Prefabricated foundations with cell reinforcement for land-based wind turbines

Elforsk rapport 13:06

Martin Nilsson

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Preface

In regions exposed to low temperatures and icing the available time-slot during the year for building of the foundations and erecting the turbines can be short. It is then interesting to look at methods for prefabricated foundations when using slab foundations. In order to look at further development possibilities for such foundations the use of so called cell reinforcement was suggested. Specifically cell reinforcement as developed by Svensk Cellarmering Fabrik AB (CELLFAB) in Töre, Sweden was suggested.

In order to investigate the possibilities of using this reinforcement a project within the Swedish wind energy research programme "Vindforsk – III" was stared as project V-374.

The work was carried out by Martin Nilsson at Luleå University of Technology with assistance by Sten Forsström, Sweco and Johan Persson, Cellfab.

This report is the final report for project V-374.

Vindforsk – III is funded by ABB, Arise windpower, AQ System, E.ON Elnät, E.ON Vind Sverige, Energi Norge, Falkenberg Energi, Fortum, Fred. Olsen Renwables, Gothia wind, Göteborg Energi, HS Kraft, Jämtkraft, Karlstads Energi, Luleå Energi, Mälarenergi, O2, Rabbalshede Kraft, Skellefteå Kraft, Statkraft, Stena Renewable, Svenska Kraftnät, Tekniska Verken i Linköping, Triventus, Wallenstam, Varberg Energi, Vattenfall Vindkraft, Vestas Northern Europe, Öresundskraft and the Swedish Energy Agency.

Comments on the work and the final report have been given by a reference group with the following members: Manouchehr Hassanzadeh at Vattenfall and Rune Rönnholm at Triventus, Göran Ronsten representing O2 and Anders Björck at Elforsk.

Stockholm, October 2012

Anders Björck

Elforsk, Electricity and Power Production

Vindforsk-III Programme manager

Sammanfattning

Lönsamma vindkraftverk placeras i områden med goda eller mycket goda vindförhållanden. Avståndet till befintlig infrastruktur, såsom elektriska ledningar och vägar, får inte vara alltför långt. Många platser i norra Sverige har goda förutsättningar för en storskalig vindkraftsutbyggnad. För att kunna bygga i fjällnära områden krävs teknik för att bl.a. hantera en kort byggsäsong, nedisning och låg temperatur. Byggsäsongen på sådana platser är kort på grund av snö, tjäle och låga temperaturer. Stora vindkraftsparker med fler än 10-talet verk kan i fjäll- eller fjällnära områden knappast uppföras under en sommarsäsong med dagens utförande och teknik. För detta ändamål krävs vidareutveckling och nya tekniker.

Prefabricerade gravitationsfundament, bestående av på plats sammanfogade vertikala triangulära väggar och horisontella triangulära plattor, som täcks med lokala schaktmassor är en nyligen etablerad vidareutveckling av befintlig anläggningsteknik. Andra möjligheter är så kallad cellarmering av band i höghållfast stål med runda hål. En förhoppning är att cellarmering kan förkorta monteringstiden och förbättra arbetsmiljön. Genom att kombinera prefabricering med cellarmering kan byggtiden på plats förkortas.

Syftet med detta projekt var att undersöka möjligheten att använda prefabricerade betongelement med cellarmering i fundament till vindkraftverk. Frågan är om cellarmering är en signifikant mera kostnadseffektiv teknik jämfört med traditionella armeringsmetoder?

Ett första steg i att undersöka möjligheterna att använda cellarmering i prefabricerade fundament var att genom prov undersöka statiska och dynamiska hållfasthetsegenskaper för betongelement armerade med cellarmering.

Resultaten från dessa prov redovisas i rapporten.

Rapporten beskriver statiska test och utmattningsförsök av balkar och plattor med traditionell stångarmering och med cellarmering. Sammanlagt fem balkar (310 × 200 × 1200 mm - B × H × L) och fyra plattor (1200 × 200 × 1200 mm - B × H × L) testades. I utmattningsförsöken var målet att elementen skall kunna klara 2 000 000 belastningscykler med spänningsvidden i armeringen till mellan 25 och 75 % av dess sträckgräns.

Proven med balkarna visade inte tillräcklig bärförmåga för elementen med cellarmering. Såväl statisk som dynamisk bärförmåga var betydligt lägre än för elementen med traditionell armering. De två cellarmerade balkarna som testades i utmattning klarade endast 135 500 och 5 000 lastcykler. De två cellarmerade balkarna som testades statiskt klarade bara 49,5 respektive 41,0 kN som ska jämföras med teoretisk last om 97,3 respektive 77,1 kN.

I plattproven uppnådde de cellarmerade plattorna tillräcklig bärförmåga både med avseende på antal lastväxlingar och brottlast.

Betong- och stålkvaliteterna tillsammans med utformningen av cellarmeringen måste vara väl korrelerade. Vid dragbelastning av en enhet av cellarmering kommer betongen i hålen att motverka deformation av cellarmeringen. Om betongen är för stark kommer den inte att krossas och all deformation kommer att lokaliseras till några få ställen, vilket innebär höga lokala spänningar i armeringen som kan resultera i brott. Hål i plattor (här cellarmering) minskar naturligtvis lastkapaciteten. Genom ett snitt i ett hål får man spänningskon-

centrationer närmast öppningen som avsevärt överstiger de genomsnittliga spänningarna i snittet under elastiska förhållanden. Påkänningarna kan vara tre gånger större än medelvärdet. Därför fås lokal flytning vid hål för relativt låga belastningar.

Cellarmeringens dimensioner måste anpassas till den omgivande betongens kvalitet för att de två materialen ska fungera tillsammans.

Mer forskning behövs för att hitta konstruktionsmodeller för cellarmerad betong. För konstruktioner under utmattningsbelastning krävs också mer teoretiska studier liksom ytterligare tester.

Frågan om cellarmering är en signifikant mera kostnadseffektiv teknik jämfört med traditionella armeringsmetoder kan inte riktigt besvaras i nuläget.

Summary

Wind power plants should be located to areas with very good wind conditions and to places close to existing infrastructure such as electric lines and roads. A large share of future large wind farms in Sweden are planned to mountain or near mountain areas, which are often difficult to access and exposed to low temperatures and icing. The construction season at such places is short due to snow, ground frost and low temperatures. Wind power plants in mountain or near mountain areas can hardly be constructed in one summer season by today's workmanship and techniques without further development and new techniques.

One such development is using prefabricated foundations consisting of vertical triangular walls and horizontal triangular slabs that are cast together at site. One new technique is so-called cell reinforcement made by strips of high strength steel with circular openings. Cell reinforcement might shorten the mounting time and improving the working environment. By combining prefabrication with cell reinforcement the construction time at site can be shortened.

The aim of this project is to investigate the possibility to use prefabricated concrete elements with cell reinforcement in foundation of wind turbines. The question is if cell reinforcement is a significantly more cost effective technique compared to traditional reinforcement?

This report describes static and fatigue testing of concrete beam and slab elements with cell reinforcement. In total five beams (310×200×1200 mm - W×H×L) and four slabs (1200×200×1200 mm - W×H×L) were tested.

In the fatigue tests it was aimed for that the elements should be able to withstand 2 000 000 load cycles with stresses in the reinforcement between 25 and 75 % of its yield strength.

No cell reinforced beam was able to carry the 2 000 000 load cycles nor could they resist the theoretical static load. However, the cell reinforced slabs had all the required capacity.

It appears that the strength of the concrete and the steel quality together with the design of the cell reinforcement have to be well correlated. When loading a unit of cell reinforcement in tension the concrete within the openings counteract the deformation of the cell reinforcement. If the concrete is too strong it will not be crushed and all the deformations will be localised in a few places, implying high local stresses resulting in failure. Openings in plates (here cell reinforcement) naturally decrease the load carrying capacity. Through a section in an opening one gets stress concentrations closest to the opening that considerably exceed the mean stresses in the section, under elastic conditions. The stresses might be three times larger than the mean stresses. Therefore yielding will occur locally at the opening at relatively low loads.

The dimensions of cell reinforcement must be matched to the quality of the surrounding concrete in order to work correctly.

More research is needed to find design models for cell reinforced elements. For structures under fatigue loading more theoretical studies are needed as well as further testing.

List of content

1 Introduction

1.1 Background

Wind power plants should be located to areas with very good wind conditions. In Sweden there are a large number of so-called high wind places along the coastline, in the forests and in the mountains or near mountain areas. Often these areas compete with other interests. Wind power plants should preferably be located to places close to existing infrastructure such as electric lines and roads.

A large share of future large wind farms in Sweden are planned to be located in mountain or near mountain areas. Such placements require further development of adapted technologies and good spirit of enterprise. These locations are often difficult to access and exposed to low temperatures and icing. The construction season is shortened since snow, ground frost and low temperatures make ground work and construction of foundations more difficult or impossible during winter. Large wind farms (more than 10-15 plants) in mountain or near mountain areas can hardly be constructed by today’s workmanship and techniques during one summer season.

All wind turbines must be anchored in some kind of foundation. Today two main types are used: gravity foundations and rock anchored foundations. Rock anchored foundations need shallow and good rock. If the quality of the rock is low or if the rock is too deep it is not feasible to use rock anchored foundations. Foundations might be of massive concrete, which demand large quantities of concrete and must be cast at site. However, foundations can be made of prefabricated parts, a technique that requires considerably less concrete.

To be able to construct and adapt foundations to distant locations where

- it is a long way to ready mixed concrete plants,
- larger mobile mixing plants cannot or are not allowed,
- the ground water level is too shallow,
- the quality of the rock is too poor,
- or the rock is located at a too large depth

New design and construction techniques have to be found. The quality of the local aggregate might also be too low making it more difficult to manufacture concrete at site.

An alternative design of foundation consists of prefabricated vertical triangular walls and horizontal triangular slabs, see figure 1. Between the vertical walls and on the horizontal slabs the local material that was dug out for the foundation is refilled. Prefabricated foundations have one big advantage compared to traditional gravity foundations when the plant is about to be dismounted. With relatively small effort the foundation can be taken apart and transported from site in pieces of the same sizes as during construction.

Figure 1 Examples of prefabricated gravity foundations, SIC (2009) and Sjijska (2012).

A number of patents, for example DK200100030 (2001) and WO2004101898A2 (2004), regarding prefabricated gravity foundations exist in Europe and the USA. However, all are designed for smaller wind power stations compared to these aimed at in this project. It is not clear if any of the foundations in these patents have been constructed.

Only a small volume of fresh concrete is needed when joining the elements together. For the alternative with vertical walls joined with horizontal slabs one need to cast concrete in the gap between them, see Figure 2. In such gap reinforcement stirrups come out from the edges of the elements.

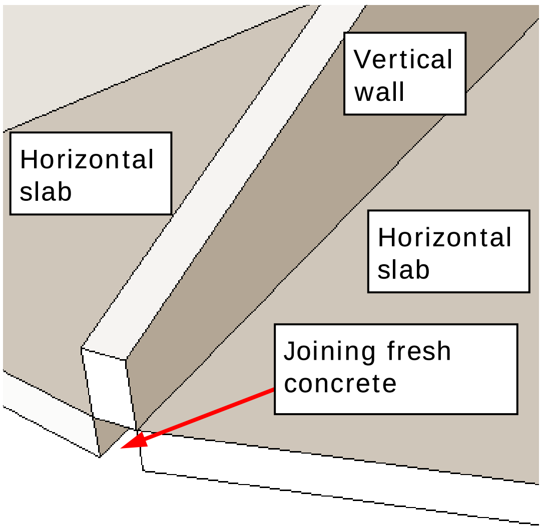

Figure 2 Joining of elements by fresh concrete or.

From an environmental point of view the studied type of foundation has one additional great advantage. The transport work can drastically be reduced, since the need of concrete is much smaller as counterweight by using on site materials. Concrete is a natural material consisting of cement (limestone, clay etc.), sand, gravel and water. The manufacturing of cement produces large amounts of carbon dioxide – like other construction material – but the industry is developing and the discharging reduces more and more. Concrete can, if not contaminated, be recycled. Old concrete can be crushed and reused in constructions of roads or as aggregate in new concrete.

When manufacturing elements in a factory it is desirable to rationalise the work with formwork, reinforcement and casting. First, the reinforcement work can largely be more effective by using more rational reinforcement techniques. One such rational reinforcement is so-called cell reinforcement; see Figure 3 and Figure 4. Cell reinforcement is made by punching holes in sheet metal. In cell reinforcement high strength steel is used compared to traditional reinforcement. By using cell reinforcement, large areas can be reinforced in short time compared to traditional bending and lashing of ordinary reinforcement bars.

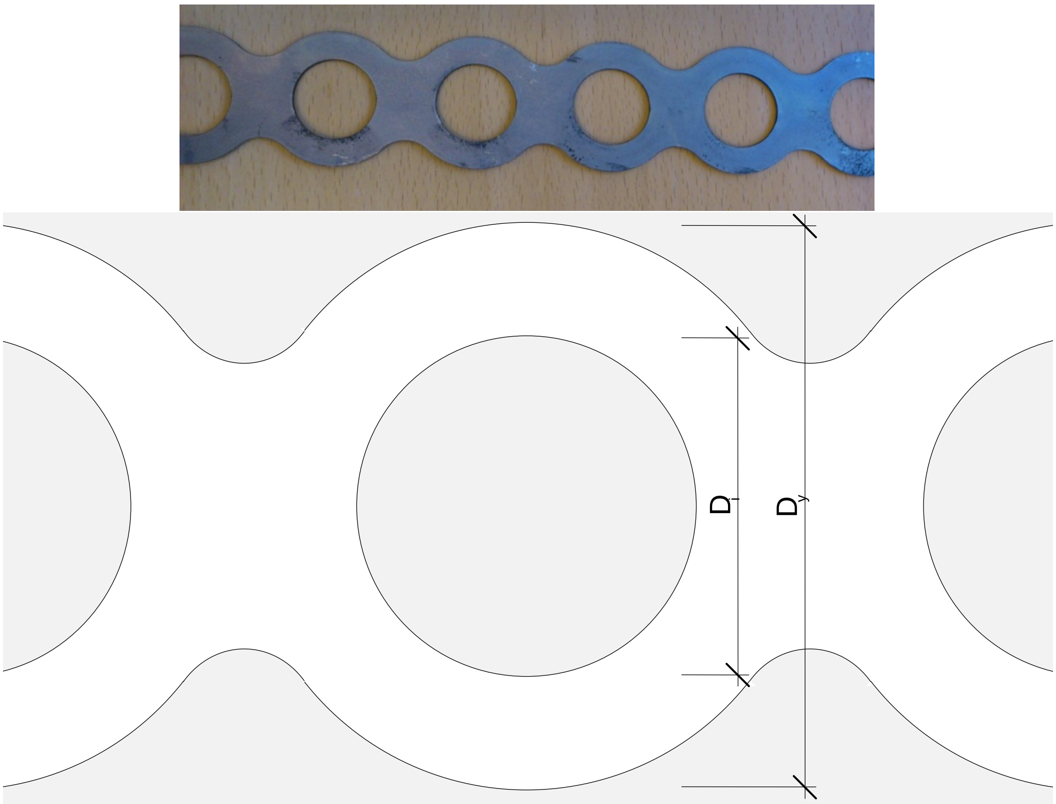

Figure 3 One row of cell reinforcement.

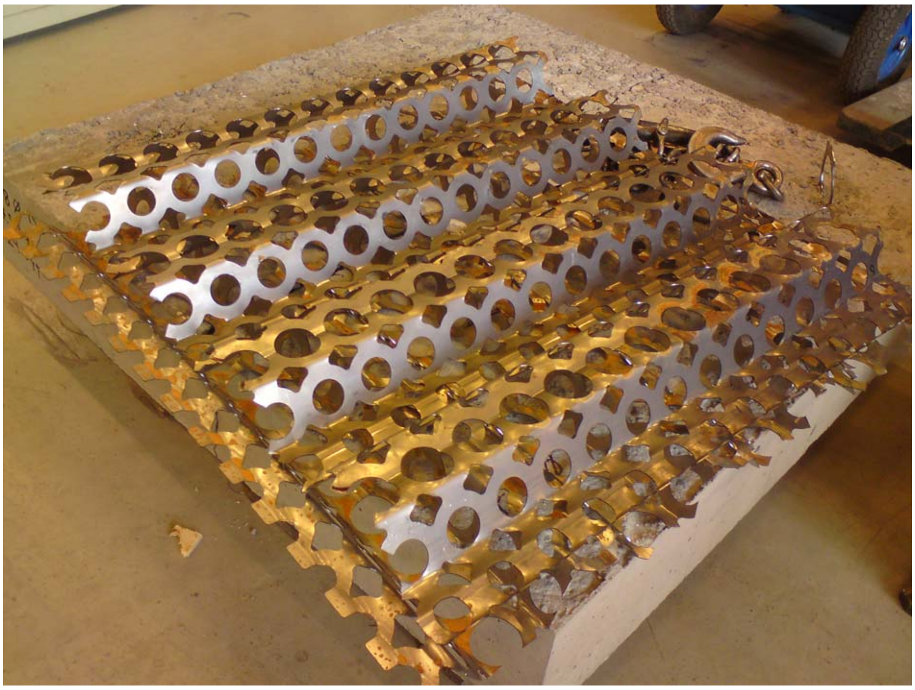

Figure 4 Cell reinforcement shaped as a corrugated plate.

To be able to meet the challenges that have been described in a cost-effective way an alternative solution of design is suggested to be used together with cell reinforcement. The design solution is based on prefabricated concrete elements that are put together to one homogeneous gravity foundation, in opposite to traditional solutions with large in-situ contract works.

Advantages with prefabricated foundations are:

- considerable savings in labour, material, machinery and transportation
- reduced construction time (possibility to construct large plants in extreme climates in one short season)
- improved/simplified transportation logistics
- improved possibility to construct in remote and extreme locations
- improved quality through production of elements in factories with good conditions
- refilling with excavated material on site as counterweight instead of concrete reduces costs and pollution load (reduced usage of concrete by about 2/3).

Construction of prefabricated foundations embodies the following steps:

- manufacturing of elements in factory in controlled environment
- excavation and levelling of bottom of foundation
- transportation of element to plant
- assembling of elements
- assembling of connecting reinforcement and casting of joining concrete
- possible stressing of post-tensioned reinforcement
- refilling with excavated material

1.2 Project description

The wind power company O2 Vindkompaniet wants to investigate if it is possible to replace traditional gravity foundations with foundations constructed with prefabricated concrete elements. To further rationalise the manufacturing of such foundation elements possible new techniques and solutions might be used, such as cell reinforcement. Cell reinforcement is a new reinforcement technique with high strength steel that is being developed by Svensk Cellarmering Fabrik AB (CELLFAB) in Töre, Sweden.

Luleå University of Technology (LTU) and the Division of Structural and construction Engineering applied for grants for a project to lead and carrying through

1. fatigue tests in LTU’s laboratory of concrete elements with cell reinforcement,
2. static failure test in LTU’s laboratory of structural parts with cell reinforcement,
3. fatigue tests in LTU’s laboratory of structural parts with cell reinforcement.

1.3 Aims and purposes

The aim of the project is to investigate the possibility to use prefabricated concrete elements with cell reinforcement in foundation of wind turbines.

The purpose is to use prefabricated foundations so that more foundations can be constructed during one season compared to traditional foundations.

Cell reinforcement might be more suitable for construction (working environment, time and to some extent also steel weight) for both prefabrication and on site.

Question: is cell reinforcement a significantly more cost effective technique compared to traditional reinforcement?

1.4 Limitations

This report only describes the first of the three parts in the original project plan, see section 1.2. The results from the fatigue tests of cell reinforced concrete elements were not clear and positive. Therefore, the project group didn’t find it fruitful to continue the project with parts 2 and 3.

2 Laboratory tests

2.1 Concrete elements with cell reinforcement tested in fatigue

The general plan for the testing of cell reinforcement in fatigue was to use beam and slabs. In the slabs two different orientations of the cell reinforcement was used, standing and lying. In all elements concrete of same concrete strength class was used as in real foundations, that is C35/45.

In the fatigue tests the load was varied between 25 and 75 % of the yield strength of the reinforcement steel. The aim for the fatigue tests was that the beams and the slabs shall be able to carry 2 000 000 load cycles.

2.1.1 Test specimen

The test plan of the fatigue capacity and behaviour of elements with cell reinforcement was to use seven beams with measures 310×200×1200 mm (B×H×L) and three slabs with measures 1200×200×1200 mm (B×H×L). Concrete strength class was C35/45 and the concrete cover 15 mm.

Three different types of beams were used. The first type (reference beam labelled BT1) was reinforced with traditional bar reinforcement in quality B500B with 3Ø10-A in bending and 10Ø10-N s120 in shear, see Figure 5. The second type (labelled BC1, BC2 and BC3) was reinforced with three lying units of cell reinforcement in steel quality SSAB Docol 1000, see Figure 6. The third type (labelled BC4, BC5 and BC6) was reinforced with three standing units of cell reinforcement, see Figure 7.

Two different types of slabs were used. The first type (reference slab labelled ST1) was reinforced with bar reinforcement in Quality B500B 12Ø10-A in one direction and 18Ø12-A in the perpendicular direction, see Figure 8. The second type (labelled SC1, SC2 and SC3) contained cell reinforcement in a net. Inside 25 standing units of cell reinforcement with inner diameter 45 mm were 15 units of lying cell reinforcement with outer diameter 45 mm.

The specimens were cast by Bröderna Hedmans cementgjuteri, a manufacturer in Älvsbyn, Sweden. The cell reinforcement was produced by laser cutting performed by Svets & Skärteknik Luleå AB in Luleå, Sweden. All the production was overlooked by Johan Persson, Cellfab.

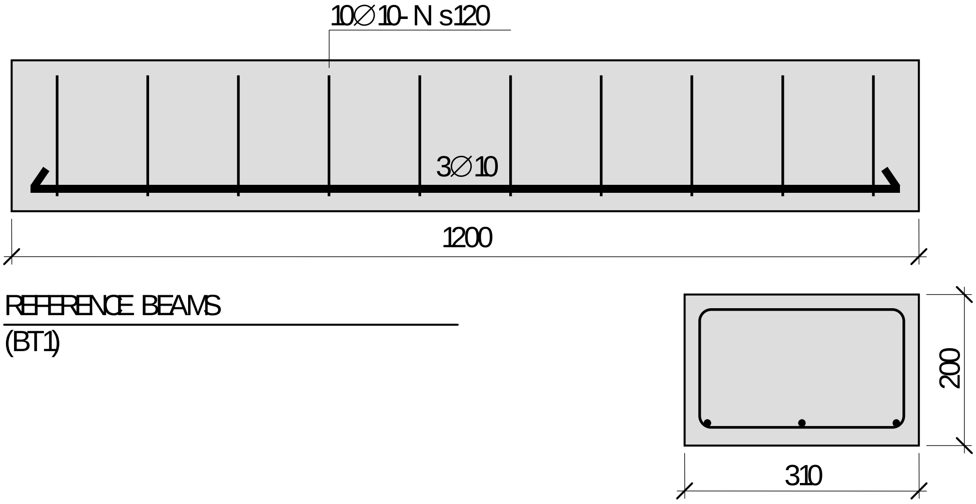

Figure 5 Drawing of reference beams

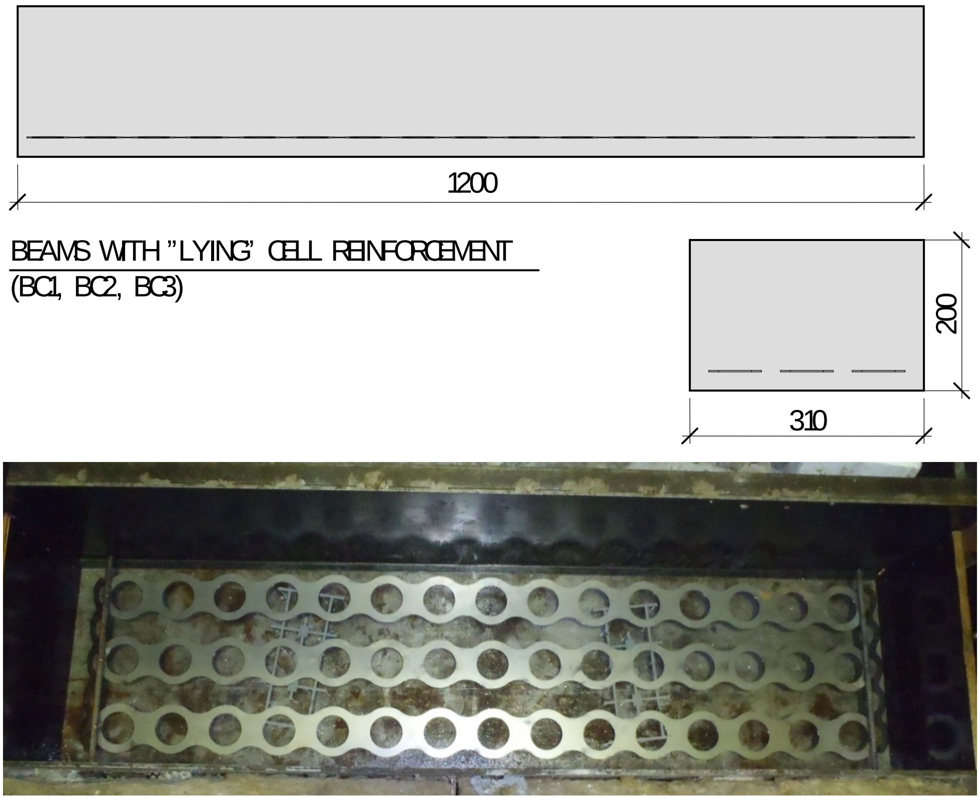

Figure 6 Drawing and photo of beams with ”laying” cell reinforcement

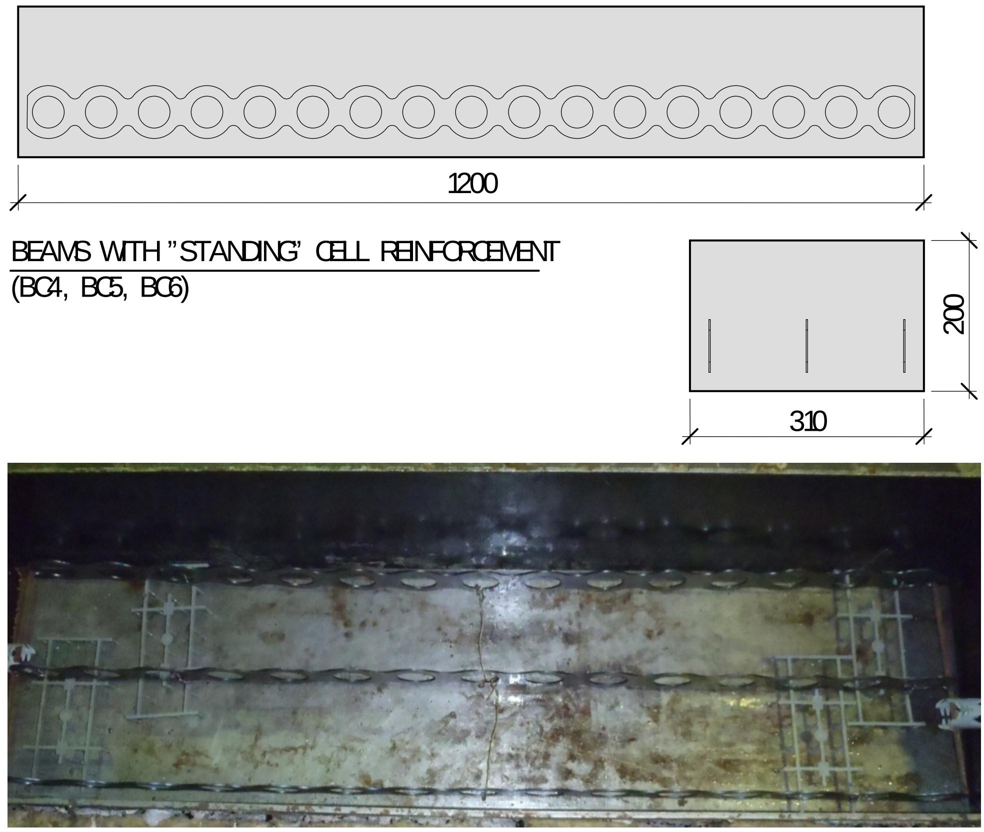

Figure 7 Drawing and photo of beams with "standing" cell reinforcement

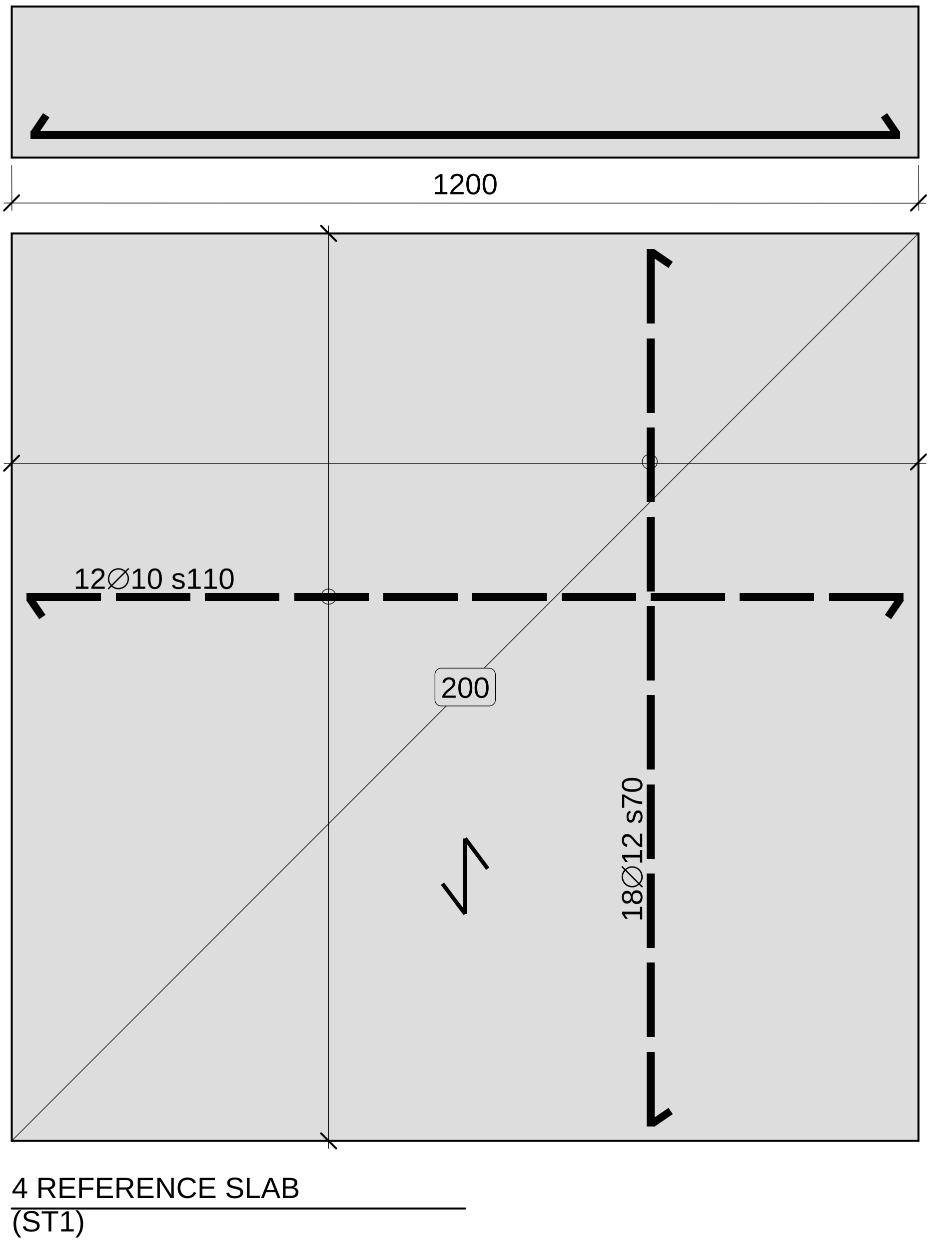

Figure 8 Drawing of reference slab with bar reinforcement.

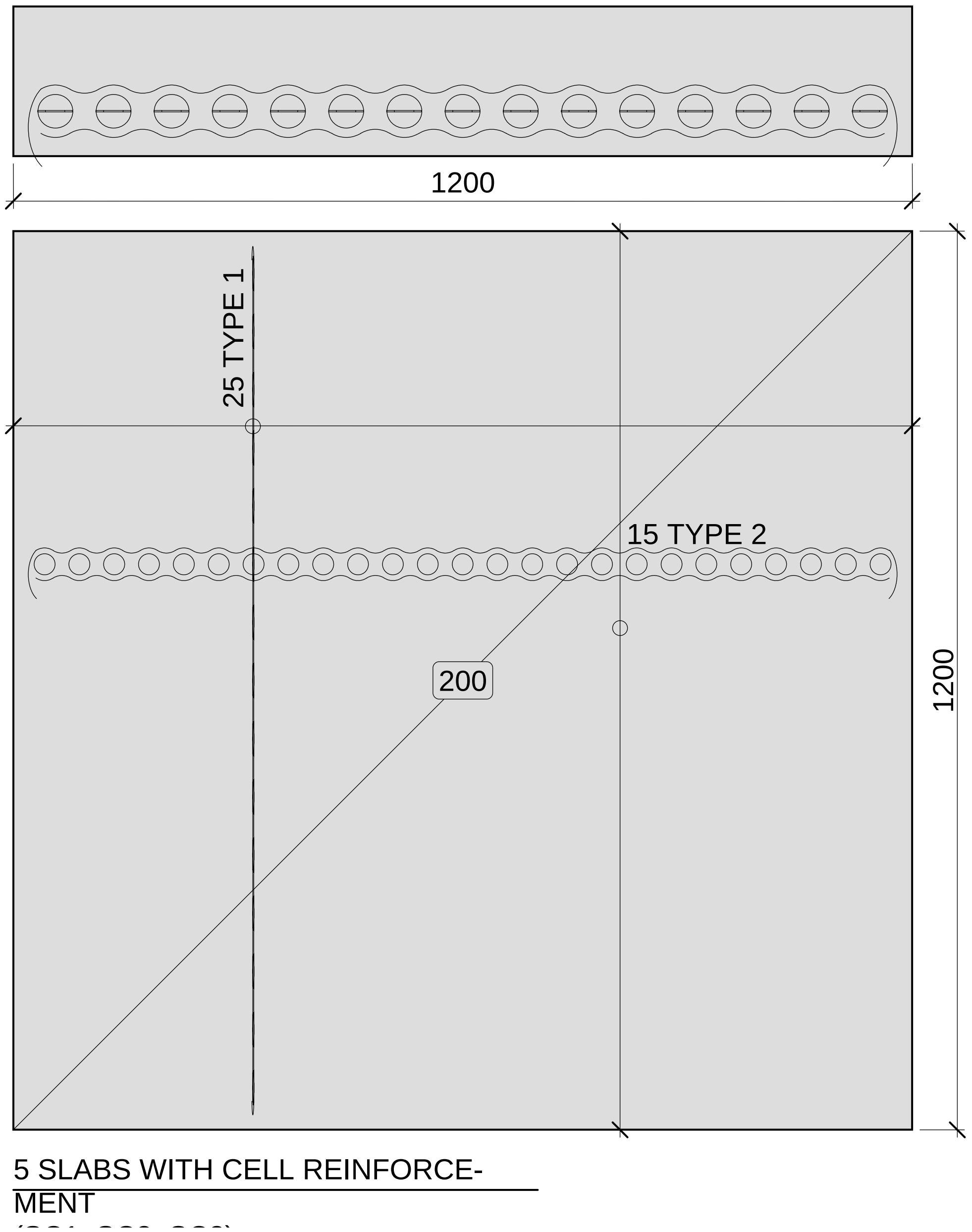

Figure 9 Drawing of slabs with cell reinforcement.

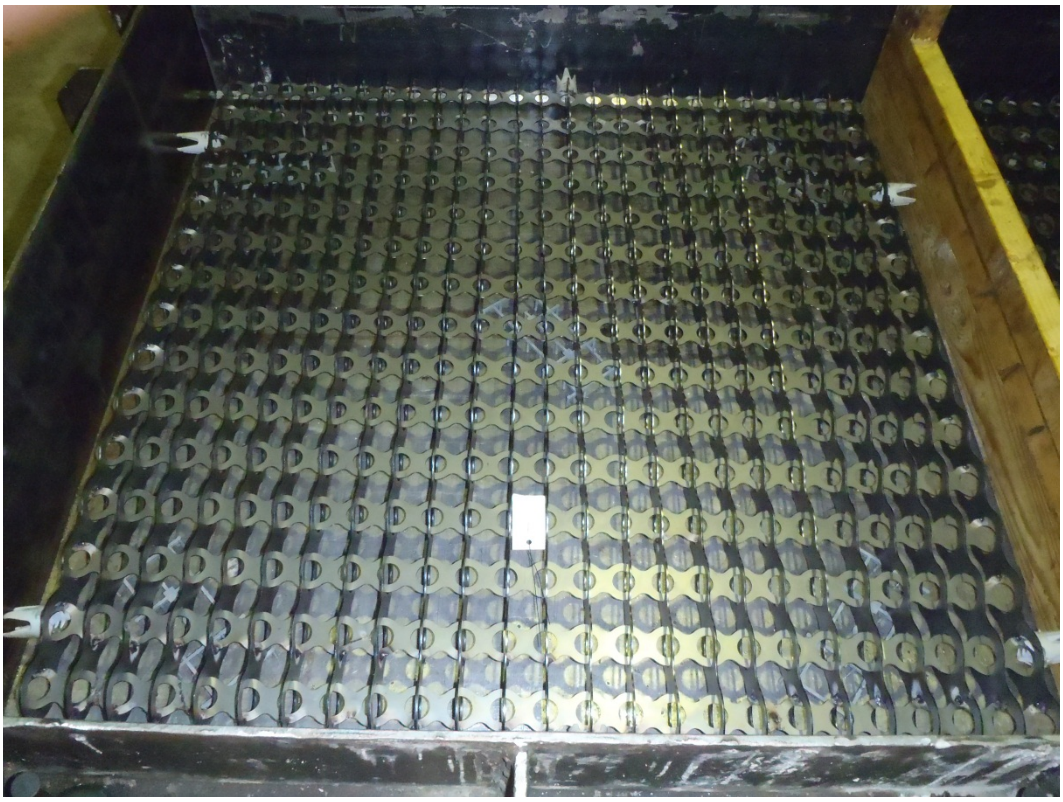

Figure 10 Photo prior casting of slab SC1 with cell reinforcement

2.2 Test setup

2.2.1 Beams

The beams were tested in four points bending, i.e. with two concentrated loads on the upper side and two supports at the ends of the beams, see Figure 11. The supports were located 50 mm and the concentrated loads 375 mm from the ends of the beams. Under the beams the deformation was measured at three points: under the loads (labelled Def_A and Def_C) and in the middle (labelled Def_B).

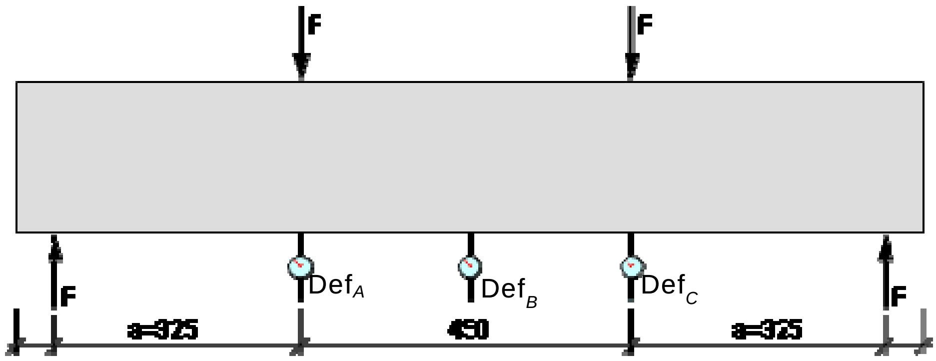

Figure 11 Test setup of four points bending of beams. Two symmetrically loads applied at the upper surface and three points where the deformations are measured.

2.2.2 Slabs

The slabs were tested in a similar way as the beams. They were supported on a circular steel ring with diameter 1100 mm. The load was applied centrically by a circular point load with diameter 300 mm in the two first slabs and diameter 450 mm in the third and last test. See Figure 12.

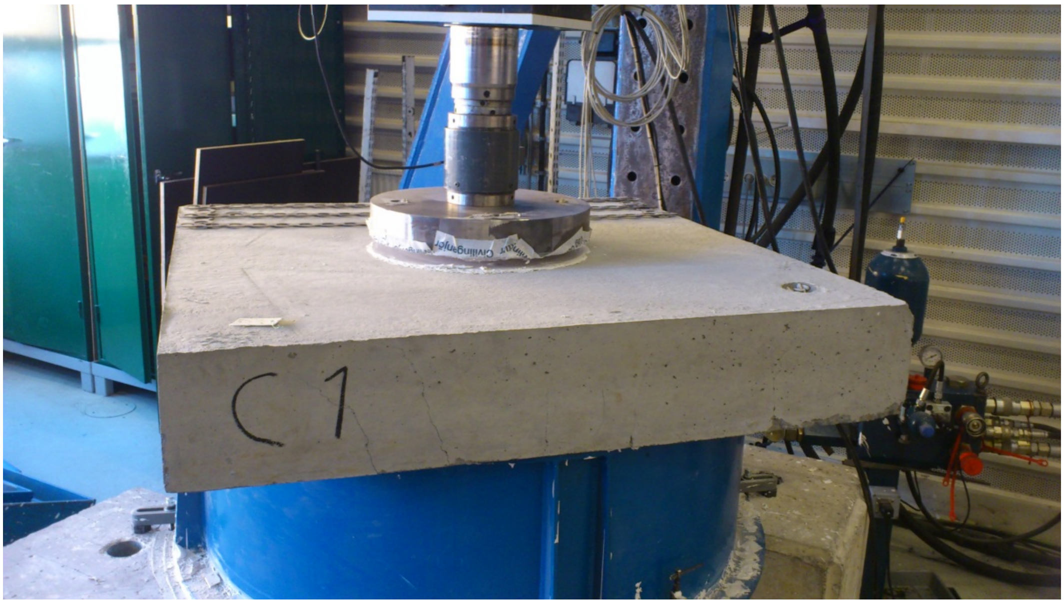

Figure 12 Test setup for bending of slabs by centric load and circular support.

2.2.3 Load levels

The design of the beams and the slabs is based on that the stresses in the reinforcement during the fatigue tests shall vary between 25 and 75 % of the yield strength of the reinforcement steel. Below in Table 1 the geometries, material data and theoretical load levels are presented, see also Appendix 1 – Test specimen design.

Table 1 Geometries, material data and load levels of test beams.

	Reference beam (BT1)		Beams with ”lying” cell reinforcement (BC1, BC2, BC3)		Beams with ”standing” cell reinforcement (BC4, BC5, BC6)	
Geometries						
Width, b [mm]	310		310		310	
Length, L [mm]	1200		1200		1200	
Span length, L_1 [mm]	1100		1100		1100	
Dist. from support to point load, a [mm]	325		325		325	
Reinforcement area, A_s [mm²]	235.6		180		180	
Effective height, d [mm]	180		184		147.5	
Concrete area, $A_c = d{\cdot}b$ [mm²]	55 800		57 040		45 725	
Reinforcement percent-age $\rho = A_s/A_c$ [–]	0.0042		0.0032		0.0039	
Material data						
Concrete						
Young’s moduls, E_c [GPa]	34		34		34	
Mean axial tensile strength, f_{ctm} [MPa]	3.20		3.20		3.20	
Reinforcement						
Young’s moduls, E_s [GPa]	200		210		210	
Yield strength, f_y [MPa]	500		1000		1000	
Loads						
Cracking load, F_I [kN]	29.9		29.8		29.0	
Fatigue load, F_{II} [kN]	15.2	45.7	24.0	71.9	19.1	57.2
Failure load, F_{III} [kN]	63.3		97.3		77.1	

Table 2 Geometries, material data and load levels of test slabs.

	Reference slab (ST1)		Cell reinforced slab (SC2, SC3)		Cell reinforced slab (SC1)	
Geometries						
Width, b [mm]	1200		1200		1200	
Length, L [mm]	1200		1200		1200	
Span length, L_1 [mm]	1100		1100		1100	
Dist. from support to point load, a [mm]	400		400		325	
∅ [mm]	10	18				
n_s [pieces]	12	12				
A_s [mm²]	785.4	1696.5	475	1250	475	1250
d_x and d_y [mm]	168.0	179.0	147.5	147.5	147.5	147.5
Concrete area, A_c = $d{\cdot}1000$ [$\cdot 10^3$ mm²]	168.0	179.0	147.5	147.5	147.5	147.5
Reinforcement percentage $\rho = A_s/A_c$ [–]	0.0047	0.0095	0.0032	0.0085	0.0032	0.0085
Mean value $\rho = (\rho_x \rho_y)^{0,5}$	0.0067		0.0052		0.0052	
Material data						
Concrete						
Young's moduls, E_c [GPa]	34		34		34	
Mean axial tensile strength, f_{ctm} [MPa]	3,20		3,20		3,20	
Reinforcement						
Young's moduls, E_s [GPa]	200		210		210	
Yield strength, f_y [MPa]	500		1000		1000	
Loads						
Cracking load, F_I [kN]	298	316	291	297	327	334
Fatigue load 0,25f_y , F_{II} [kN]	135	202	144	368	162	487
Fatigue load 0,75f_y , F_{II} [kN]	400	561	433	1103	414	1241
Failure load, F_{III} [kN]	645		719		719	

3 Results

The results from the testing are presented below in Table 3 (beams) - Table 4 (slabs) and Figure 13 - Figure 30. In the load deformation diagrams the load values shown is the load applied by the hydraulic cylinder. These values shall be divided by two to be compared with the values in Table 1. Beams BC3 and BC5 were not tested since the tests of beams BC1 and BC4 did not give any results.

Table 3 Results from testing of beams.

		Load levels [kN]		Load cycles [$\cdot 10^3$]		
Label	Test	aimed	used	aimed	used	Figures
BT1	Static (failure) fatigue	15.5-45.0	15.5-44.2	2 000	2 000	Figure 13 - Figure 15
BC1	Static (cracking) Fatigue	24.0-71.5	23.9-68.9	2 000	135.5	Figure 16 - Figure 18
BC2	Static (failure)	97.3	49.5			Figure 19 - Figure 20
BC3	Fatigue	24.0-71.5	Not tested	2 000	Not tested	
BC4	Fatigue	19.0-57.0	22.0-64.8	2 000	0.005	Figure 21 - Figure 23
BC5	Fatigue	19.0-57.0	Not tested	2 000	Not tested	
BC6	Static (failure)	77.1	41.0			Figure 24 - Figure 25

Table 4 Results from testing of slabs.

		Load levels [kN]		$\varnothing_{load}$ [mm]	Load cycles [$\cdot 10^3$]		
Label	Test	aimed	used		aimed	used	Figures
ST1	Fatigue	150-450	135-397	300	2 000	2 000	Figure 26
SC1	Fatigue	162-487	86-254	450	2 000	2 000	Figure 28
SC2	Fatigue	162-487	76.5-226	300	2 000	2 000	Figure 29
SC3	Static	719	600	300			Figure 30

3.1 Beams

3.1.1 Reference beam (BT1)

Fatigue test and static failure test (after fatigue test).

Cracking load: about 60 kN $\Rightarrow F_I$ = 30 kN

Failure load: about 127 kN $\Rightarrow F_{III}$ = 63.5 kN

Number of load cycles: > 2 000 000

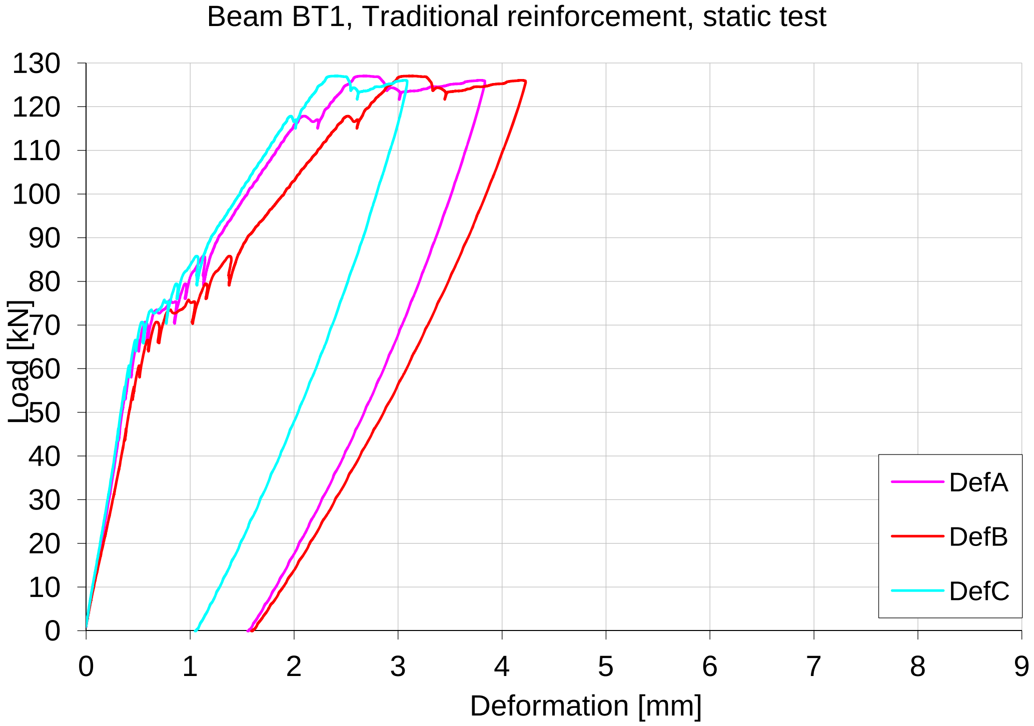

Figure 13 Load-deformations (deformations under the loads and in the middle) for static test of beam BT1 with traditional bar reinforcement.

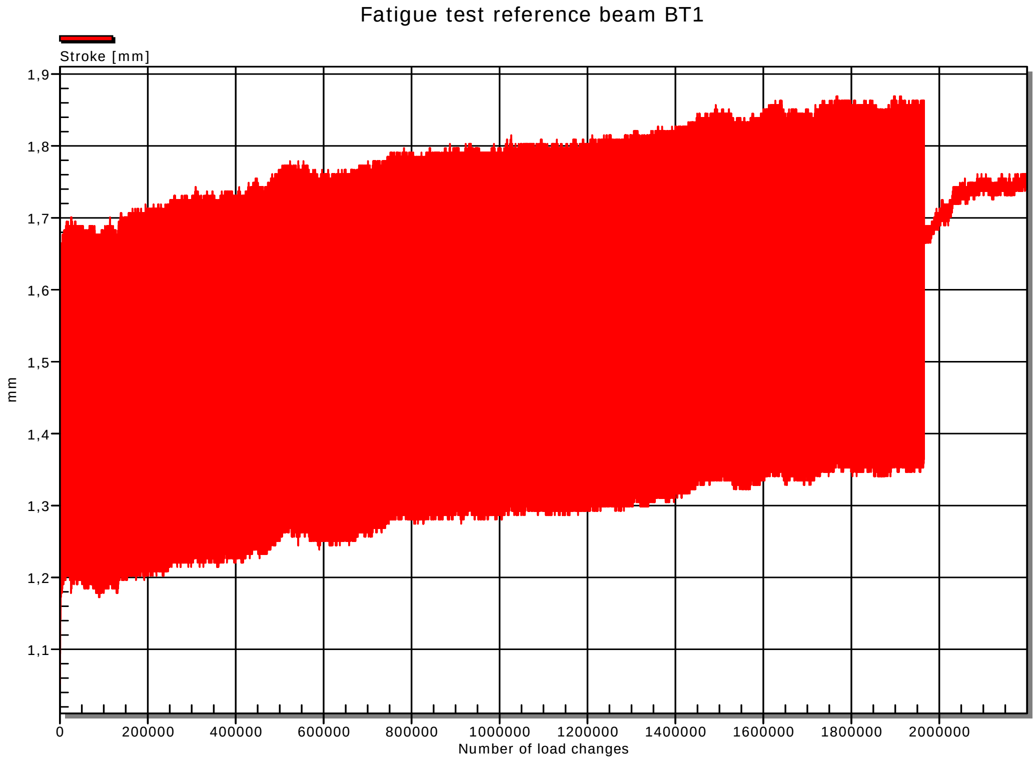

Figure 14 Diagram of deformation versus number of load changes for fatigue test of beam BT1 with traditional bar reinforcement

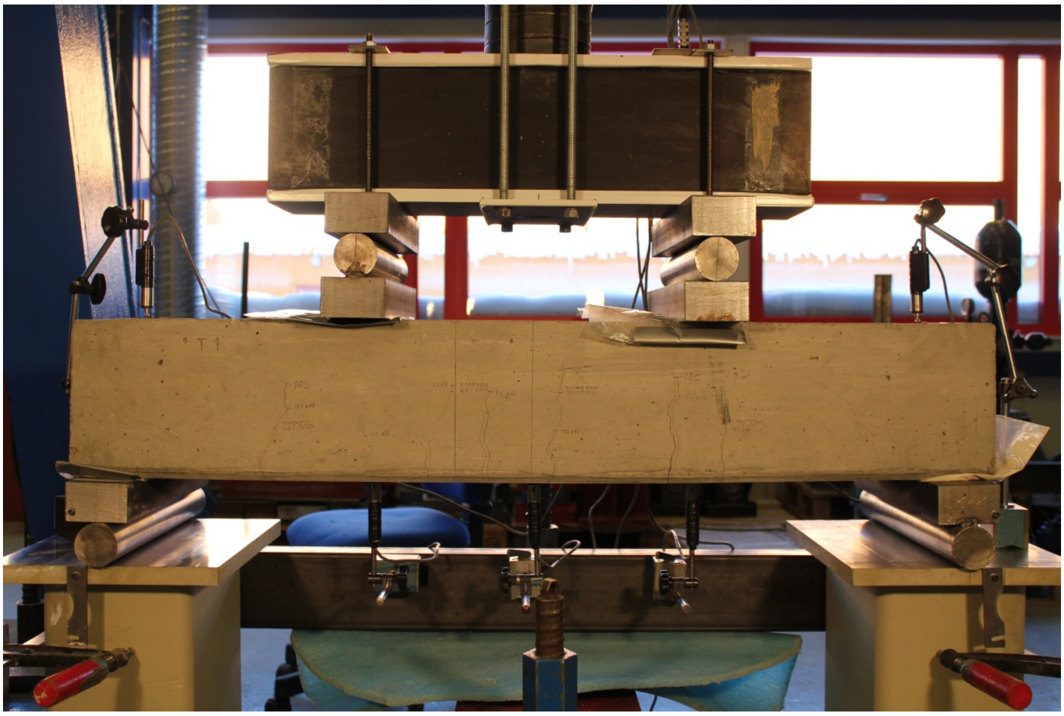

Figure 15 Beam BT1 at load level 105 kN

3.1.2 Beams with lying cell reinforcement (BC1 & BC2)

Beam BC1
Static test up to cracking of the concrete and fatigue test to failure

Cracking load: about 55 kN ⇒ F_l = 27.5 kN

Number of load cycles: about 135 000

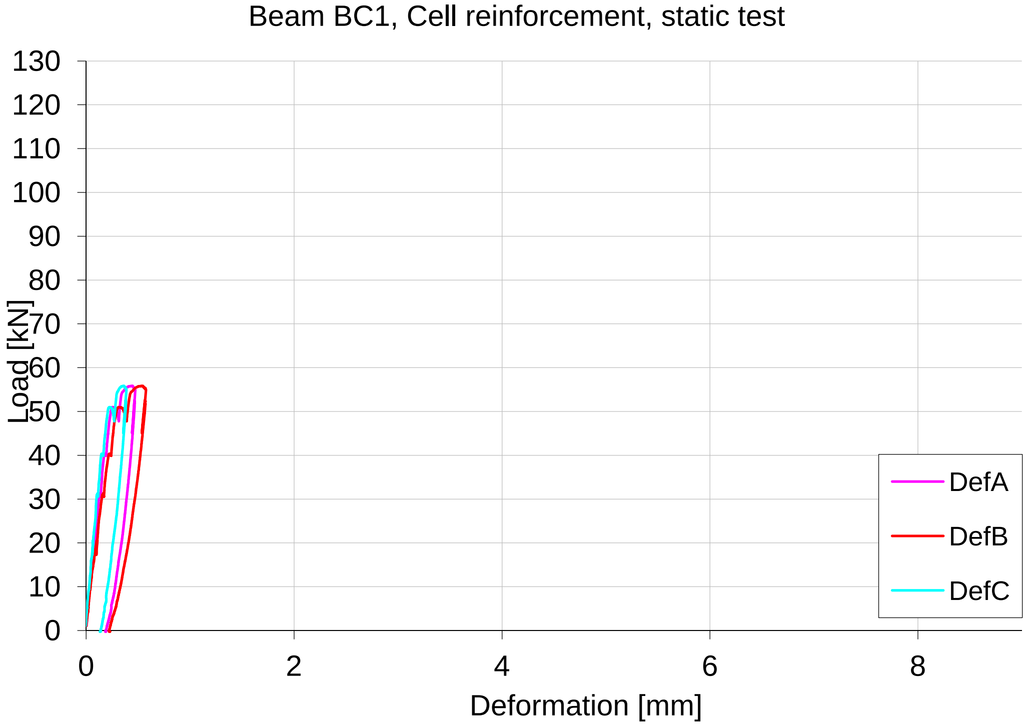

Figure 16 Load-deformations (deformations under the loads and in the middle) diagram for static test up to cracking of beam BC1 with cell reinforcement.

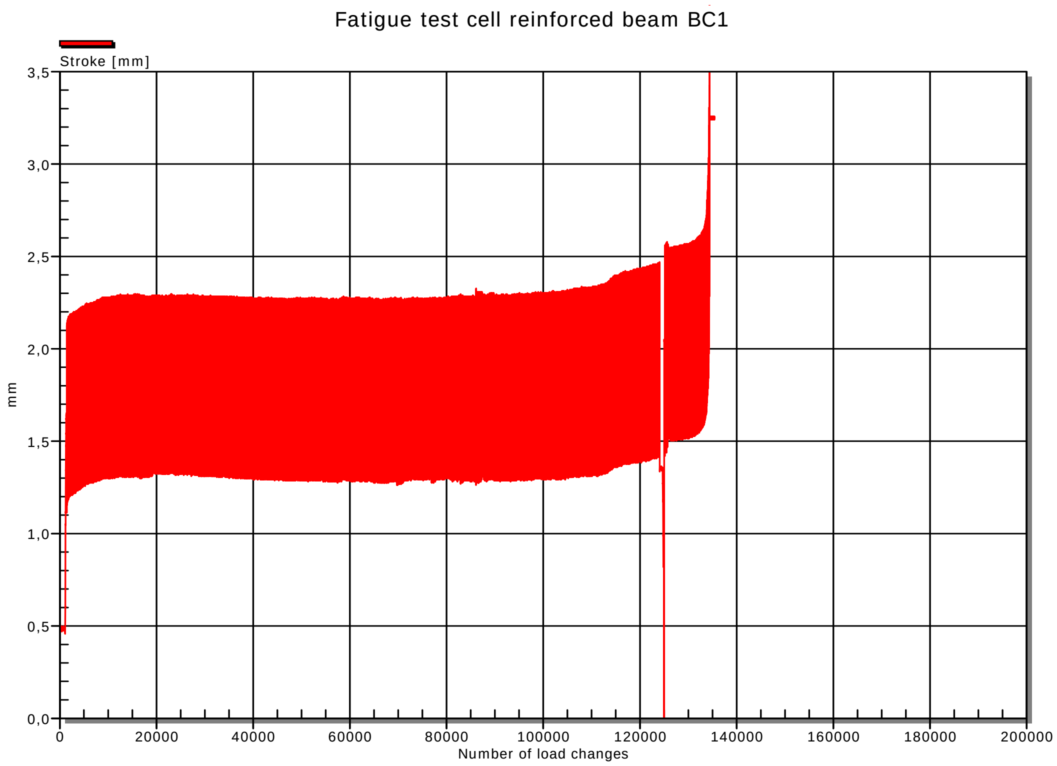

Figure 17 Diagram of deformation versus number of load changes for fatigue test of beam BC1 with cell reinforcement.

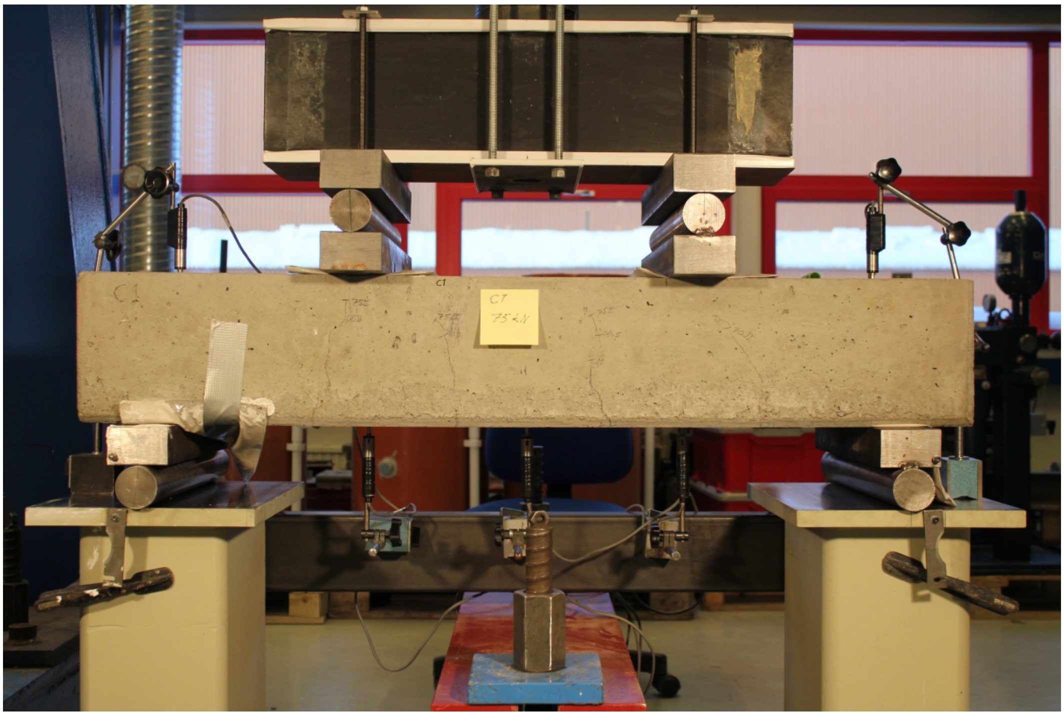

Figure 18 Beam BC1 at load level 75 kN with four distinctive cracks.

Beam BC2
Static test up to failure.

Cracking load: about 50 kN $\Rightarrow F_I$ = 25 kN

Failure load: about 99 kN $\Rightarrow F_{III}$ = 49.5 kN

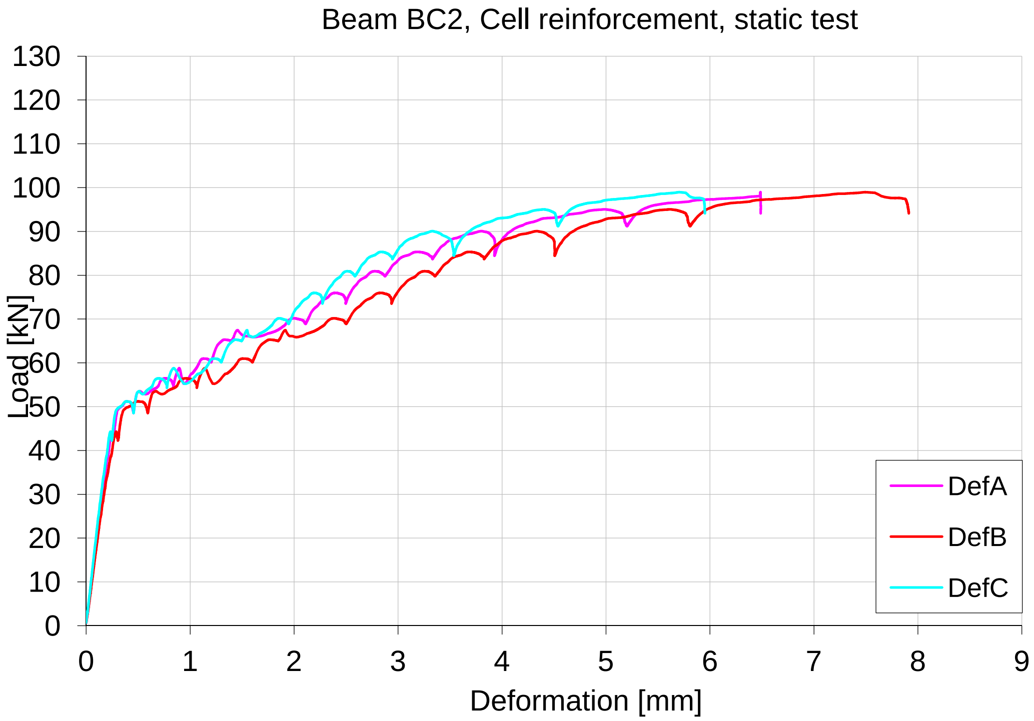

Figure 19 Load-deformations (deformations under the loads and in the middle) diagram for static test of beam BC2 with cell reinforcement.

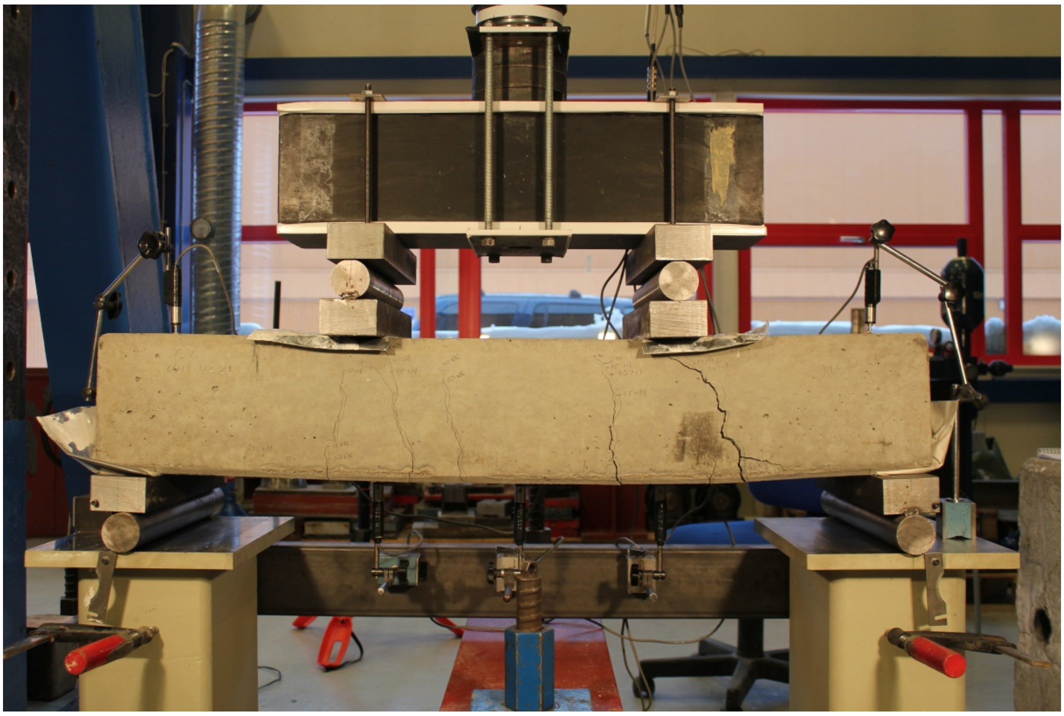

Figure 20 Photo of beam BC2 at load level 75 kN.

3.1.3 Beams with standing cell reinforcement (BC4 & BC6)

Beam BC4
Static test up to cracking and fatigue test to failure

Cracking load: about 45 kN ⇒ F_l = 22.5 kN

Number of load cycles: 5 070

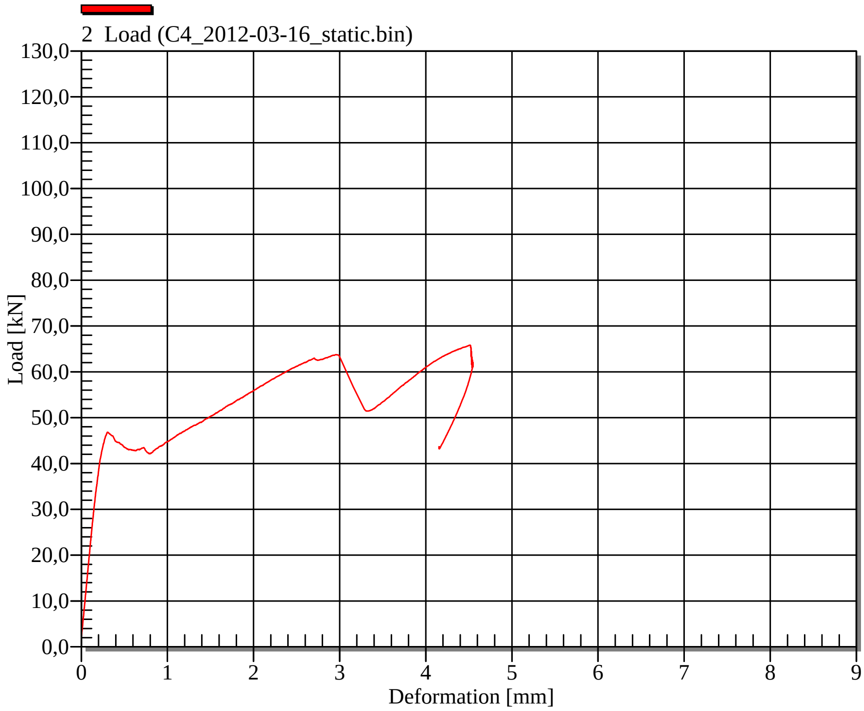

Figure 21 Load-deformations diagram for static test of beam BC4 with cell reinforcement.

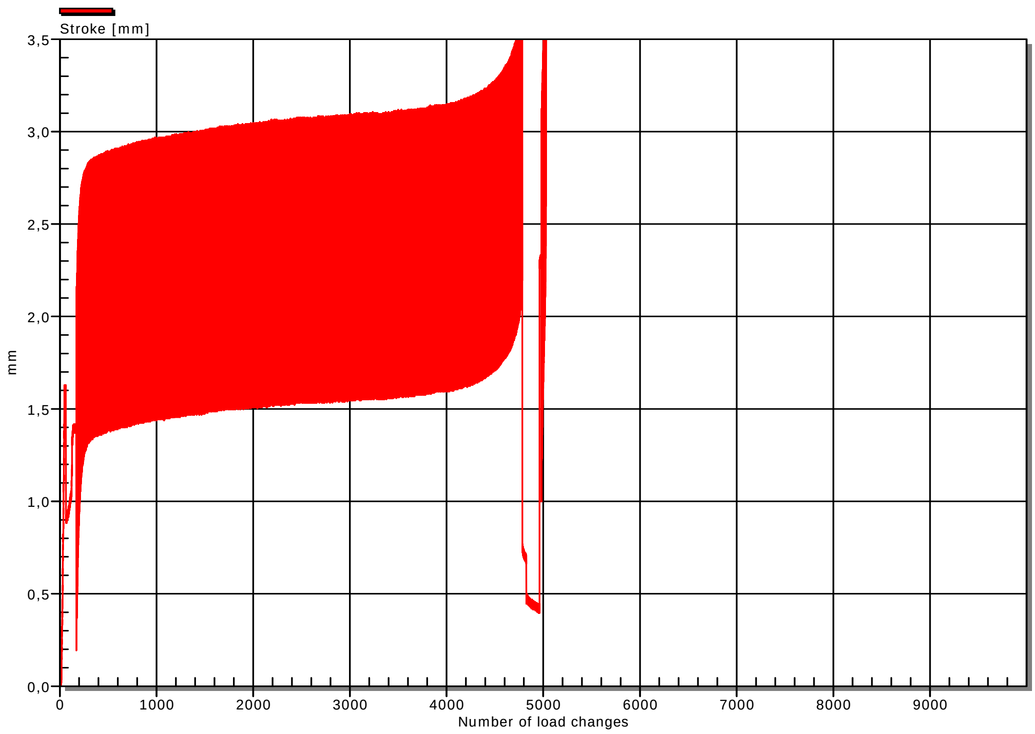

Figure 22 Diagram of deformation versus number of load changes for fatigue test of beam BC4 with cell reinforcement

Figure 23 Photo of beam BC4 after testing.

Beam BC6
Static test up to failure.

Cracking load: about 40 kN ⇒ F_I = 20 kN

Failure load: about 87.5 kN ⇒ F_{III} = 43.8 kN

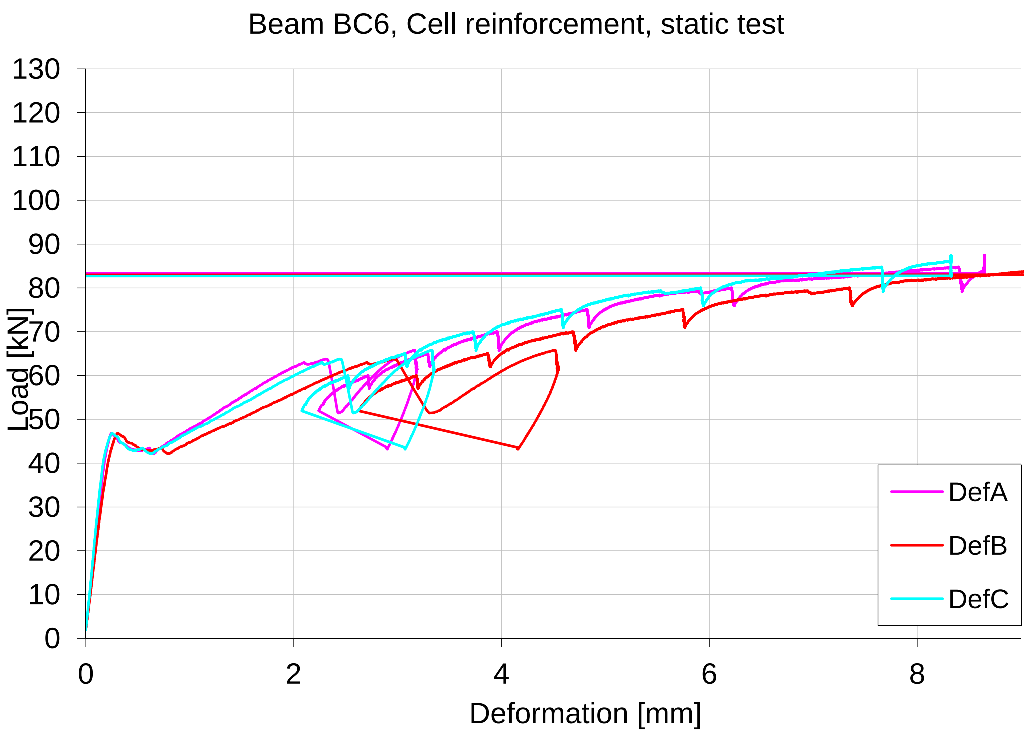

Figure 24 Load-deformations (deformations under the loads and in the middle) diagram for static test of beam BC6 with cell reinforcement.

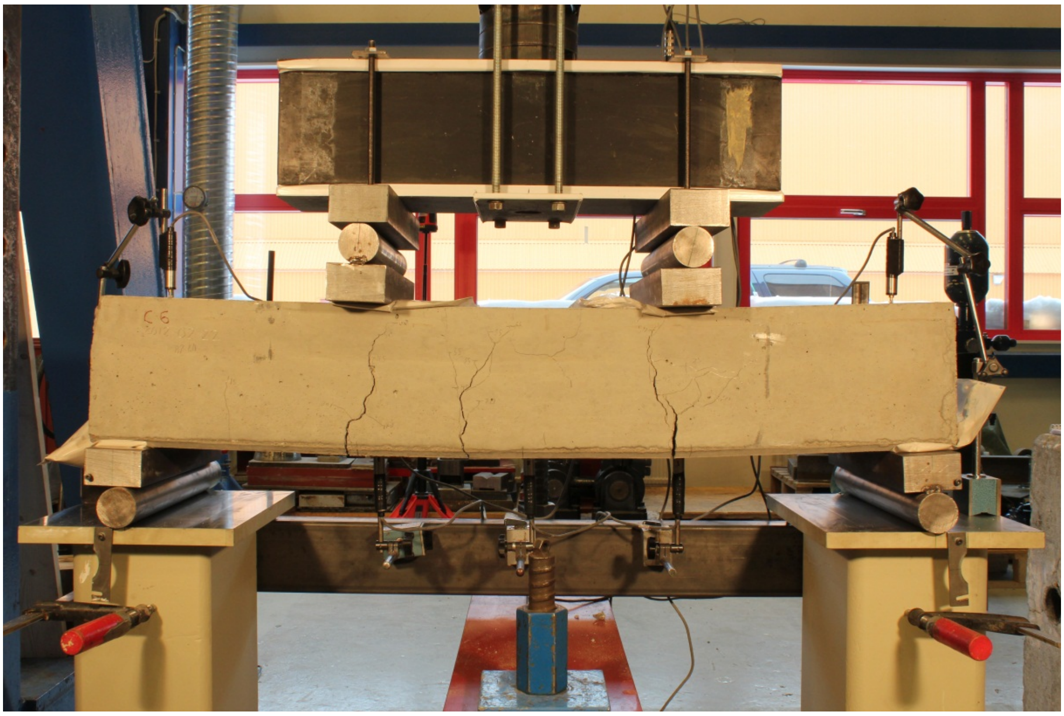

Figure 25 Photo of beam BC6 at load level 65 kN

3.2 Slabs

3.2.1 Reference slab (ST1)

Fatigue test and static test up to failure (after fatigue test)

Cracking load: about 200 kN ⇒ F_l = 200 kN

Number of load cycles: 2 000 000

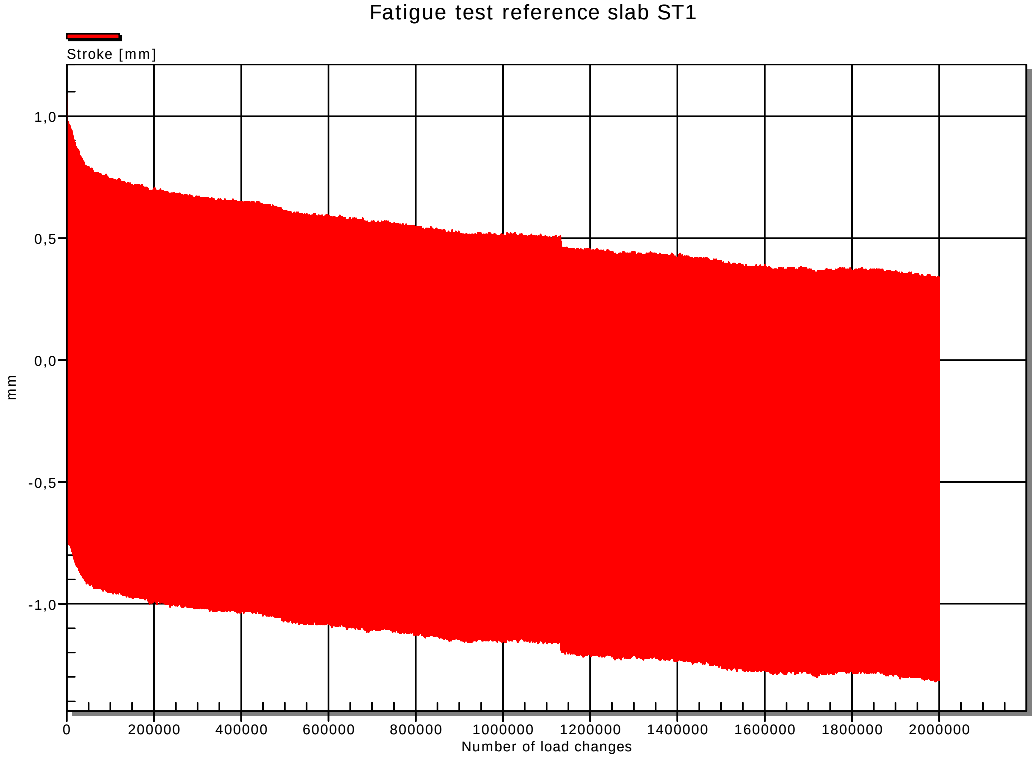

Figure 26 Diagram of deformation versus number of load changes for fatigue test of slab ST1 with traditional bar reinforcement.

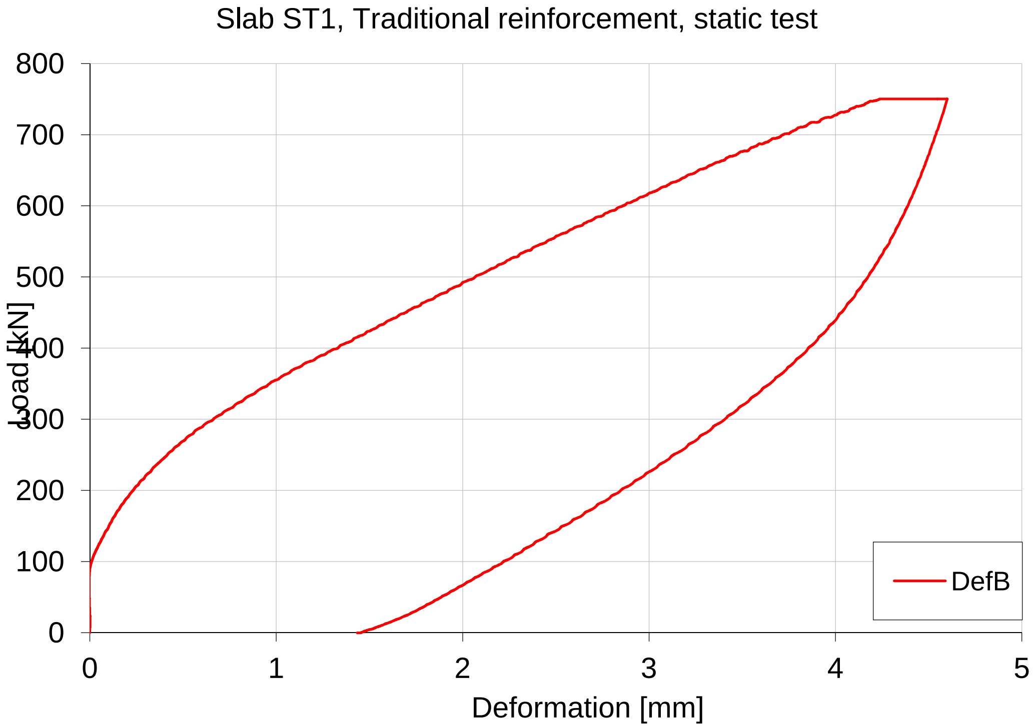

Figure 27 Load – deformation (in the middle) for static failure test of slab ST1.

3.2.2 Slabs with cell reinforcement (SC1, SC2 and SC3)

Slabs SC1 and SC2
Number of load cycles: 2 000 000

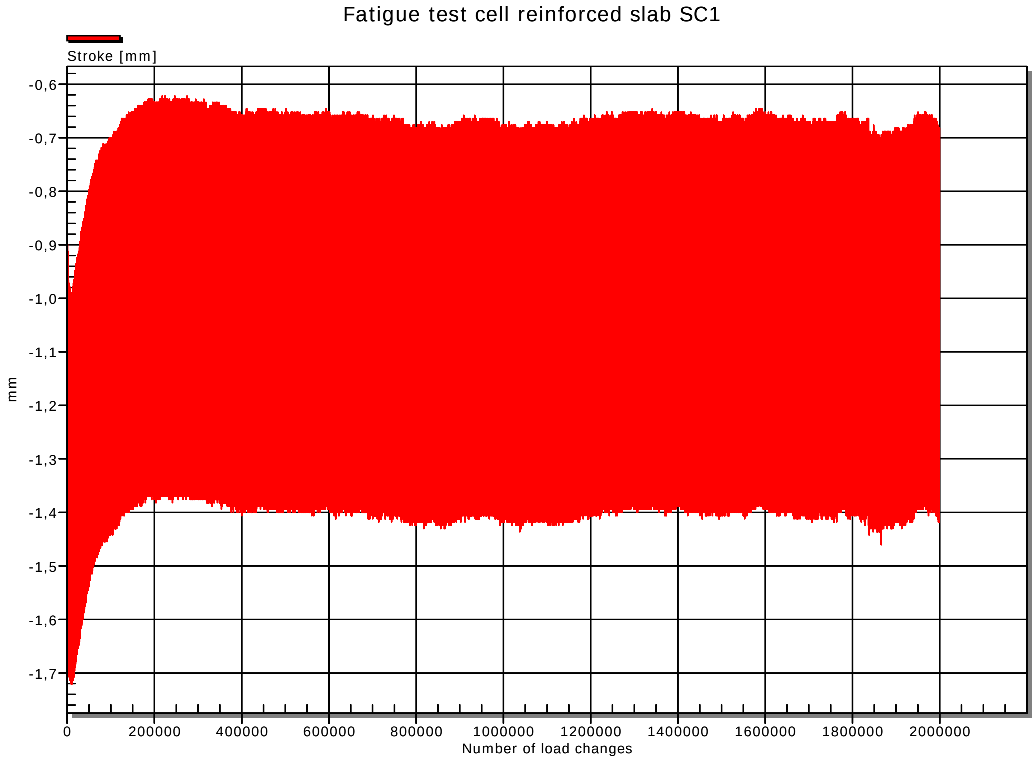

Figure 28 Diagram of deformation versus number of load changes for fatigue test of slab SC1 with cell reinforcement

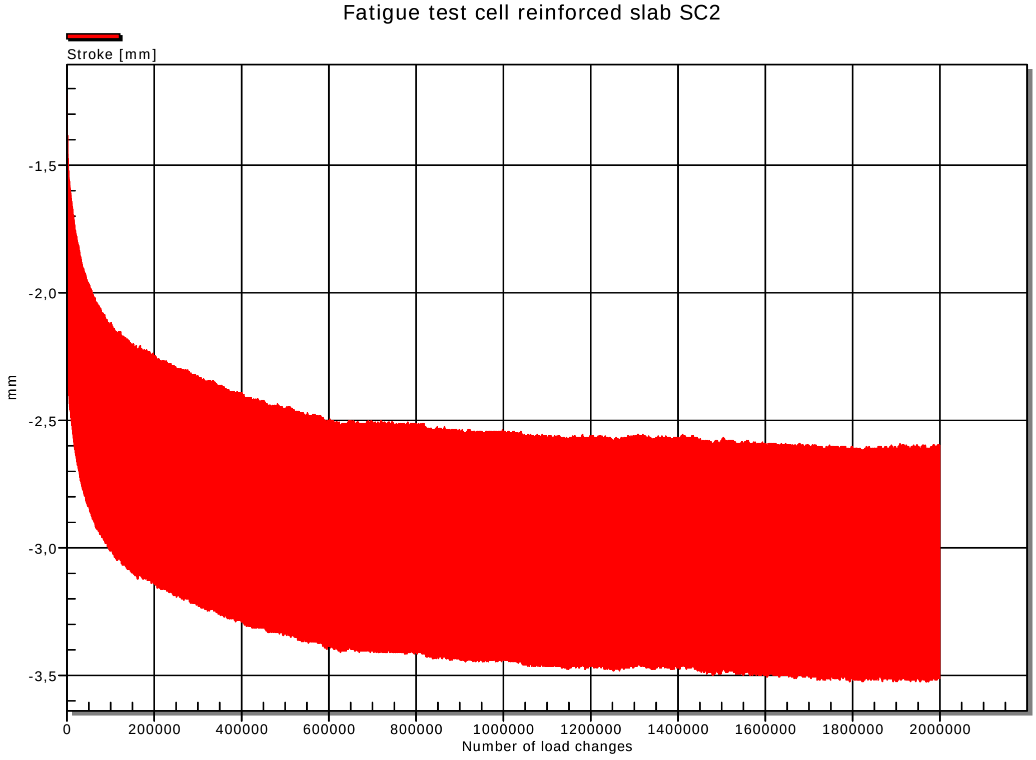

Figure 29 Diagram of deformation versus number of load changes for fatigue test of slab SC2 with cell reinforcement

Slab SC3
Cracking load: about 200 kN ⇒ F_I = 200 kN

Failure load: about 600 kN ⇒ F_{III} = 600 kN

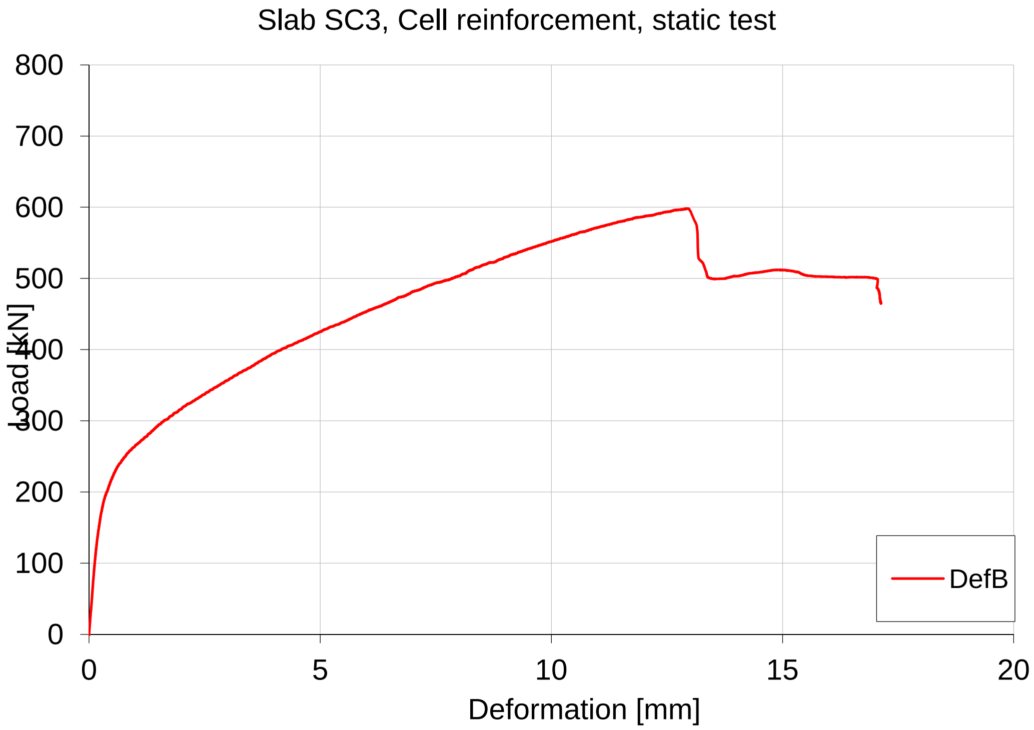

Figure 30 Load – deformation (in the middle) for static failure test of slab SC3.

4 Analysis and discussion

From the results presented above it is clear that the cell reinforced beams are not able to carry the theoretical static load, see Appendix 1, or to withstand the prescribed numbers of load. The difference in number of load cycles in the dynamic tests is so big that all beams were not tested. However, all the tested slabs had the capacity to carry the theoretical static load as well as the aimed number of load cycles. Why it differs between beams and slabs are right now not clear. In the beams the load is carried in one direction whereas in the slabs it is distributed over the whole slab. The reinforcement percentage is higher in the slabs than in the beams, see Table 1 and Table 2.

In Figure 31 the load deformations diagrams are shown for the static tests of the beams. The deformation is in the middle of the span. It is clear that the reference beam with traditional bar reinforcement is able to carry more load than the beams with cell reinforcement. The beams with cell reinforcement are more ductile. The cracking load is almost the same for all beams, though it is a bit larger for the reference beam. The beam with standing cell reinforcement (BC6) carried a bit smaller load than the beams with lying cell reinforcement (BC1 and BC2), which probably depends on that it has a smaller effective height.

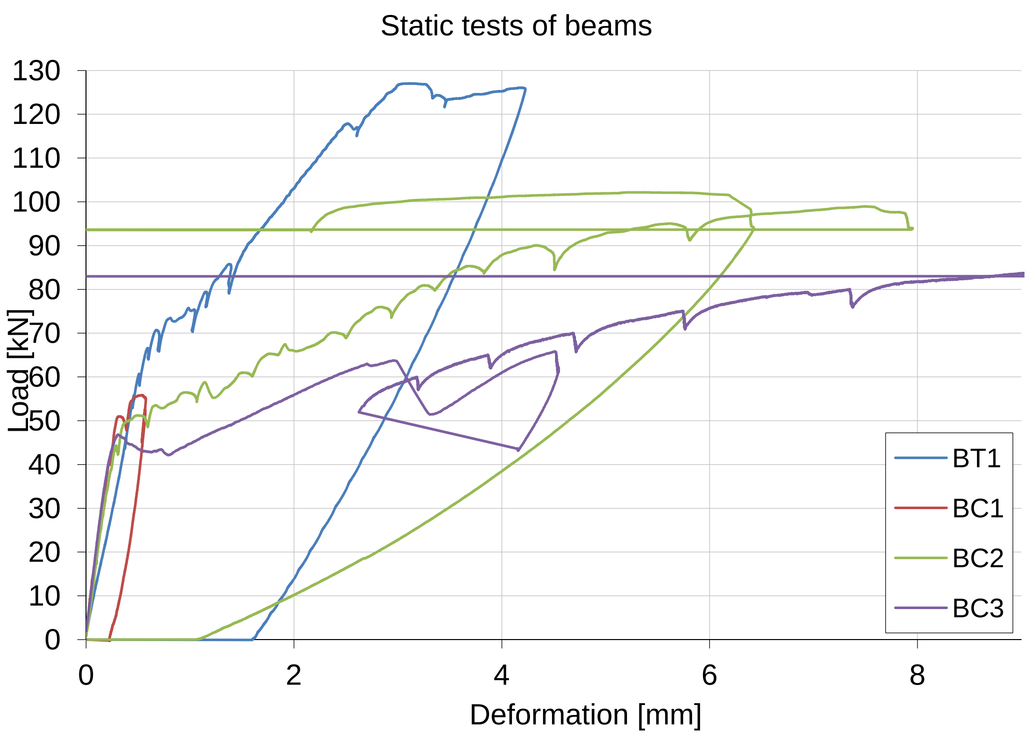

Figure 31 Load-deformation (middle – gage DefB) diagram for static tests of beams. BT1 with traditional reinforcement, BC1 and BC2 with lying cell reinforcement and BC6 with standing cell reinforcement.

In Figure 32 the load-deformation for the static tests of slabs ST1 (traditional reinforcement) and SC3 (cell reinforcement) is shown. The load capacity is higher for the slab with traditional reinforcement and the cell reinforced slab is more ductile. The same results are found for the beams. The cracking load is the same for both types of reinforcement.

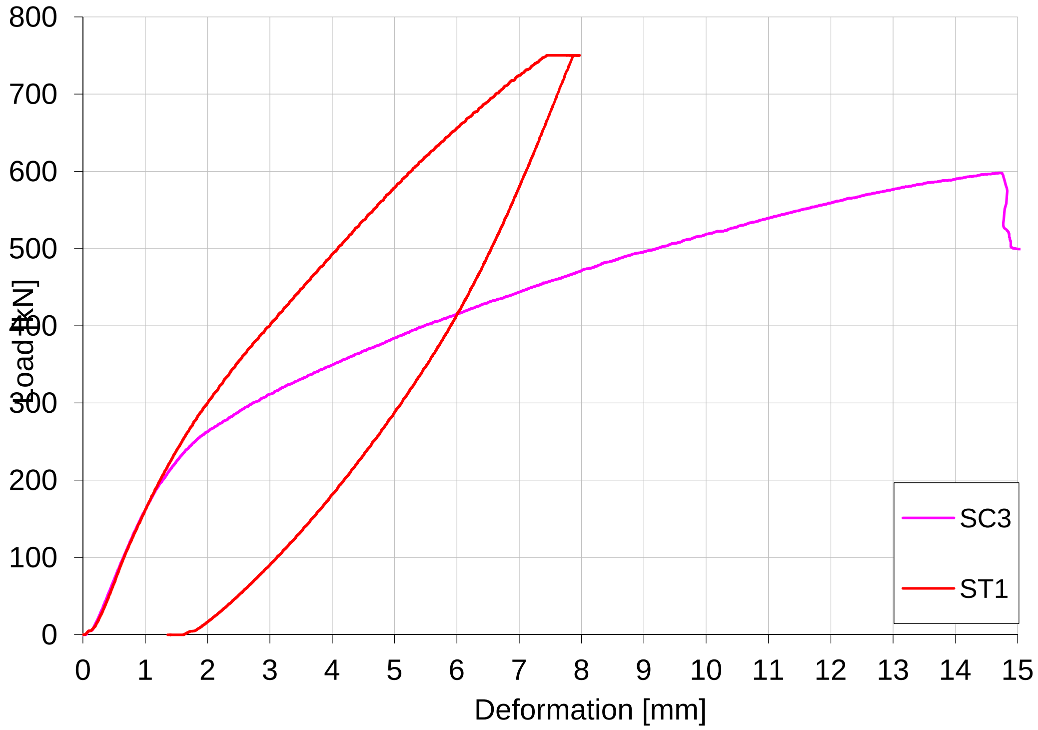

Figure 32 Load-deformation (middle – gage DefB) for static tests of slabs. ST1 with traditional reinforcement and SC3 with cell reinforcement.

Early within the project after the first tests were started it was clear that the strength of the concrete and the steel quality together with the design of the cell reinforcement have to be well correlated. If the concrete is too strong in relation to the cell reinforcement the fatigue capacity, in terms of number of load cycles, will be too low. When the concrete is too strong there will be just a few cracks larger cracks where the reinforcement will work. If the concrete is weaker (or the reinforcement stronger) there will be more and smaller cracks. The strength class of the concrete is for civil engineering structures, such as foundations for wind turbines, always about C35/45, and therefore it was used in the first tests. In combination with the available quality and configuration of the cell reinforcement it just does not work well. This implies that to design, build and test structural parts with cell reinforcement both under static and fatigue loading is a delicate task that demands much more research and development, which cannot be included within this project.

When loading a unit of cell reinforcement in tension the openings in the steel want to deform, see Figure 33. The concrete within the openings counteract this deformation. In order for a cell reinforced concrete element to work properly the steel must be able to crush the concrete in the openings letting the deformation of the element be distributed over a large distance. If the concrete is to strong or the steel plates to thick the concrete will not be crushed and all de deformations will be localised in a few places, implying high local stresses resulting in failure.

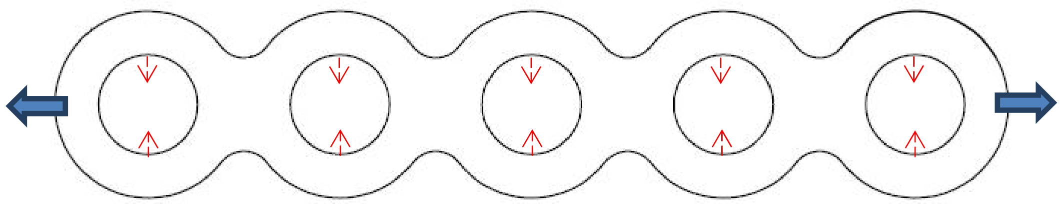

Figure 33 Deformation of cell reinforcement unit under tension.

The static tests resulted in lower capacity for the cell reinforced beams than for the reference beams with traditional bar reinforcement. This might to a certain extent dependent on the openings in the cell reinforcement naturally decrease the load carrying capacity. Through a section in an opening, usually called a local weakening, one gets stress concentrations closest to the opening that considerably exceed the mean stresses in the section, under elastic conditions. The stresses might be three times larger than the mean stresses. Therefore yielding will occur locally at the opening at relatively low loads.

A FEM model of a fourth of an opening and the steel in a row of cell reinforcement has been set up, see Figure 34. In the model the hole is filled with concrete and the dimensions of the steel and the hole can be varied as well as the yield strength of the steel and the quality of the concrete. The model is made and run in Matlab by Lars Bernspång at LTU. By applying a tensile force in the steel to the right the height of the steel ring will be smaller giving a tensile stress in the steel. The concrete will instead be under compression. Some simple results are shown below in Figure 34. The left figure shows the stresses in the *x* direction, i.e. the longitudinal direction; and the right figure shows the stresses in the *y* direction, i.e. the transverse direction. In the left figure it is clearly shown that high tensile stresses occur in the steel closest to the concrete.

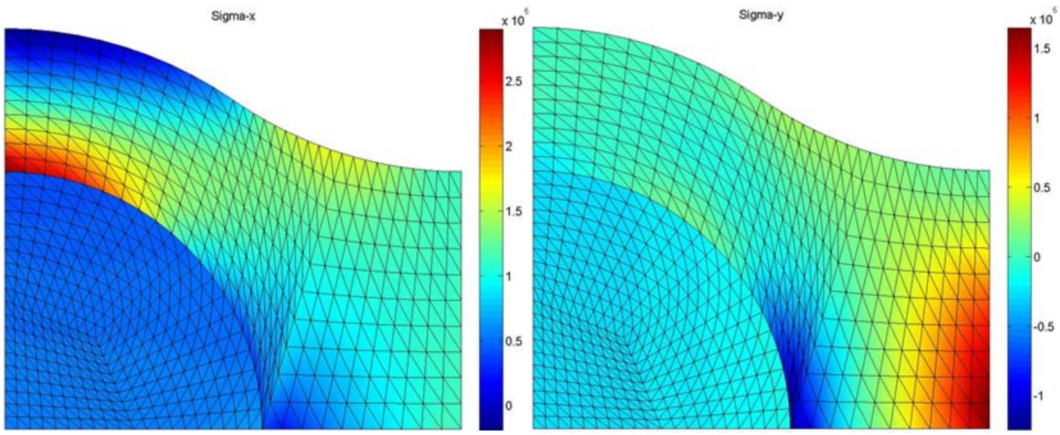

Figure 34 Results from a simple elastic FEM model of a fourth of a concrete filled hole and a steel ring under tensile loading to the right

By using Martensite steel in the cell reinforcement the fatigue capacity might probably increase. However the dimensions of the cell reinforcement must be changed to meet the quality of the concrete.

In a co-operation between Skanska, O2 Vindkompaniet and Jämtkraft a large wind farm with 30 wind turbines has been constructed on the mountain Sjisjka in Gällivare i Sweden. The farm has been built with foundations of prefabricated elements, see Figure 1, that have been cast together at site with small amounts of in-situ mixed concrete. The elements and the mounting system are well designed, and will probably be used in many future farms.

The material price of high strength steel in cell reinforcement is about 2-3 times higher compared to the material in traditional bar reinforcement.

5 Conclusions

- Statically the failure load of the cell reinforced beams was lower than for the reference beam with traditional bar reinforcement.
- The fatigue capacity in term of number of load cycles is too low in the beam elements with cell reinforcement.
- Both the static failure load capacity as well as the fatigue capacity (number of load cycles) is sufficient for slabs with cell reinforcement.
- The here tested combination of concrete and cell reinforcement did not work properly together regarding fatigue or static loading. The concrete was too strong compared to the dimensions of the cell reinforcement.
- Cell reinforcement might work well in statically loaded structures such as prefabricated slabs and walls.
- There are conditions for continued research and development for optimisation of the cell reinforced structure.
- The dimensions of the cell reinforcement must be changed to meet the quality of the concrete in civil engineering structures.

5.1 Proposed future studies

- Basic research is needed of the combination of cell reinforcement and concrete.
- Basic research and development is needed of design criteria and models for cell reinforced concrete.
- For structures under fatigue loading more theoretical studies are needed as well as further testing.
- Investigation if Martensite steel might increase the fatigue capacity of cell reinforcement.
- Cellfab is doing completing tests with cell reinforced beams and slabs with Martensite steel and re-designed dimensions (smaller) of the reinforcement. Results will soon be available.

6 References

DK200100030 (2001). *Stjernefundament med elementer til foundering af tårne*. Patent

SIC (2009). *http:/www.si-construction.com*. http://www.si-construction.com/files/billeder/Stjernefundament_komprimeret.jpg

Timoshenko, S. P. & Woinowsky-Krieger, S. (1959). *Theory of plates and shells*. Second edition. New York, U.S.A.: McGraw-Hill Book Company, Engineering Societies Monographs. pp. 580.

WO2004101898A2 (2004). *Foundation for a wind energy plant.* Patent

Sjijska (2012). *http://www.sjijskavind.se*. http://www.sjisjkavind.se/press-och-nyheter/bilder/20120508_113126/. 2012-10-24

Appendix 1 – Test specimen design

Cracking load, fatigue load levels and load capacity

The beams and slabs are under constant shear force between the supports and the applied loads equalling the value of the concentrated loads.

The beams are exposed to a constant moment between the concentrated loads, equalling the value of the concentrated loads multiplied by the distance between the supports and the concentrated loads, see Figure 35. The slabs are almost exposed to a constant moment under the load plate.

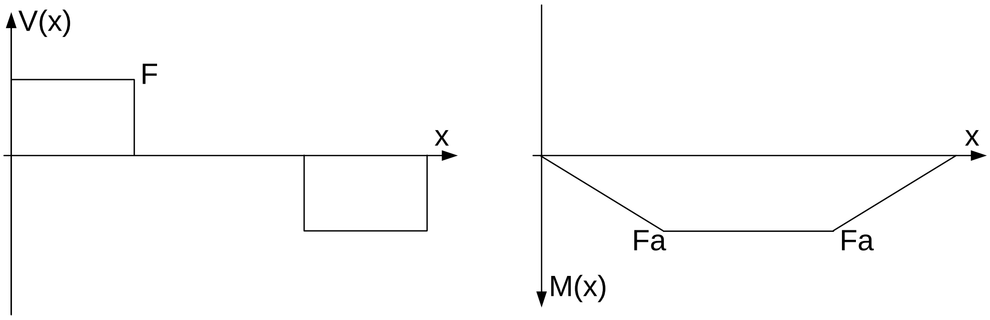

Figure 35 Shear force and moment variations along the beams.

Cracking load (stadium I)

The beams and slabs will crack in bending when the normal stresses of the bending moment are larger than the flexural tensile strength of the concrete. Navier's equation

$$M_I = \frac{I_I}{e_I} f_{ctm,fl}$$

where M_{cr} is the cracking moment, I_I is the moment of inertia in stadium I (un-racked cross section), e_I is the distance from the centre of gravity to the bottom surface, and $f_{ctm,fl}$ is the flexural tensile strength of the concrete.

The moment of inertia is determined by the parallel-axis theorem by firstly calculating the cross-section area A_I and its centre of gravity y_I from the top surface and then the moment of inertia

$$A_I = bh + \left(\frac{E_s}{E_c} - 1\right) A_s$$

$$y_I = \frac{bh\frac{h}{2} + \left(\frac{E_s}{E_c} + 1\right) A_s d}{A_I}$$

$$I_I = \frac{bh^3}{12} + bh\left(\frac{h}{2} - y_I\right)^2 + \left(\frac{E_s}{E_c} - 1\right)A_s\left(d - y_I\right)^2$$

where b is the width and h is the height of the cross-section, respectively, E_s is Young's modulus of the reinforcement, E_c is Young's modulus of the concrete, A_s is the cross section area of the reinforcement and d is the effective height.

The distance from the centre of gravity to the bottom of the cross section is

$$e_I = h - y_I$$

The flexural tensile strength of the concrete, SS-EN 1992-1-1:2005 3.1.8 (3.23),

$$f_{ctm,fl} = \max\begin{cases}\left(1,6 - \dfrac{h}{1000}\right)f_{ctm} \\ f_{ctm}\end{cases}$$

where f_{ctm} is the mean axial tensile strength of the concrete in MPa.

The cracking load for the beams is then calculated from the moment distribution, see Figure 35.

$$F_I = \frac{M_I}{a}$$

where a = 325 mm is the distance between the supports and the concentrated loads.

The cracking load for the slabs is in turn calculated by the maximum moment according to elastic theories for plates, see Timoshenko & Woinowsky-Krieger (1959):

$$F_I = \frac{4\pi \cdot M_I}{(1+\nu)\log\dfrac{r}{c} + 1 - \dfrac{(1-\nu)c^2}{4r^2}}$$

where r is the radius of the support under the slab, c is the radius of the loaded surface and ν = 0.2 is Poisson's ratio for the concrete.

The calculations for the reference beams and the beams with cell reinforcement is presented below in Table 5.

Table 5 Calculation of cracking loads for beams.

	Reference beams	"Lying" cell reinforcement	"Standing" cell reinforcement
Input			
b [mm]	310	310	310
h [mm]	200	200	200
a [mm]	325	325	325
E_c [GPa]	34	34	34

f_{ctm} [MPa]	3,20	3,20	3,20
E_s [GPa]	200	210	210
A_s [mm²]	235,6	180	180
d [mm]	180	184	147,5
Calculations			
A_I [mm²]	63150	62932	62932
y_I [mm]	101,5	101,2	100,7
I_I [mm^4]	$2,139 \cdot 10^8$	$2,131 \cdot 10^8$	$2,087 \cdot 10^8$
e_I [mm]	98,5	98,8	99,3
$f_{ctm,fl}$ [MPa]	4,48	4,48	4,48
M_I [kNm]	9.72	9,67	9,42
F_I [kN]	**29,9**	**29,8**	**29,0**

Table 6 Calculation of cracking loads for reference slabs.

Input		
b [mm]	1200	
h [mm]	200	
c [mm]	300	
r [mm]	1100	
E_c [GPa]	34	
f_{ctm} [MPa]	3,20	
E_s [GPa]	200	
∅ [mm]	10	18
n_s [pieces]	12	12
A_s [mm²]	785.4	1696.5
d [mm]	168.0	179.0
Calculations		
A_I [mm²]	223835	228283
y_I [mm]	101.2	102.9
I_I [mm^4]	$7.508 \cdot 10^8$	$7.832 \cdot 10^8$
e_I [mm]	98.8	97.1
$f_{ctm,fl}$ [MPa]	4.48	
M_I [kNm]	34.03	36.12
F_I [kN]	**297.7**	**316.0**

Table 7 Calculation of cracking loads for cell reinforced slabs.

Input				
b [mm]	1200		1200	
h [mm]	200		200	
c [mm]	300		450	
r [mm]	1100		1100	
E_c [GPa]	34		34	
f_{ctm} [MPa]	3,20		3,20	
E_s [GPa]	200		200	
A_s [mm²]	475.0	1250.0	475.0	1250.0
d [mm]	147.5	147.5	147.5	147.5

Calculations				
A_I [mm²]	222459	226471	222459	226471
y_I [mm]	100.5	101.4	100.5	101.4
I_I [mm^4]	$7.388 \cdot 10^8$	$7.475 \cdot 10^8$	$7.388 \cdot 10^8$	$7.475 \cdot 10^8$
e_I [mm]	99.5	98.6	99.5	98.6
$f_{ctm,fl}$ [MPa]	4.48		4.48	
M_I [kNm]	33.27	33.95	33.27	33.95
F_I [kN]	**291.1**	**297.0**	**327.4**	**334.1**

Fatigue load amplitude (stadium II)

In the fatigue tests the beams and slabs are loaded so that the stresses in the reinforcing steel vary between 25 and 75 % of the yield strength. The bending moment in a cracked cross section of rectangular beam (and assume also in a slab) is

$$M_{II} = \frac{\sigma_c x_{II}}{2} b \left(d - \frac{x_{II}}{3} \right)$$

where σ_c is the stress in the concrete and x_{II} is the height of the compressed zone in stadium II, determined as

$$x_{II} = \frac{E_s}{E_c} \frac{A_s}{b} \left(\sqrt{1 + \frac{2}{\frac{E_s}{E_c} \frac{A_s}{bd}}} - 1 \right)$$

The stress in the concrete is calculated as

$$\sigma_c = \frac{\sigma_s E_c}{E_s} \frac{x_{II}}{d} \left(1 - \frac{x_{II}}{d} \right)$$

The load in stadium II is then determined in the same way as the cracking load, for the beams

$$F_{II} = \frac{M_{II}}{a}$$

and for the slabs

$$F_{II} = \frac{4\pi \cdot M_{II}}{(1+\nu)\log\frac{r}{c} + 1 - \frac{(1-\nu)c^2}{4r^2}}$$

Table 8 Calculation of fatigue load amplitude for beams.

	Reference beams	”Lying” cell reinforcement	”Standing” cell reinforcement
Input			
b [mm]	310	310	310
a [mm]	325	325	325
E_c [GPa]	34	34	34

E_s [GPa]	200		210		210	
f_y [MPa]	500		1000		1000	
A_s [mm²]	235.6		180		180	
d [mm]	180		184		147.5	
Calculations						
x_{II} [mm]	35.9		32.9		29.1	
σ_s [MPa]	125	375	250	750	250	750
σ_c [MPa]	5.29	15.88	8.82	26.46	9.96	29.89
M_{II} [kNm]	4.9	14.8	7.8	23.4	6.2	18.6
F_{II} [kN]	**15.2**	**45.7**	**24.0**	**71.9**	**19.1**	**57.2**

Table 9 Calculation of fatigue load amplitude for reference slabs.

Input				
b [mm]	1100			
E_c [GPa]	34			
E_s [GPa]	200			
f_y [MPa]	500			
∅ [mm]	10		12	
n_s [pieces]	12		18	
A_s [mm²]	784.5		1696.5	
d [mm]	168.0		179.0	
Calculations				
x_{II} [mm]	33.6		48.6	
σ_s [MPa]	125	375	125	375
σ_c [MPa]	5.3	14.7	7.9	23.8
M_{II} [kNm]	15.4	45.7	23.1	64.1
c [mm]	300			
F_{II} [kN]	**135**	**400**	**202**	**561**

Table 10 Calculation of fatigue load amplitude for cell reinforced slabs.

Input				
b [mm]	1100			
E_c [GPa]	34			
E_s [GPa]	200			
f_y [MPa]	500			
A_s [mm²]	475		1250	
d [mm]	147.5		147.5	
Calculations				
x_{II} [mm]	25.5		39.0	
σ_s [MPa]	250	750	250	750
σ_c [MPa]	8.46	25.39	14.56	43.68
M_{II} [kNm]	16.51	49.52	42.03	126.09
c [mm]	300			
F_{II} [kN]	**144**	**433**	**368**	**1103**
c [mm]	450			

F_{II} [kN]	**162**	**487**	**414**	**1241**

Failure load (stadium III)

The ultimate load capacity of the beams (stadium III) is calculated according to traditional design for bending and for shear.

The moment capacity in a normally reinforced cross section is

$$M_{III} = f_y A_s \left(d - 0,4 x_{III}\right)$$

where f_y is the yield strength of the reinforcement and x_{III} is the height of the compressed zone in the concrete, determined as

$$x_{III} = \frac{f_y A_s}{0,8 f_c b}$$

where f_c is the compressive strength of the concrete.

To check if the cross section is normally reinforced the strain in the reinforcement must be calculated and compared to the yield strength of the steel, i.e. if the strain in the reinforcement ε_s is larger than the yield strength ε_{sy}. The strain in the reinforcement

$$\varepsilon_s = \varepsilon_{cu} \frac{d - x_{III}}{x_{III}} = 3,5 \frac{d - x_{III}}{x_{III}}$$

and the yield strain (Hooke´s law)

$$\varepsilon_{sy} = \frac{f_y}{E_s}$$

Again, the ultimate load is determined as before

$$F_{III} = \frac{M_{III}}{a}$$

For the slabs the failure load is determined according to the yield line theory. In Figure 36 a slab is shown loaded by a uniformly distributed circular load and supported o a circular support. The slab is two-way reinforced with the moment capacity m_f.

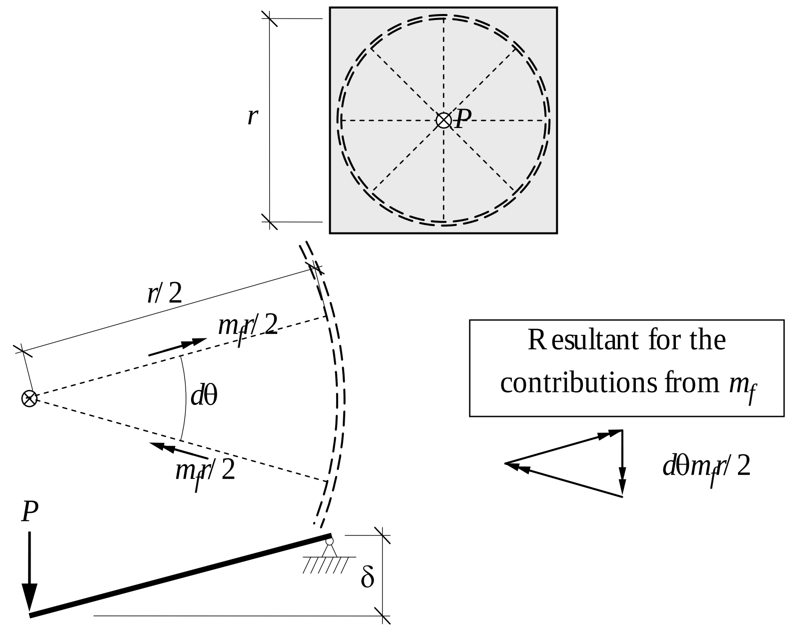

Figure 36 Circular load on slab with circular yield line pattern.

External work

$$W_y = P\delta$$

The internal work can be determined by studying a piece of the cracked slab with the angle $d\theta$, see Figure 36, and integrate over the angle 2π giving

$$W_i = \int_0^{2\pi} dW_i = \delta m_f \int_0^{2\pi} d\theta = 2\pi\delta m_f$$

The external and the internal work shall be equal, giving

$$P = 2\pi m_f$$

Table 11 Calculation of failure load (stadium III) for beams

	Reference beams	”Lying” cell reinforcement	”Standing” cell reinforcement
Input			
b [mm]	310	310	310
a [mm]	325	325	325
E_c [GPa]	34	34	34
E_s [GPa]	200	210	210
f_y [MPa]	500	1000	1000
A_s [mm²]	235,6	180	180
d [mm]	180	184	147,5
Calculations			
x_{III} [mm]	13,6	20,7	20,7
ε_s [‰]	42,9	27,6	21,4
ε_{sy} [‰]	2,5	4,8	4,8

M_{III} [kNm]	20,6	31,6	25,1
F_{III} [kN]	**63,3**	**97,3**	**77,1**

Table 12 Calculation of failure load (stadium III) for slabs

	Reference slab		Cell reinforced slab	
Input				
E_c [GPa]	34		34	
f_c [MPa]	35		35	
E_s [GPa]	200		210	
f_y [MPa]	500		1000	
A_s [mm²]	785.4	1696.5	475.0	1250.0
d [mm]	168.0	179.0	162.5	147.5
Calculations				
x_{III} [mm]	14.02	30.29	16.96	44.64
ε_s [‰]	38.43	17.18	26.93	8.06
ε_{sy} [‰]	2.50		4.76	
m_f [kNm/m]	63.77	141.55	66.84	162.05
m_{III} [kNm/m]	102.66		114.45	
F_{III} [kN]	**645.0**		**719.1**	

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