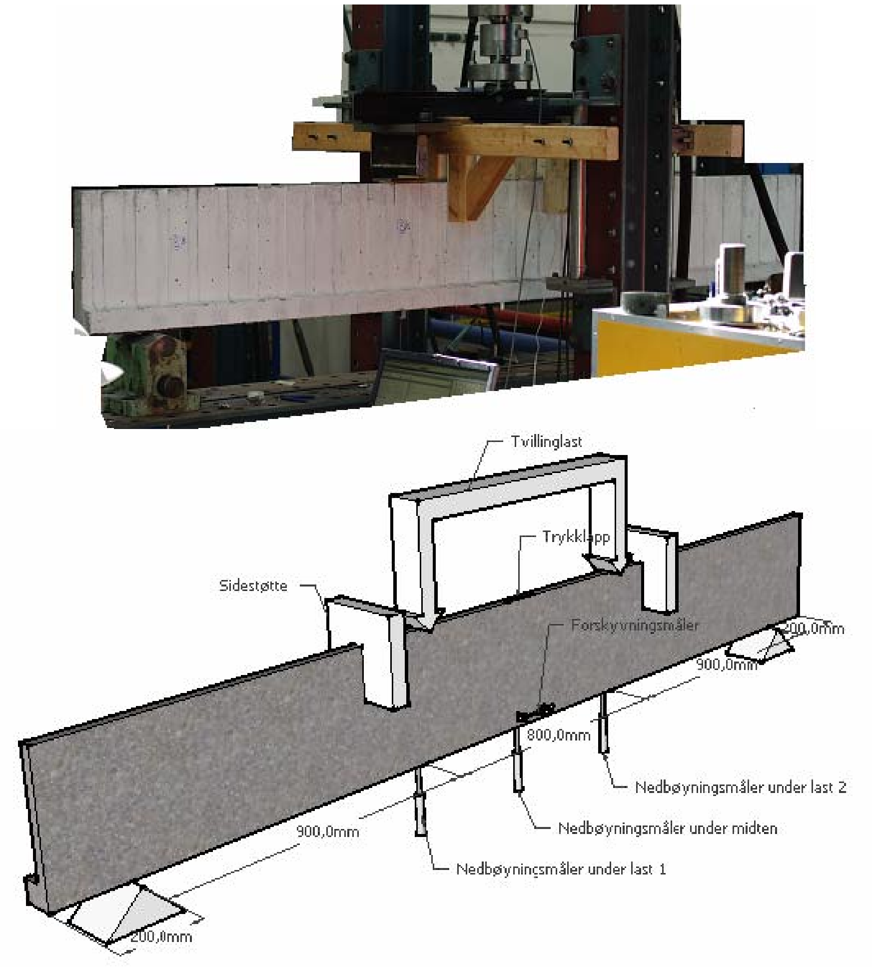


Figure 8 – Test rig and instrumentation for full scale beams

Series 1:
Because of separation problems in casting series 2 where these beams belong, it was decided to skip this series. However the beams were used to try the test rig out, and evaluate the effect of some of the variables. In these tests the beams were not instrumented, and only the load was recorded. After the testing it seemed like some of the results anyway were relevant, and the results are therefore also partly considered in the final evaluation of the results.

Series 2:
Based on the experience from series 1, it was decided to adjust the conventional reinforcement for some of the beams to achieve different types of failure:

(a) In 4 of the beams the shear reinforced was reduced to 50%
(b) In 2 beams the tensile reinforcement between the loads were reduced to 50%

The reinforcement nets are shown in figures 9-11, while an overview of the beams with the varying parameters is presented in table 5.

Series 3:
After finishing series 1 and 2, it was clear that the capacity was too low, and therefore decided that some of the beams should be tested with the additional cast-in-place concrete included (B20). The beams then correspond to the finished structure shown in figure 1. The six beams used for this purpose were originally meant for longtime testing.

In the final structure the spacing between the LWAC beam and the isolation shall be filled with B20 concrete, which contributes to the capacity. To achieve good bond and composite action, the beam surfaces (horizontal and vertical) are embossed with 5mm deep teeth as shown in the previous figures 1 and 2. The cast-in-place concrete (B20) was added when the beams were 4 months old, while the tests were carried out approximately 30 days later. The following beams were tested: 2 beams without fibres, 2 beams with 0,5% Barchip Shogun synthetic fibres and 1 beam with 0,5% Dramix 65/35 steel fibres.

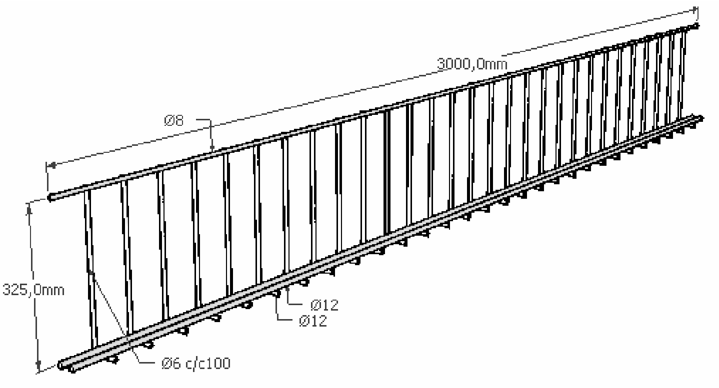


Figure 9 – The ordinary reinforcement net solution

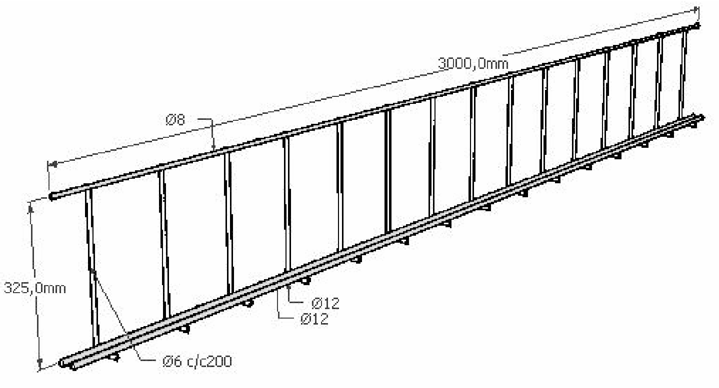


Figure 10 – Solution with reduced shear reinforcement

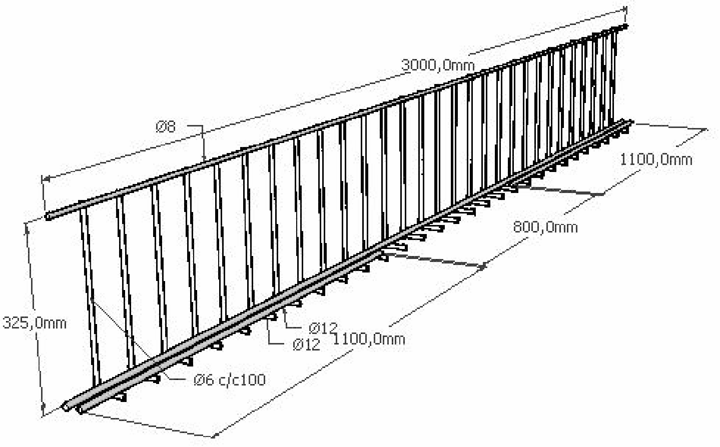


Figure 11 – Solution with reduced tensile reinforcement

Table 5 – Full scale beam tests, including fibre type and –volume, reinforcement solution, name of concrete mix, casting date, test date and receipt no.

	Beam No.	Fibre		Reinforce-ment	Concrete-mix	Date		Receipt
		Type	Volume			Casting	Testing	
Series 1	O1A	Dramix 65/35	0,50 %	Normal	LB2	26.02.09	25.03.09	M2
	O1B						25.03.09	
	O1C	Dramix 65/35	0,50 %	Normal	LB3	26.02.09	26.03.09	
	O1D						26.03.09	
	O2A	-	-	Normal	LB4	26.02.09	25.03.09	
	O2B						26.03.09	
	O3A	Barchip Shogun	1,00 %	Normal	LB6	26.02.09	25.03.09	
	O3B						25.03.09	
	O3C	Barchip Shogun	1,00 %	Normal	LB9	27.02.09	25.03.09	M3
	O3D						26.03.09	
	O4C	Dramix 65/35	1,00 %	Normal	LB11	27.02.09	26.03.09	
Series 2	O5A	-	-	Red. Shear	LB18	30.03.09	29.04.09	M4
	O5B	-	-	Normal	LB18	30.03.09	27.04.09	
	O6A	Dramix 65/35	0,50 %	Red. Shear	LB20	30.03.09	29.04.09	
	O6B						30.04.09	
	O6C	Dramix 65/35	0,50 %	Normal	LB21	30.03.09	29.04.09	
	O6D						29.04.09	
	O8A	Novocon URW	0,50 %	Red. Shear	LB25	31.03.09	30.04.09	
	O8B						30.04.09	
	O8C	Novocon URW	0,50 %	Red. Mom	LB26	31.03.09	28.04.09	
	O8D	Novocon URW	0,50 %	Normal	LB26	31.03.09	28.04.09	
	O9A	Barchip Shogun	0,50 %	Red. Shear	LB27	31.03.09	29.04.09	
	O9B						29.04.09	
	O9C	Barchip Shogun	0,50 %	Red. Mom	LB28	31.03.09	28.04.09	
	O10A	Barchip Shogun	0,50 %	Normal	LB29	31.03.09	27.04.09	
Serie 3	5C	-	-	Normal	LB19	30.03.09	Aug 09	
	5D	-	-	Normal	LB19	30.03.09	Aug 09	
	9D	Barchip Shogun	0,50 %	Normal	LB28	31.03.09	Aug 09	
	10B	Barchip Shogun	0,50 %	Normal	LB29	31.03.09	Aug 09	
	7A	Dramix 65/35	0,50 %	Normal	LB22	30.03.09	Aug 09	

3 Materials

3.1 Fibres

In general were six macro fibre types used, five steel fibre types, and one synthetic fibre. In addition were synthetic microfibres of the type Propex Fibermesh 150 added in some of the mixes. The fibres and their properties are presented in table 6 where the different values are taken from the suppliers' data sheets.

Table 6 – Fibre types used in the test programme

Fibre		**Material**	**Shape**	**l_f [mm]**	**d_f [mm]**	**l_f/d_f [-]**	**E [GPa]**	**Density [kg/m^3]**	**f_{ft} [MPa]**
Macro	**Dramix 65/60**	Steel	Smooth with end hooks	60	0,90	67	200	7800	1000
	Dramix 65/35	Steel	Smooth with end hooks	35	0,55	64	200	7800	1100
	Novocon URW 1050	Steel	Wave shaped	50	1,00	50	200	7800	1100
	Novocon HE 1050	Steel	Smooth with end hooks	50	1,00	50	200	7800	1050
	Novocon FE 1050	Steel	Smooth with flat end anchor	50	1,00	50	200	7800	1050
	Barchip Shogun	Polyolefin	Straight with embossed surface	48	0,90	53	10	900-920	550
Micro	**Propex Fibermesh 150**	Polyolefin	Smooth and straight	graded	< 0,30			910	

3.2 Concrete

The tests have been carried out with a LWAC developed recently by maxit Group. This is a concrete containing Leca aggregate and foam to achieve stability and sufficiently low density. The self weight is within the range 11-1200 kg/m3, and the desired compressive cylinder strength is 20 N/mm^2 to achieve strength class LB15.

The foam content is confidential, but rather stable with a self weight of 60-80 g per litre. The foam's behaviour and appearance can be compared to shaving foam. It is produced in a generator by mixing two liquids. When producing small volumes at SINTEF & NTNU the generator was replaced by a Kenwood mixmaster. The amount of added foam is based on experience and precisely determined by density measurements in the fresh state.

In addition to the LWAC an ordinary concrete with quality B35 were used in some of the pullout tests.

Figure12 – Foam being added

3.2.1 Proportioning

The concrete was initially produced according to a receipt developed by maxit in the autumn 2008. This receipt was used for the concrete cast at NTNU the 4/2 and 6/3, but in the production at Lillestrøm a rotational mixer giving false air and separation problems, was used. It was therefore necessary to modify the concrete composition, and therefore four different receipts have been used. The development was due to changes in additives (stabilizer and defoaming agent), increase of the sand/Leca-relation, and finally addition of the synthetic microfibres which was the final decisive factor to achieve a stable concrete. The receipts are summarized in table 7. Furthermore it should be noted that the development of the concrete and the production of the full scale beams were chaired by maxit group.

3.2.2 Concrete production

The test specimens were cast in five series where two were made at maxit in Lillestrøm and three in the concrete laboratory at NTNU. Comments and experience due to the different casting series are summarized in Enclosure 1. The coupling between receipt, time and place is shown in table 8.

Table 7 - Receipts

Receipt	**M1**	**M2**	**M3**	**M4**	**B35**
w/c	0,434	0,484	0,487	0,474	0,548
Unit	kg/m	kg/m	kg/m	kg/m³	kg/m
Norcem STD FA (cement type)					368,9
Norcem Anlegg (cement type)	266,0	295,6	295,6	267,5	
Norcem Industri (cement type)	114,0	126,7	124,4	114,6	
Silica fume	35,0	35,6	35,6	35,2	22,1
Sand 0-4 mm	216,3	215,6	322,2	402,2	
Leca 2-4 mm	324,5	322,2	271,1	231,3	
Gravel Årdal 0-8mm					966,7
Gravel Årdal 8-16mm					644,5
Glenium Sky 542 (SP)				7,2	
Gelnium sky 552 (SP)	4,6				
Glenium ACE 30 (SP)		3,5	3,5		
Glenium 151 (SP)					3,7
Glenium Stream (stabilisator)				2	
Rheomatrix 101 (stabilisator)		1	1		
Defoaming agent				0,1	
Water	165	204,4	204,4	181	202,07
Foam	slump	slump	slump	slump	
Propex Fibermesh 150				1,5	

. Table 8: Casting dates, -place and receipts

Casting dates	**Place**	**Receipt**
4.february	M-lab NTNU	M1
26.february	maxit, Lillestrøm	M2
27.february	maxit, Lillestrøm	M3
16.march	M-lab NTNU	M1
30-31.march	maxit, Lillestrøm	M4
16.march	M-lab NTNU	B35

4 Results

4.1 Compressive Strength Testing

Most of the cylinders had air bubbles at the surface, and possible irregularities at the top- and bottom surfaces were removed by grinding. The results presented in figure 13 (receipt M1-M3) and 14 (receipt M4) show that the strength varies a lot within the same receipt. Especially this holds for receipt M3. In addition it is an important point that the strength for most of the mixes are below the planned value of 20 MPa.

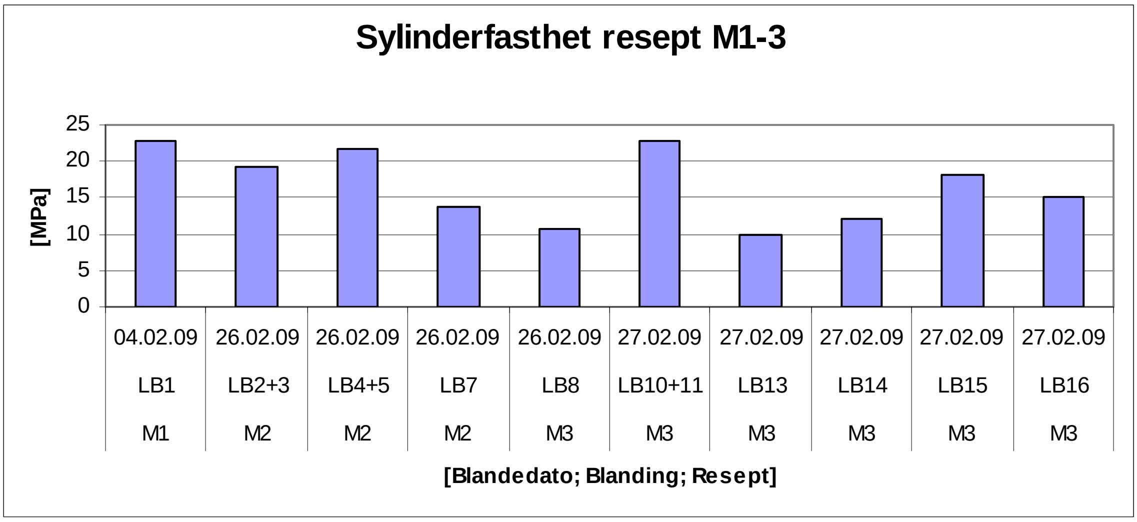


Figure 13 – Cylinder strength for the receipts M1-M3

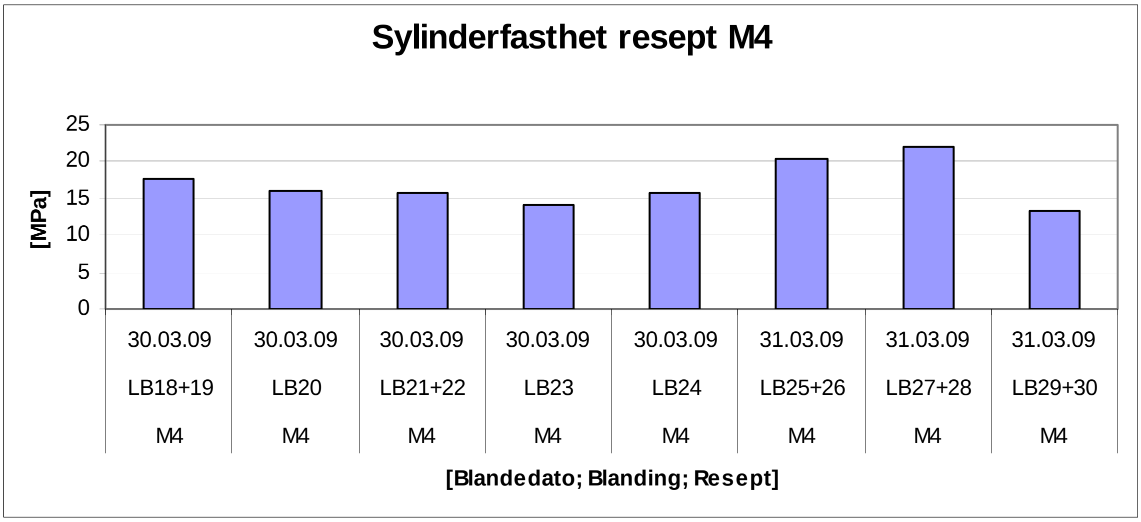


Figure 14 –Cylinder strength for the receipt M4

4.2 Pullout of Single Fibres

The results from the comparison of the different fibre types is shown in figure 15 (L=50-60mm). Each curve is the average of 5 tests. For full anchor length, i.e. equal to half fibre length, it is seen that Novocon URW (wave shaped fibre) has clearly the largest pullout capacity. For anchor length $l_f/6$, however, the same fibre type is the one with lowest capacity, and consequently the average capacity contributions for this fibre and the traditional Dramix-fibre with end hooks are approximately similar. Novocon FE (with flat end hooks) has somewhat lower pullout capacity than the two firstly mentioned fibre types. However, it should also be noted that the Novocon ffibres have L=50mm, while the Dramix fibres have L=60mm.

Compared to the results from the ordinary concrete, the pullout tests with the LWAC show surprisingly large capacity. For further details concerning the test description and the results, see e g [Aamodt 2009].

Further is the effect of added microfibres (0,3% fibremesh) investigated for Dramix 65/35 and 65/60 with $l_f/2$ and $l_f/6$ as anchor length. The results for the shortest fibre (35 mm) are shown in figure 16, where it is a clear effect of microfibres both on maximum pullout force and ductility. For instance increases the average maximum force by 16%, while the pullout force (expressed by fibre stress) at 2 mm displacement increases from approximately 70 N/mm^2 to 250 N/mm^2. Corresponding tests with anchor length $l_f/2$ show also a clear positive effect of added fibremesh, but not as much as for $l_f/6$.

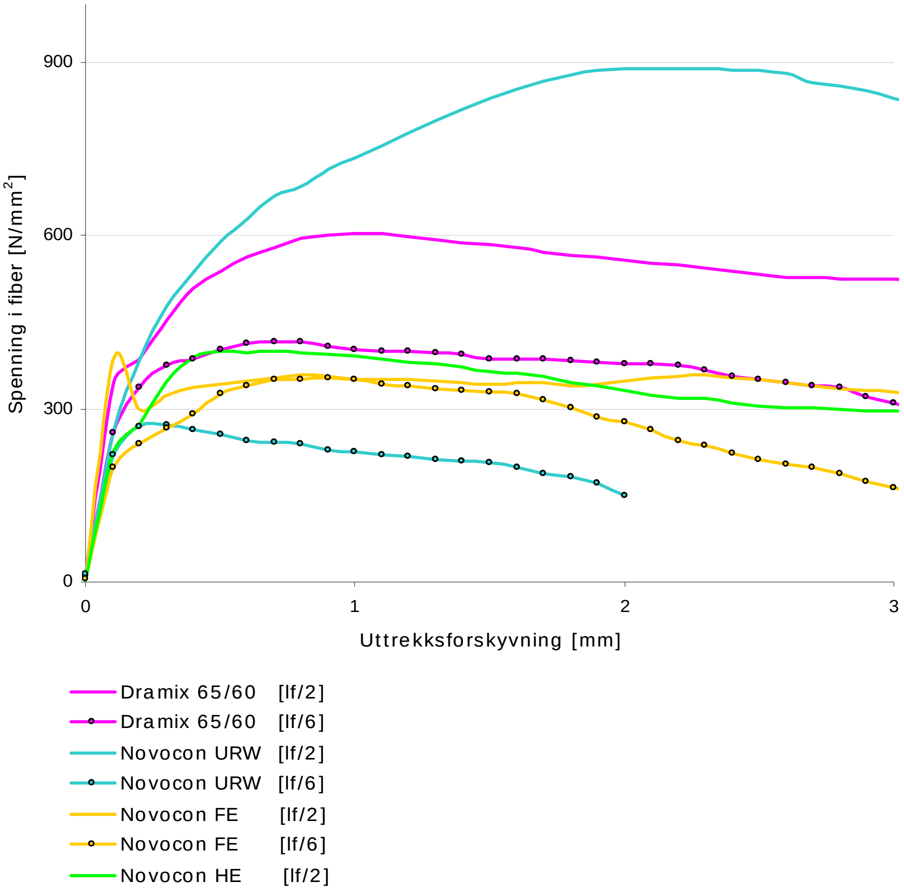


Figure 15. Results from pullout tests with various fibre types and two different anchorage lengths. Note that for the Dramix-fibres: L=60, while for Novocon fibres: L=50.

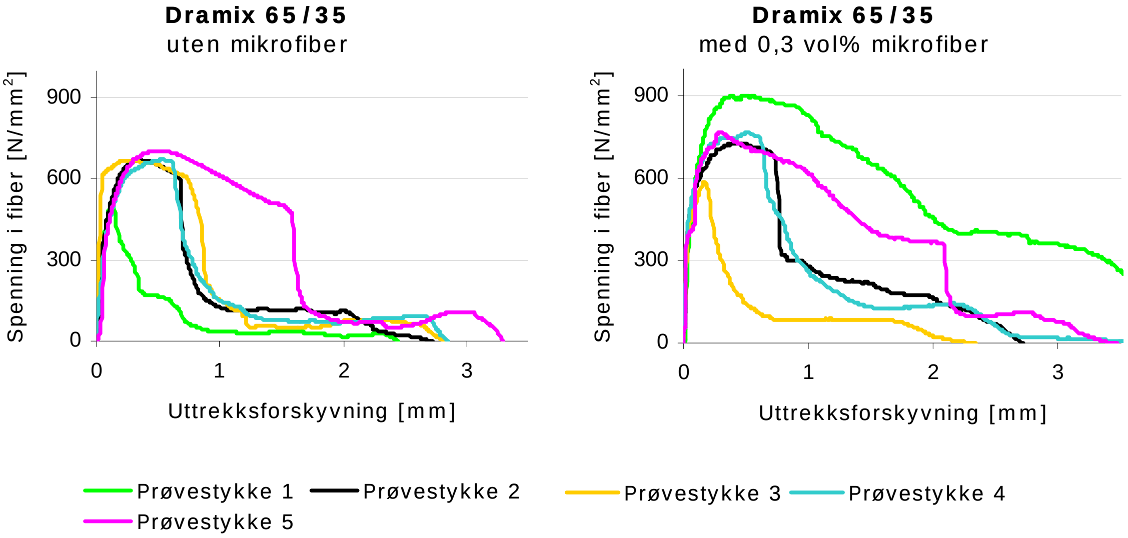


Figure 16. Stress – pullout displacement for Dramix 65/35 in LWAC, anchorage length $l_f/6$.
a) Without microfibres, b) with 0,3% microfibres (fibremesh).

4.3 Uniaxial Tensile Testing

The results from these test series are described by [Strandgård 2008] and [Engesæter 2008]. In this report are only some selected results and a relatively short description and discussion of these included. In general the scatter is large also for this test series which reduces the utility value of the results, for instance does the uniaxial tensile strength vary between 1,0 and 3,7 N/mm^2. The reasons for this are discussed previously in chapter 3. The lowest values were achieved for the tests with 2,5% Barchip fibres cast in the first series at Lillestrøm, while the highest were achieved for 2,5% Dramix fibres mixture cast at NTNU. However, the fibre effect is mainly interesting only after cracking and therefore we concentrate on this phase, and only refer to the master student reports for further information on the strength data.

Figures 17a-f show a selection of the test results. Firstly figure 17 a and b show load-displacement curves for beams with 1% and 2,5% Barchip Shogun synthetic fibres. If maximum load is compared, one can see it is highest for 1% fibre content, which probably is due circumstances around the concrete production and casting because the two concrete types are from different casting series. However, it can be seen that the ductility is considerably better for 2,5% fibres than for 1,0%, and that the load displacement relation indicates that the minimum reinforcement criterion in principle seems to be fulfilled for 2,5%. Furthermore, it can also be seen from figure 17 b that the ductility for specimens stored under realistic climatic conditions (RH=50%, T=20^{o}C) is considerably poorer than for specimens stored under wet conditions.

In the figures 17 c and d can the effect of 1,0% Novocon URW (wave shaped 50mm) and 2,5% Dramix 65/60 be compared. The previously described results from the pullout testing indicated that these two fibre types are comparable, and therefore can this comparison be considered as an effect of the fibre content. The effect on the tensile strength is large, but it is surprising that increased fibre content seems to give poorer ductility. This can be explained by that this concrete does not match as high fibre content as 2,5% when the cracking reaches a certain level, i.e. that collective anchorage of the fibres is too low after cracking.

Figure 17 e and f present results for the prisms with 0,27% bar reinforcement (1 Ø6), and either 1,0% steel- or synthetic fibres. It can be seen that both the capacity and the ductility is considerably better for the steel fibre concrete than for the concrete with synthetic fibres.

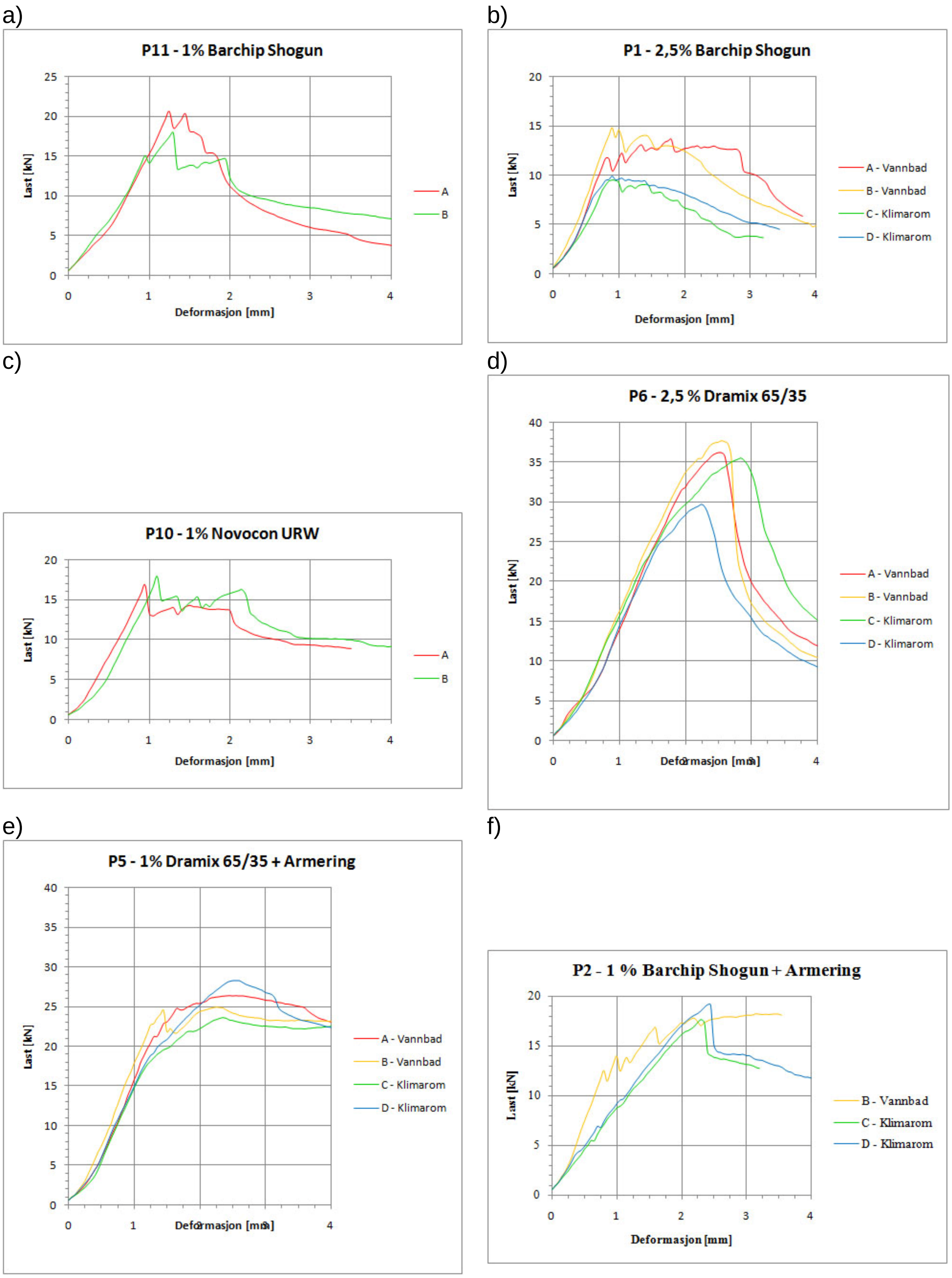


Figure 17. Selected test results for the uniaxial tensile tests. (a) 1% Barchip Shogun, (b) 2,5% Barchip Shogun, (c) 1% Novocon URW, (d) 2,5% Dramix 65/35, (e) 1,0% Dramix 65/35 and 0,27% bar reinforcement (1Ø6), (f) 1% Barchip Shogun and 0,27% bar reinforcement (1Ø6).

4.4 Standard Test Beams (150/150/600)

4.4.1 Visual evaluation of the test specimens

The first series cast at maxit Lillestrøm was strongly influenced by separation of the fresh concrete. There was a clear division in layers in some of the beams, which had to be skipped. Casting of the second series was satisfactory and the beams had no visual damage.

At the surface of the beams stored in climatic rooms, comprehensive shrinkage cracking occurred, and in the picture in figure 18 the typical crack pattern is marked by blue marker at the bottom face of three beams (beam 8 D-F). We can see the fracture from the bending test following the existing shrinkage cracks. The beams which were stored under isolated conditions showed essentially no visible shrinkage cracks.

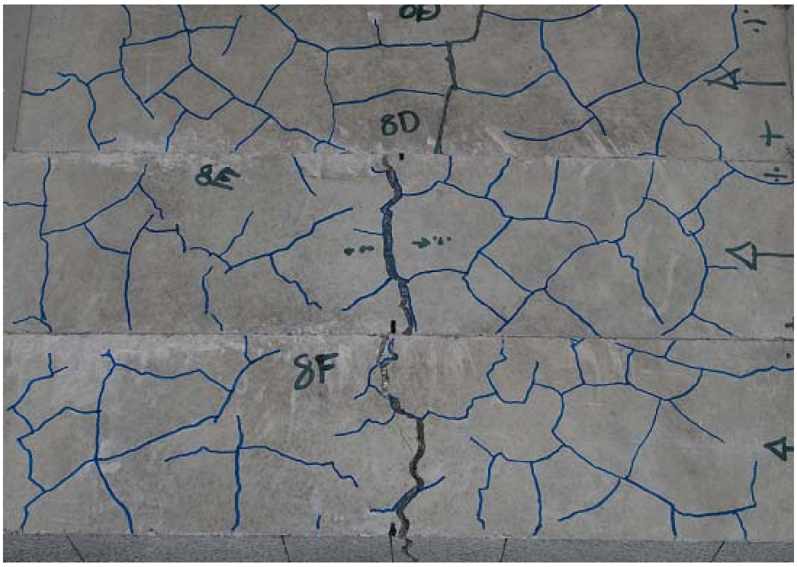


Figure 18 – Crack pattern in Standard test beams

4.4.2 Expected Residual Tensile Strength

In advance the equivalent residual bending strength was calculated according to the Norwegian guidelines [Thorenfeldt et. al, 2006], based on previous experience and the results from the pullout testing. The residual strength calculated from the pullout test results gave not surprisingly considerably lower values than reference data from normal strength concretes. The variable concrete quality can influence the evaluation of the standard beam tests based on the pullout testing. The compressive strength of the concrete used in the pullout tests (cast at NTNU) is however generally higher than the strength of the concrete used in the standard beams (maxit, Lillestrøm).

4.4.3 Results from the Testing

From the standard beam test results, the values of the residual bending strength, $f_{ftk,eq}$, are calculated according to the procedure described in the Norwegian guidelines [Thorenfeldt et. al, 2006]. The values are reported in table 9, and the following equation has been used:

$$f_{ft,eq} = F_{(\delta 12)} \frac{L}{bd^2}$$

$F_{(\delta 12)}$ = The mean value of the load in the prescribed deflection range: 0,5 - 2,5mm

In general one can see that residual bending strength calculated from the beam tests is between the two others discussed in the previous section ($f_{ftk,eq}^3 < f_{ftk,eq}^1 < f_{ftk,eq}^2$ referred to the table). A relatively large variation between the beams from the same slab is also seen. This can be due to the fibre distribution and the orientation. For the beams from series 2, fibre counting has been carried out, and the results are presented and discussed later in the report. For series 2 with isolated storage conditions, one can see that 0,5% hooked end steel fibres results in larger residual tensile strength than 1,0% of the same hooked end fibres. The reasons for this abnormal situation are due to unsatisfactory fibre distribution in the fresh state, and is probably also due to low tensile and bond strength of the actual LWAC.

Table 9 – Results from the standard beam testing

	Beam notation	**Fibre volume**	**Hardening conditions**	$f_{ftk,eq}^{1}$		$f_{ftk,eq}^{2}$	$f_{ftk,eq}^{3}$	f_{cck}	
				Beam	**Mean**				
				[MPa]	**[MPa]**	**[MPa]**	**[MPa]**	**[MPa]**	**Receipt**
Series 1	B3A	0,5% Dramix 65/35	Isolated	1,20	1,66	2,25	0,87	9,8	M3
	B3B			1,45					
	B3C			2,33					
	B3D	0,5% Dramix 65/35	Climaroom	1,86	1,43	2,25	0,87		
	B3E			0,87					
	B3F			1,56					
	B5A	0,5% Barchip Shogun	Isolated	2,07	1,65	1,13	0,53	18,1	
	B5B			1,21					
	B5C			1,68					
	B5D	0,5% Barchip Shogun	Climaroom	1,13	1,14	1,13	0,53		
	B5E			0,95					
	B5F			1,35					
	B6A	1,0% Barchip Shogun	Isolated	1,31	1,45	2,25	1,06	15,0	
	B6B			1,25					
	B6C			1,79					
	B6D	1,0% Barchip Shogun	Climaroom	1,16	1,52	2,25	1,06		
	B6E			1,10					
	B6F			2,30					
Series 2	B8B	0,5% Dramix 65/35	Isolated	2,07	2,38	2,25	0,87	14,2	M4
	B8A			2,29					
	B8C			2,78					
	B8E	0,5% Dramix 65/35	Climaroom	0,50	1,15	2,25	0,87		
	B8D			1,06					
	B8F			1,90					
	B9A	1,0% Dramix 65/35	Isolated	2,26	2,26	4,50	1,74	14,2	
	B9B			2,35					
	B9C			2,16					
	B9E	1,0% Dramix 65/35	Climaroom	1,47	1,67	4,50	1,74		
	B9D			1,74					
	B9F			1,79					
	B10B	0,5% Novocon URW	Isolated	1,52	1,82	2,25	1,05	15,85	
	B10C			1,81					
	B10A			2,14					
	B10D	0,5% Novocon URW	Climaroom	1,76	1,94	2,25	1,05		
	B10F			1,75					
	B10E			2,32					
	B12B	1,0% Barchip Shogun	Isolated	1,81	2,06	2,25	1,06	13,43	
	B12A			2,14					
	B12C			2,24					
	B12F	1,0% Barchip Shogun	Climaroom	1,79	1,88	2,25	1,06		
	B12E			1,82					
	B12D			2,02					

$f_{ftk,eq}^{1}$ = Determined from standard beam testing on the actual LWAC
$f_{ftk,eq}^{2}$ = Calculated theoretically based on data from normal strength concrete, $\eta_0 = 1/3$
$f_{ftk,eq}^{3}$ = Calculated from pullout test results on the actual LWAC, $\eta_0 = 1/3$

4.4.4 Structural Behaviour during the Tests

In most of the beams one major crack developed towards failure. In the beams stored under climatic conditions, the failure developed most often in an existing shrinkage crack, while within the isolated beams it seemed more arbitrary where the failure crack developed. The first crack could be recorded by a fall in the recorded force. After the fall, the force usually increased to a maximum load which was higher then the cracking load. The load-deformation relations (average of three beams) for each beam type in series 2 are shown in figure 19 below.

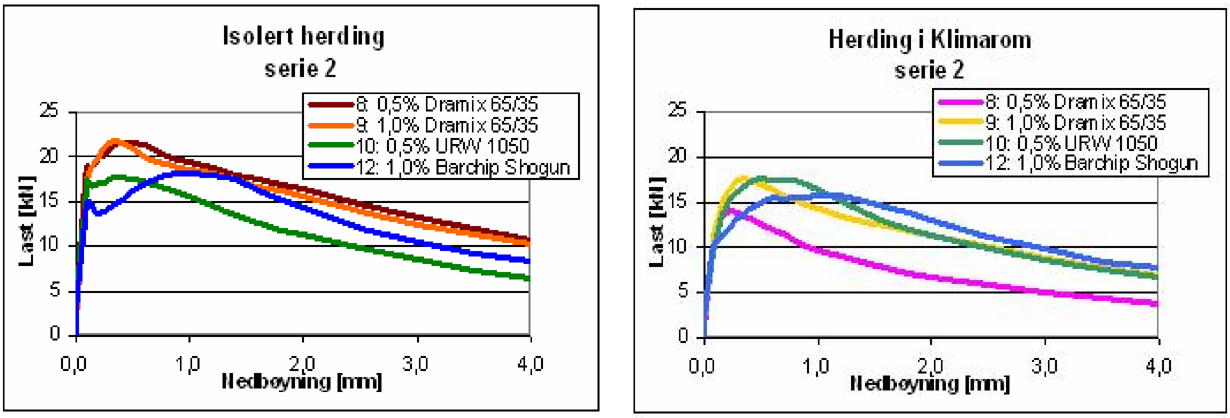


Figure 19 – Results from standard beam testing, series 2 (each curve represents 3 beams)

4.4.5 Fibre Counting

The beams in the second test series were cut, in the failure section, into two pieces after the testing, so that the fibres crossing the crack could be counted. This was done mainly to determine the correspondence between the real and the theoretical fibre distribution, and to investigate if possible variations in the fibre distribution could explain the variations in capacity.

The relation between equivalent residual bending strengths calculated from the test result, and the fibre area crossing the failure surface in beams with the same hardening conditions and the same nominal (planned) fibre content is shown in figure 20.

For fibre volume 0,5 %, the results in the figure indicates that URW and Dramix have approximately equal efficiency per fibre area, and thereby also per fibre volume given the same distribution and orientation. In addition it seems that steel fibres are approximately two times as efficient as synthetic fibres. The results for 1,0 volume % steel fibre show clearly that the increase in fibre content from 0,5 to 1,0 % not gives correspondingly increased residual tensile strength.

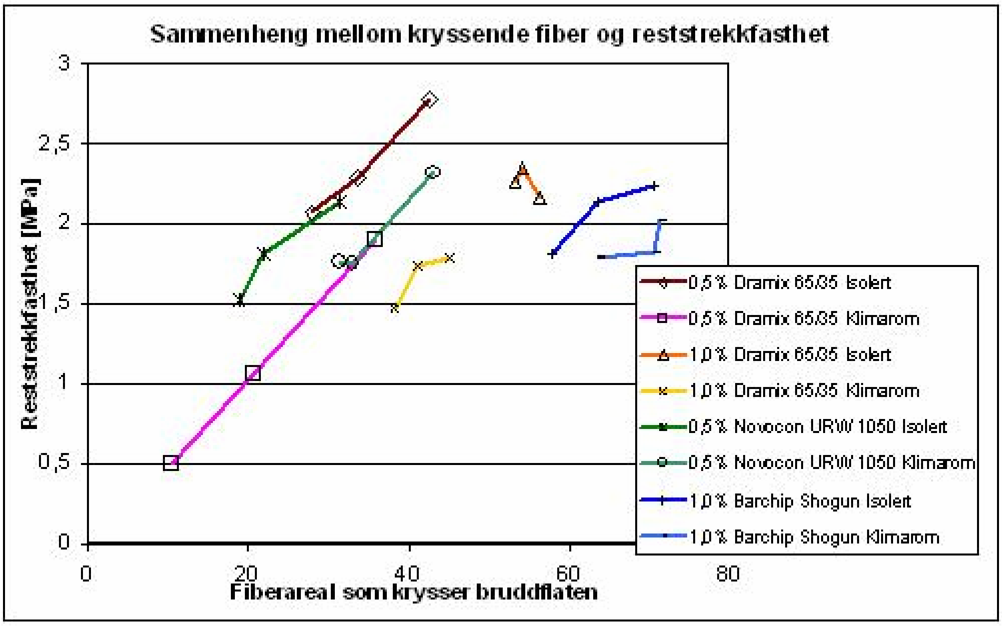


Figure 20- Relation between fibre area crossing the failure surface and the equivalent bending tensile strength

4.5 Fullscale Prefabricated Beams

4.5.1 General

The results from the compressive strength testing, presented previously in figure 13 and 14, showed relatively large variation, and in addition there were some problems with the casting due to the relatively dense reinforcement grid.

Series 1 was influenced by separation of the concrete. For all the beams there were too small amount of binder in the tensile zone, and some of the beams had even openings into the reinforcement at some points. The series was in general firstly skipped, and it was decided to make a new series. However, some of the beams were anyhow tested to evaluate the test rig and gain experience before series 2. These tests were conducted without deformation measurements, and only the failure load was reported.

The concrete in series 2 was in general satisfactory. There were a certain amount of air pores at the surface, but that is very difficult to avoid in a concrete with around 15% air content. All the beams had some shrinkage cracks before testing as discussed in a later section.

Figure 21 – Beam with separating concrete in the flange

4.5.1 Results which are relevant for the SLS

4.5.1.1 Crack Development

The crack development during the tests was quite similar for all the beams, and there were new vertical cracks in the lower part of the web at relatively low load levels as shown in figure 22a. These increased in length and width, in addition to formation of new cracks at increasing load. From 10 kN and upwards some of the vertical cracks started to direct towards the load application points in diagonal direction (figure 22b), in addition to that new diagonal cracks appeared. These cracks were first observed on the embossed side of the web (figure 22 c). From 20 kN and upwards gradually more and larger cracks developed, until the beams failed, in most cases as compressive failure below one of the loading points (figure 22 d). Before and after testing, the number of cracks was counted in the flange and in the web.

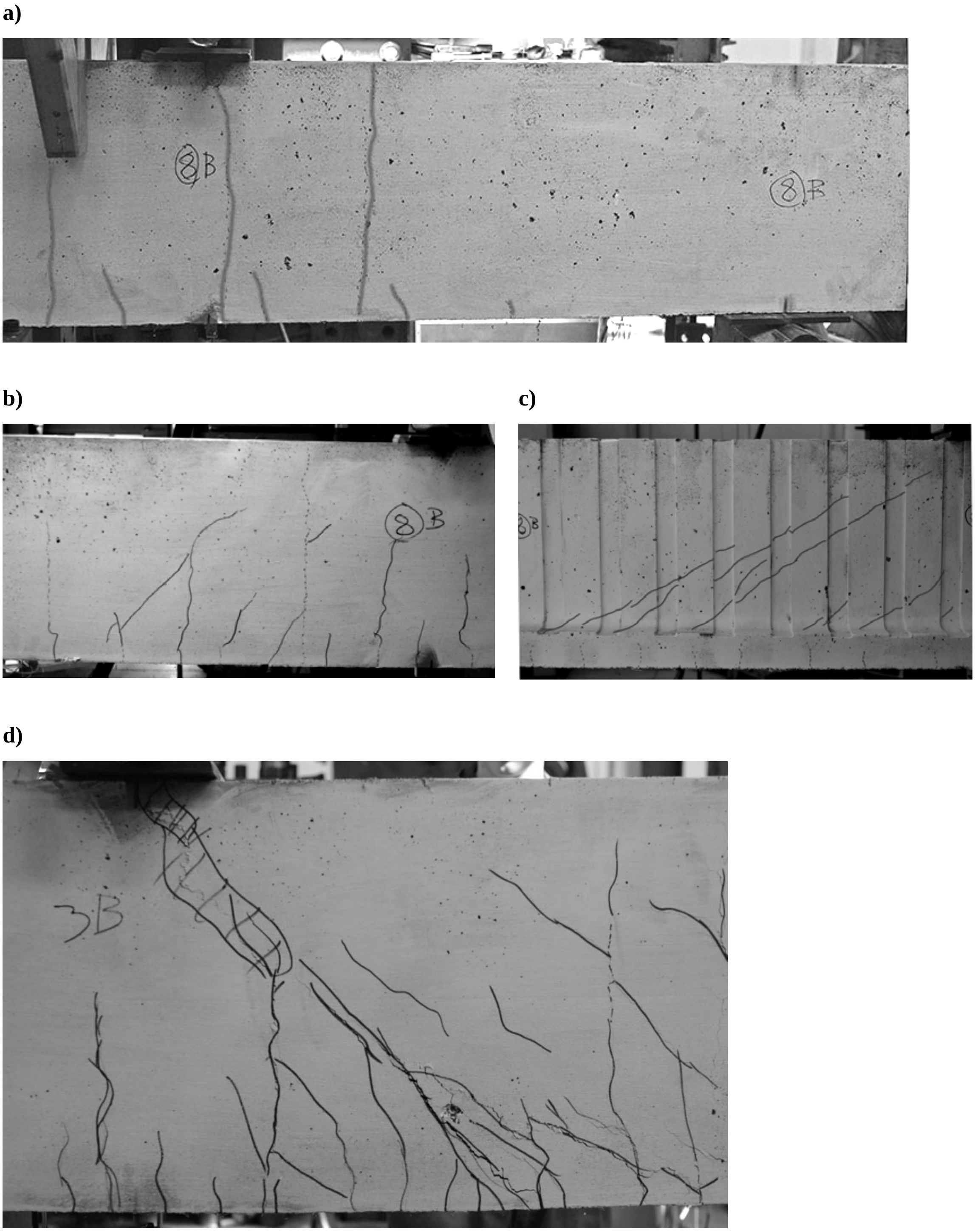


Figure 22. Crack pattern, (a) Early in the load test, (b) At approximately15 kN, (c) Diagonal cracking at the side with embossed surface pattern, (d) Crack pattern at failure.

4.5.1.2 Crack distribution

Considering the crack pattern appearing before the load test, it was expected that the degree of shrinkage cracking (number and crack width) would be reduced with fibre content. Even if the crack width not was measured, are both conditions fulfilled, for instance was the average number of shrinkage cracks in the beams without fibres (in the web) 12,5, while it was 11 in the beams with synthetic fibres, 7,5 for wave formed steel fibres and 6,5 for steel fibres with end hooks.

After the load test the situation is the opposite, also as expected since it is generally accepted that better crack distribution is achieved by fibre addition. After the test the average number of cracks (in the webs) were 44 in the beams without fibres and 45, 49, and 52 in the beams with wave shaped steel fibres, with synthetic macro fibres and steel fibres with end hooks, respectively.

Pictures of the beams in series 3, taken after the test, shown in figure 26 illustrate the crack pattern.

4.5.1.3 Deformations

The measured deformations for the service load level are larger than calculated according to the naked stage II theory without fibre contribution, and it is not possible to sort out the fibre effect as shown in figure 23. At this load level the average measured deformation is 8 mm, while the calculated deformation is 4,2 mm. This is surprising because measured deformations usually are smaller than the theoretical naked stage II values because the latter neglect the stiffness contribution from the concrete between the cracks (”tension stiffening”).

After some evaluation, calibration of measurement instruments and control of the managing computer program used in the load test, it can be concluded that the large measured deformation mainly is caused by the effect of the shrinkage cracks in the compressive zone. For example is the effect of closing cracks with a width of 0,1 mm and 200 mm spacing large enough to explain the deviation between experiment and the theory. Figure 24 below shows measured and calculated compressive strain development at the top of the beam for one of the tests. In this case the agreement is reasonably good, and because this measurement represents the concrete between the cracks, the results in this figure support the hypothesis presented above.

For further consideration of the behaviour relevant for the SLS, the measured deflection of the standard beams in the uncracked stage was compared to calculated values The relatively small size of the numbers taken into consideration, the agreement is reasonably good between theory and experiment for this case. In average the theoretical values are somewhat larger than the measured ones, and the deviations are therefore in opposite direction of the stage II deviations for the large beams.

The beams in series 3 has in average 3,3 mm deflection at service load, and this value is less than calculated with naked stage II theory for the net LWAC beam (4,2mm).

The general conclusion is therefore that the calculation basis for the SLS is in satisfactory agreement with the test results, even if models taking the fibre contribution into account not are considered.

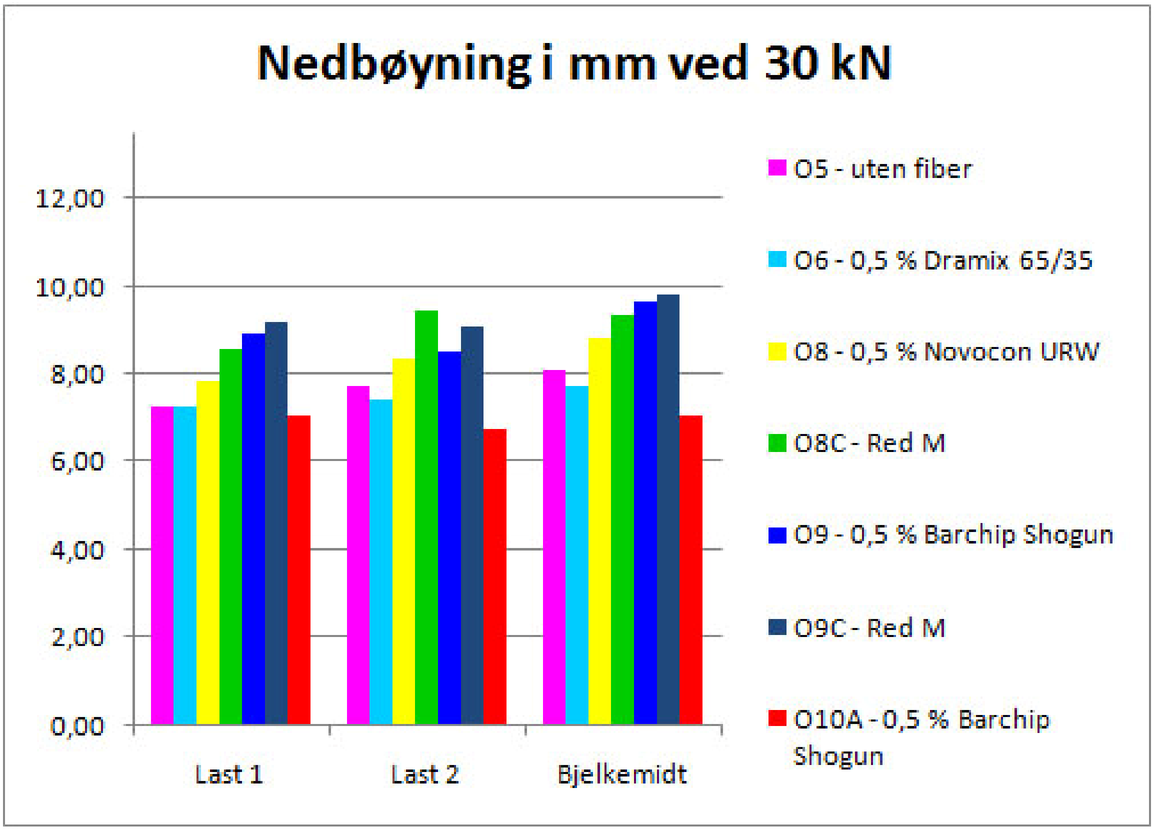


Figure 23. Deflection of the different full scale beams at SLS load level (50% of failure load). Calculated deformation with naked stage-II stiffness (cracked section) is 4,2 mm.

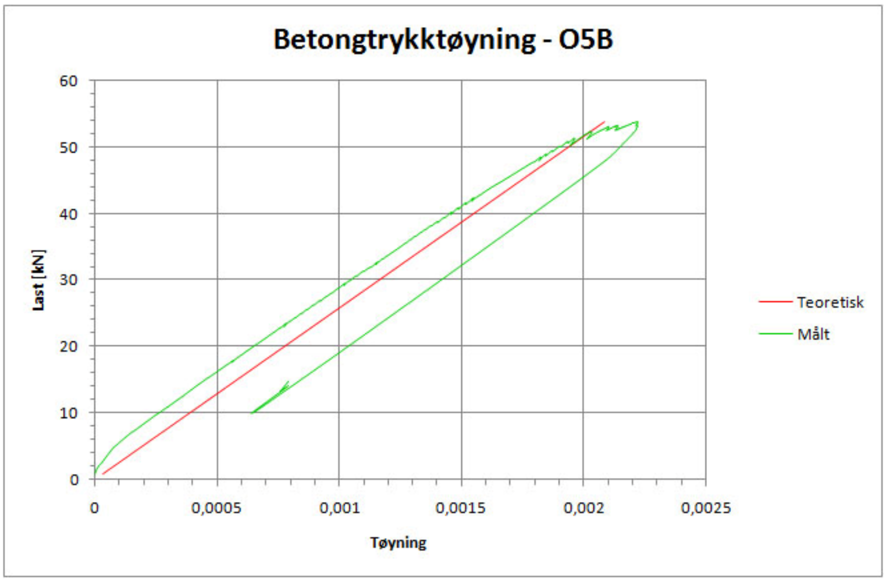


Figure 24 – Theoretical (naked stage II theory) vs. measured compressive concrete strain for a beam without fibres

4.5.3 Results relevant for the ULS

4.5.1.4 Experimental vs calculated capacity

Series 1

Maximum load, corresponding moment and shear force, and failure type are presented in table 10 and 11 which thereby summarizes the most relevant test results for the ultimate limit states (ULS).

The results from the tests in series 1 presented in table 10, show that the failure load varies a lot, and that it is below the design capacity for the beams without fibres and the beams with synthetic fibres. Even if these results not should be given too much confidence, (because of the poor quality of the concrete) it seems that steel fibres increase the strength considerably. Precalculated capacity without fibres is 58 kN, and we may see that the beams with Dramix 65/35 in average have sufficient capacity (58,3 with 0,5% fibres and 60,7 with 1,0% fibres). The other beams do not have sufficient capacity. For the Barchip synthetic fibre concrete it should be noted that the compressive strength is rather low (10,8 and 13,7 MPa).

Series 2

The results from series 2 confirm the results from series 1, adding 0,5% Dramix gives increased capacity both for the original reinforcement solution, for the beams with 50% shear reinforcement (reduced shear), and for the beams with reduced amount of tensile reinforcement (reduced moment). It is also still clear that the Dramix 65/35 steel fibres give larger capacity contribution than the other fibre types. As for series 1, the beams with 0,5% Dramix 65/35 fibres have sufficient capacity compared to calculated values, while this is not the case for the other beams.

It seems that the safety margin is too small, and that the calculation methods might be uncertain because the calculated capacity does not take the fibres into account. The test conditions might however explain some of this, because the slenderness and eccentricity of the beams resulted into sideways deflection of the beam in the test rig. The beams were braced, but it was difficult to establish sufficiently stiff sideways support, because vertical frictional forces than would be rather large. The real capacity is therefore probably somewhat higher than the experimental values

Table 10 - Results for full scale beams series 1

Beam	**Fibres**		**Rein-forcement**	**Compr. strength**	**F_{max}**		**Failure type/ capacity**
	Type	**V_f**			**beam**	**mean**	
		[%]		**[MPa]**	**[kN]**	**[kN]**	
O1A	Dramix 65/35	0,5	Normal	19,3	51,2	58,3	Moment-compr failure M = 32,4 kNm (V_{max}=29,2kN)
O1B				19,3	58,0		
O1C	Dramix 65/35	0,5	Normal	19,3	60,6		
O1D				19,3	63,3		
O2A	Ingen	0,0	Normal	21,7	43,4	43,7	Shear-Anchorage V=21,9 kN (M_{max}=24,3)
O2B				21,7	44,1		
O3A	Barchip	1,0	Normal	13,7	43,9	46,3	Moment-compr failure M = 25,7 kNm (V_{max}=23,2kN)
O3B				13,7	49,4		
O3C	Barchip	1,0	Normal	10,8	47,5		
O3D				10,8	44,5		
O4C	Dramix 65/35	1,0	Normal	22,9	60,7	60,7	Moment-compr failure M = 33,7 kNm (V_{max}=30,4kN)

Series 3

Table 12 presents in the same way the results for series 3 – beams with additional cast-in-place concrete. All these beams achieved considerably higher capacity than the net LWAC beams, and the behavior shows that sufficient composite action between the two materials is achieved. Five such beams were tested, and the ratio between experimental and precalculated capacity was 1,41 for the beams without fibres, 1,27 for the beams with synthetic fibres and 1,73 for the beams with steel fibres. This safety margin comes in addition to the margin given by the load and material coefficients.

Table 11 - Results for full scale beams series 2

Beam	Fibre Type	 V_f	Reinforcement	Compr strength	F_{max} beam	 mean	Failure type/ capacity
		[%]		[MPa]	[kN]	[kN]	
O5A	No	0,0	Red. Shear	17,8	54,1	54,1	Shear-Anchorage V=27,1 kN (M_{max}=30,1)
O5B	No	0,0	Normal	17,8	53,8	53,8	Moment-compr failure M = 29,9 kNm (V_{max}=26,9kN)
O6A	Dramix 65/35	0,5	Red. Shear	16,0	58,8	59,5	Moment-compr failure M = 33,1 kNm (V_{max}=29,8kN)
O6B				16,0	60,2		
O6C	Dramix 65/35	0,5	Normal	16,0	58,3	60,4	Moment-compr failure M = 34,7 kNm (V_{max}=31,3kN)
O6D				16,0	62,5		
O8A	Novocon URW	0,5	Red. Shear	20,5	58,2	56,7	Moment-compr failure M = 31,5 kNm (V_{max}=28,3kN)
O8B				20,5	55,3		
O8C	Novocon URW	0,5	Red. Mom	20,5	47,9	47,9	Moment-compr failure M = 26,6 kNm (V_{max}=24,0kN)
O8D	Novocon URW	0,5	Normal	20,5	54,3	54,3	Moment-compr failure M = 30,2 kNm (V_{max}=27,2kN)
O9A	Barchip Shogun	0,5	Red. Shear	22,0	53,2	54,2	Moment-compr failure M = 30,1 kNm (V_{max}=27,1kN)
O9B				22,0	55,1		
O9C	Barchip Shogun	0,5	Red. Mom	22.0		51,0	Moment-compr failure M = 28,3 kNm (V_{max}=25,5kN)
O10A	Barchip Shogun	0,5	Normal			56,4	Moment-compr failure M = 31,3 kNm (V_{max}=28,2kN)

Table 12 - Results for full scale beams series 3 (with ordinary concrete added)

beam	Fibre		Reinforcement	Compr strength	F_{max}		Failure type/ capacity
	Type	V_f			beam	mean	
		[%]		[MPa]	[kN]	[kN]	
5C	-	-	Normal	17,8	81,4	82,0	M_{max} = 45,6kNm V_{max}=41,0kN
5D	-	-	Normal		82,5		
9D	Barchip Shogun	0,50 %	Normal	22,0	76,5	73,8	M_{max} = 41,0 kNm V_{max}=36,9kN
10B				13,4	71,0		
7A	Dramix 65/35	0,50 %	Normal	15,9	100,5	100,5	M_{max} = 55,8 kNm V_{max}=50,3 kN

4.5.1.5 Discussion of the ULS-results

Net LWAC-beams (Series 1 and 2)

In advance the moment capacity was calculated to 26,1 kNm (total load 58 kN for the present test rig), while the capacities for shear tensile failure and shear compressive failure were calculated to 50,1 kN (total load100,2 kN) and 38,6 kN (77,2 kN) respectively. Ideally the moment capacity is decisive, but introductory tests at maxit gave anchorage initiated shear failure, and this together with a general experience based insecurity due to brittle behavior and shear capacity of such an extreme LWAC made it necessary to also plan for shear failure.

In the beams without fibre reinforcement the failure started, as it did at the maxit-tests, as an anchorage failure for the tensile reinforcement near the support and developed further as shear failure. All fibre types improved the capacity against bond failure considerably so that this failure type did not occur in the fibre reinforced beams. The final failure was then instead a moment compressive failure under one of the loads which happened after comprehensive diagonal cracking. The beams with 35 mm end hooked steel fibres gave generally highest capacity. The concretes compressive strength is lower than presumed in the test planning, and the strength variation is so large that it is difficult to see the effects of the other fibre types.

Based on the experience from series 1, test series 2 was expanded to include some beams with reduced shear reinforcement (c/c 200 instead of c/c 100), and some with reduced tensile reinforcement (1Ø12 instead of 2).

The 50% reduction of the shear reinforcement had no significant effect on the capacity. For the end hooked fibres it lead to 1,5% reduction in the capacity, while for the wave formed fibre it lead to 5% increase, and finally for the synthetic fibre to 4,5 % reduction. This simply means that the shear capacity is not a not is limiting factor in these beams, as long as they are fibre reinforced, and that the amount of shear reinforcement can be halved without reduced.

The effect of halving the tensile reinforcement is not large either. For the wave formed steel fibre the result was a capacity reduction of 11,5%, while for the synthetic fibre the reduction was 10%. This means that the cross section with the original reinforcement is in the over reinforced range, and that it is the compressive that is decisive for the moment capacity. This result is therefore not directly relevant for the composite cross section with additional concrete since the compressive zone then is strengthened.

Beams with ordinary concrete added (Series 3)

The failure mechanism is the same for all the beams in this series. Comprehensive diagonal cracking in the LWAC occur, and a relatively dense crack pattern because of the shear reinforcement. The additional concrete cracks in considerably less degree, but gradually a dominating diagonal crack towards the loading point is developed. This diagonal crack starts down with the flange in the horizontal transition zone between the two materials, and it is therefore assumed that the embossed surface is important for the failure behavior and the capacity. When the crack development gets more comprehensive the additional concrete loosens from the LWAC beam and the composite action gets poorer, and the beam reaches its maximum load and fails. The failure can be described as a shear failure due to the two materials different crack pattern.

It is probable that the more distributed crack pattern in the steel fibre reinforced beam is favourable, and makes the composite action work till a higher load level than for the other beams.

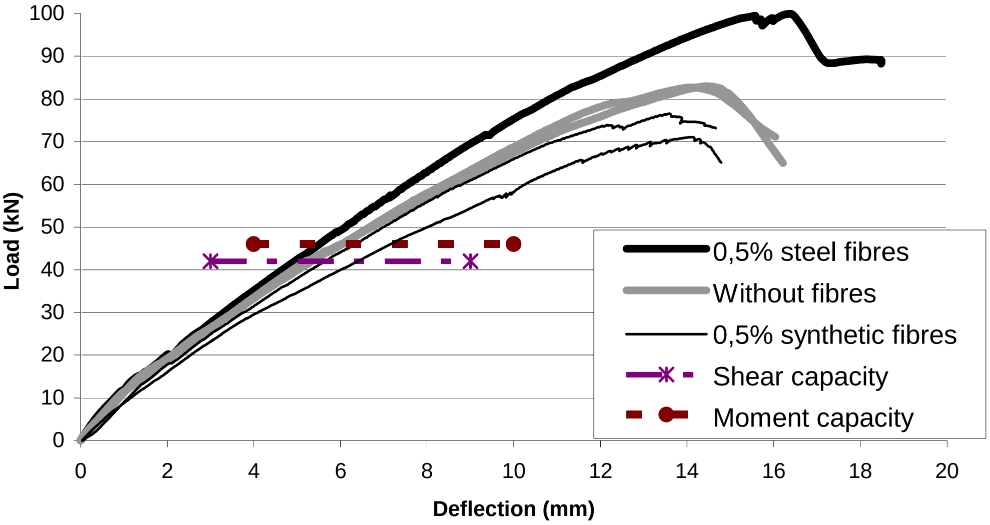


Figure 25. Measured load-deflection relations for composite beams with 0,5% Dramix steel fibres, 0,5 Barchip Shogun, and reference composite beams without fibres.

Figure 26. Pictures of composite beams, a) 5C- No fibres, b) 5D -No fibres, c) 9B – 0,5% Barchip synthetic fibres, d) 10B – 0,5% Barchip, d) 7A-0,5% Dramix end hooked steel fibres (65/35).

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Enclosure 1; Description of the different casting series

Series 1

Dato	**Sted**	**Resept**	**Blandinger**	**Hva ble støpt**
4. februar	M-lab NTNU	M1	1	- Alle uttrekksprøver for lettbetongen - Sylindre for fasthetsprøving

Skummet ble laget i en Kenwood kjøkkenmaskin og hadde egenskaper innenfor kravene i alle målinger. Lecakulene ble forfuktet for å unngå at de sugde vann fra betongen.

Blandeprosedyren fulgte følgende framdrift:

1. 1 min tørrblanding
2. 2 min våtblanding, Glenium Sky 542 tilsettes umiddelbart etter vann.
3. 2 min henstilling
4. 1 min etterblanding
5. Densitetsmåling: 1281 kg/m^3
6. Tilsetning av 8liter skum.
7. Densitetsmåling: 1144kg/m^3
8. Målt luftinnhold etter 10min: 16%
9. Målt luftinnhold etter 20min: 15%

Etter utstøping av de uttrekksformene som skulle være uten mikrofiber ble 0,3 vol-% mikrofiber tilsatt slumpen og de siste formene ble fylt.

Betongen var stabil og lett støpelig. Etter tilsetning av mikrofiber ble betongen noe stivere, men det var fortsatt ingen problemer med utstøpingen.

Series 2

Dato	Sted	Resept	Blandinger	Hva ble støpt
26. februar	maxit Lillestrøm	M2 (M3)	6xM2 1xM3	- Overdekningsbjelke O1A - O3B - Småbjelker B1A - B2C - Strekkprismer P1A - P1D - Sylindre til fasthetsprøving
27. februar	maxit Lillestrøm	M3	8	- Overdekningsbjelke O3C - O4D - Småbjelker B3A - B6F - Strekkprismer P2A - P4D - Sylindre til fasthetsprøving

Skummet ble laget i en spesialbygd skumgenerator og hadde stabile egenskaper i alle målinger. Betongen ble blandet i en innleid rotasjonsblander som viste seg å ikke fungere optimalt for denne betongresepten.

Blandeprosedyren fulgte følgende framdrift:
1. Tørrblanding av tilslag og bindemiddel
2. Tilsetter vann
3. Tilsetter tilsetningsstoff
4. Måler densitet
5. Tilsetter skum til ønsket densitet
6. Tilsetter makrofiber

På tross av stor innsats lyktes det ikke å ha klar en stabil betongresept innen støpingen tok til. M2 hadde derfor en tendens til å gi betong med separasjonsproblemer og mye falskluft. Det ble forsøkt å stabilisere betongen ved å variere typen og mengden av tilsetningsstoffer mellom hver blanding, så mengden tilsetningsstoff i resepten stemmer nødvendigvis ikke 100% for alle blandingene. Variasjone i tilsetningsstoffer hjalp ikke nevneverdig og til siste blanding på første dag ble det besluttet å øke sand/leca-forholdet i håp om at dette skulle stabilisere betongen. Den nye betongresepten fikk navnet M3 og ble benyttet på resten av blandingene denne støperunden. Dette gav noe bedre stabilitet og små utslag på densiteten. Likevel var ikke problemet med separasjon helt borte og utprøvingen av forskjellige tilsetningsstoffer fortsatte også med denne resepten.

Til blanding nr 5 på andre dag ble det tilsatt 0,3 vol-% mikrofiber for å finne effekten ved denne i herdet betong. Denne tilsetningen løste separasjonsproblemene for betongen og vi fikk en stabil betong å jobbe med.

Vi fant både baller av mikrofiber og silikaklumper i blanderen.

Utstøpingen:
- 1 vol-% syntetisk fiber hadde bedre støpelighet enn 1 vol-% stålfiber

Prøvestykkene fra denne støperunden ble preget av den ustabile betongen. Flere av prøvestykkene ble forkastet på grunn av tydelig ujevn fordeling av bindemiddel og dårlig utstøping.

Series 3

Dato	Sted	Resept	Blandinger	Hva ble støpt
16. mars	M-lab NTNU	M1	2	- Strekkprismer P5A - P6D

Skummet ble laget i en Kenwood kjøkkenmaskin og hadde egenskaper innenfor kravene i alle målinger. Lecakulene ble forfuktet for å unngå at de sugde vann fra betongen.

Blandeprosedyren fulgte følgende framdrift:
1. 1 min tørrblanding
2. 2 min våtblanding, Glenium Sky 542 tilsettes umiddelbart etter vann.
3. 2 min henstilling
4. 1 min etterblanding og tilsetning av 1 vol-% makrofiber
5. Densitetsmåling: 1309 kg/m^3
6. Tilsetning av 8liter skum.
7. Densitetsmåling: 1163 kg/m^3
8. Målt luftinnhold: 17%
9. Utstøping av P5A-P5D
10. Tilsetning av 1,5 vol-% makrofiber
11. Densitetsmåling: 1495,9 kg/m^3
12. Utstøping av P6A-P6D

Betongen var stabil, men det store fibervolumet gav utfordringer under utstøpingen.

Series 4

Dato	Sted	Resept	Blandinger	Hva ble støpt
18. mars	M-lab NTNU	B35	1	- Alle uttrekksprøver for Normalbetongen - Sylindre for fasthetsprøving

Dette er en velprøvd og godt fungerende selvkomprimerende betong som ikke bød på spesielle problemer under produksjonen.

Blandeprosedyren fulgte følgende framdrift:
1. 1 min tørrblanding
2. 2 min våtblanding.
3. 2 min henstilling
4. 1 min etterblanding

Betongen var stabil og lett støpelig. Etter tilsetning av mikrofiber ble betongen noe stivere, men det var fortsatt ingen problemer med utstøpingen.

Series 5

Dato	Sted	Resept	Blandinger	Hva ble støpt
30. mars	maxit Lillestrøm	M4	7	- Overdekningsbjelke O5A - O7B - Småbjelker B7A - B10F - Strekkprismer P7 - P8 - Sylindre til fasthetsprøving
31. mars	maxit Lillestrøm	M4	6	- Overdekningsbjelke O8C - O10D - Småbjelker B3A - B6F - Strekkprismer P11A - P12F - Sylindre til fasthetsprøving

Erfaringene med mikrofiber fra støp 2 gjorde at maxit hadde kommet fram til resept M4 som gav en god og stabil betong med det samme utstyret som ved støp 2. Skummet ble også denne gang laget i en spesialbygd skumgenerator og hadde stabile egenskaper i alle målinger.

Blandeprosedyren fulgte følgende framdrift:
1. Tørrblanding av tilslag og bindemiddel
2. Tilsetter mikrofiber
3. Tilsetter vann
4. Tilsetter tilsetningsstoff
5. Måler densitet
6. Tilsetter skum til ønsket densitet
7. Tilsetter makrofiber

Rotasjonsblanderen var fortsatt ikke optimal og vi fant også denne gang både baller av mikrofiber og silikaklumper i de fleste blandingene. Lecakulene var denne gang lettere enn spesifikasjonen og det er dermed grunn til å tro at fastheten også var lavere.

Utstøpingen:
- 1 vol-% syntetisk fiber hadde bedre støpelighet enn 1 vol-% stålfiber*
- URW hadde bedre støpelighet enn Barchip*

*Vurderingen av støpeligheten er gjort på grunnlag av hvor lett utstøpingen gikk, ikke på grunnlag av andre tester.

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