Prestressed BFRP tendons in concrete beams

by

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i. Ágrip

Titill á íslensku: Steyptir bitar með forspenntum BFRP teinum

Í steinsteypt mannvirki eru yfirleitt notaðir stálteinar til styrkingar. Í umhverfi þar sem mikil hætta er á tæringu, t.d. þar sem mikill raki er eða önnur efnaáraun, þá hentar hefðbundið stál ekki vel. Í þesshátttar aðstæðum hefur notkun á trefjastöngum (FRP) verið reynd víða um heim með ágætum árangri. Basalttrefjastangir (BFRP) eru hinsvegar frekar nýjar á markaðnum. Togstyrkur BFRP stanga er u.þ.b. tvöfaldur á við hefðbundið stál en fjaðurstuðullinn er aðeins 40-50 GPa meðan fjaðurstuðull stáls er um 200 GPa. Elastísk lenging BFRP teina verður því mun meiri en stálteina, sem veldur þá allt of miklum togsprungum í steypunni ef reynt er að nýta togstyrk BFRP teina til jafns á við steypustyrktarteina úr stáli. Með því að forspenna BFRP teina er hægt að nýta hærra hlutfall af togstyrk þeirra áður en steypan byrjar að springa.

Í þessari ritgerð eru niðurstöður tilraunar á forspenntum BFRP teinum kynntar. Helstu þættir þessarar rannsóknar voru að forspenna BFRP teina sem styrkingu í steyptum bitum, með það að markmiði að líkja eftir forspenntum plötueiningum og áætla hve hátt hlutfall forspennukraftsins tapast til langs tíma. Fjórir bitar voru steyptir, þrír þeirra forspenntir og einn óspenntur. Streitan í forspenntu teinunum var mæld í um þrjár vikur, eða frá því byrjað var að strekkja og þar til bitarnir voru brotnir. Leitað var eftir sambærilegum rannsóknum við erlenda háskóla og rannsóknamiðstöðvar til að geta borið niðurstöður þessarar rannsóknar saman við viðeigandi tilrauna- og reikniaðferðir.

Helstu niðurstöður voru þær að vel er hægt að forspenna BFRP teina og nota þá til styrkingar steypu. Í ljós kom að þörf er á að þróa betri festibúnað fyrir FRP teina. Stífni og burðargeta bitanna jókst borið saman við óspenntan bita. Langtíma spennulosun í teinunum er áætluð 20%. Gæta skal sérstakrar varúðar þegar skerþol bita eða platna með FRP langjárnum er reiknað, þar sem skerþol er minna ef FRP teinar eru notaðir en ef stálteinar eru notaðir.

Lykilorð: Basalttrefjar, BFRP, forspenna, steyptir bitar, tilraun

ii. Abstract

Concrete structures are normally reinforced with steel tendons. In marine or chemical environment steel has its limitations. Replacing steel with FRP reinforcement has been practiced for many years but using basalt fiber reinforcement polymer tendons (BFRP) as a structural material is rather new. Tension strength of BFRP tendon is about twice the tension strength of steel reinforcement but the elastic modulus is only 40-50 GPa while steel has 200 GPa. Therefore elastic lengthening of BFRP tendons is much more than of steel and strain in tension zone of concrete structures will be over acceptable limits. To utilise the high tension strength of BFRP and prevent too much cracking of concrete, the tendons could be prestressed.

This thesis presents results of an experimental work. The main topics of this experimental work are to use prestressed BFRP tendon as reinforcement in concrete beam, simulating precasted slabs for example, and try to estimate the relaxation of the prestress force in the tendon. Four beams were casted, three of them with prestressed BFRP tendons and one not prestressed. To study the behaviour of the prestressed tendons the strain was measured over a period of approximately three weeks. Then the beams were broken under four point bending test. Relevant literature was reviewed to compare calculations and testing procedure with the experimental work.

The main findings were that it is possible to prestress BFRP tendons, but because of low transverse strength anchoring the tendons can be difficult. The stiffness and bearing capacity of the beam did increase relative to un-prestressed beam. Long-term relaxation of prestressed BFRP tendons is estimated close to 20%. Special care is needed while calculating shear strength of beams with longitudinal FRP reinforcement.

Keywords: Basalt fiber, BFRP, prestress, concrete beam, experimental work

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Finally, I thank my family and friends for their support through my M.Sc study and especially my wife, Ásrún Ösp and son, Anton Dagur.

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Symbols

Roman upper case letters

A_s	Cross-sectional area of longitudinal steel reinforcement
A_f	Cross-sectional area of longitudinal fiber reinforcement
$C_{Rd,c}$	EC2 - recommended value $0.18/\gamma_c$
E_c	Modulus of elasticity of concrete
E_f	Modulus of elasticity of FRP
E_s	Modulus of elasticity of steel
F_f	Force developed in a FRP bar
F_{ult}	Ultimate force
M_{ult}	Ultimate moment
V_c	Nominal shear strength provided by concrete with steel flexural reinforcement
V_{cf}	Nominal shear strength provided by concrete with FRP flexural reinforcement
V_{frp}	Shear resistance provided by FRP stirrups
V_n	Nominal shear strength of a reinforced concrete cross section
V_p	Vertical component of prestressing force
$V_{Rd,c}$	Design shear resistance of the member without shear reinforcement
$V_{Rd,ct}$	Concrete contribution to the shear capacity
V_{req}	Required shear strength

Roman lower case letters

b and b_w	Width of rectangular cross-section
c	Distance from extreme compression fiber to neutral axis,
d	Effective depth of a cross-section
e	Eccentricity
f_c	Compressive strength of concrete
f_c'	Specified compressive strength of concrete (ACI)
f_{ck}	Characteristic compressive cylinder strength of concrete at 28 days
f_y	Yield strength of reinforcement
h	Height of rectangular cross-section
k	Factor see EC 2, chapter 6.2.2 and ACI 440.1R-06 ratio of the depth of the neutral axis to the reinforcement depth
k_1	Factor equal to 0.15 according to EC 2
x	Neutral axis depth
z	Lever arm

Greek letters

β_1	Factor taken as 0.85 for concrete strength f_c up to and including 4000 psi. For strength above 4000 psi, this factor is reduced continuously at a rate of 0.05 per each 1000 psi of strength in excess of 4000 psi, but is not taken less than 0.65
γ_c	Partial factor for concrete
ε_{c1}	Compressive strain in the concrete at the peak stress f_c
ε_{cc1}	Compressive strain in the concrete at the peak stress f_{cc1}
ε_{cu}	Compressive strain in the concrete at the confined compressive stress f_{cc}
ε_f	Ultimate tensile strain of FRP
ε_y	Tensile strain in steel at yield stress
η	Factor defining effective strength of concrete
λ	Modification factor reflecting the reduced mechanical properties of lightweight concrete
λ	Factor defining effective height of compression zone
ν_{min}	See EC2 chapter 6.2.2
ξ	Ratio of the neutral axis depth to the effective depth
ρ_f	Reinforcement ratio for FRP
ρ_l	Reinforcement ratio for longitudinal reinforcement
ρ_s	Reinforcement ratio for longitudinal steel reinforcement
σ_{cp}	Compressive stress in the concrete from axial load or prestressing
τ_{rd}	Design shear stress
φ_ε	Strength reduction factor, represent ratio of allowed FRP and steel strain

Abbreviations

FRP	Fiber reinforced polymer
AFRP	Aramid fiber reinforced polymer
BFRP	Basalt fiber reinforced polymer
CFRP	Carbon fiber reinforced polymer
GFRP	Glass fiber reinforced polymer
SLS	Serviceability limit state
ULS	Ultimate limit state

1 Introduction

1.1 Background

Reinforcing concrete members such as beams and slabs is necessary for carrying capacity of the member due to almost zero tension resistance of the concrete. Steel reinforcement in concrete structures is well known, and it is also known that it has limitations. Especially in marine environments and chemical plants, where corrosion resistance is a major concern (Sim, Park, & Moon, 2005). To prevent the steel from oxidise, the concrete cover must be quite big to prevent water and air to reach the steel and the concrete must not crack very much. It is well known to use some waterproofing materials on the surface to minimise the amount of water and air that can reach the steel through cracks. Waterproof materials are often expensive and need maintenance several times over the structures lifetime. Although concrete cover has almost no advantages for structures bearing capacity, it is necessary for fire resistance and for environmental conditions. It is worth exploring if there is available method and material to prevent or minimize these negative effects.

Fiber reinforced polymer (FRP) tendons have been investigated and used instead of steel for about 50 years (Bank, 2006). There are available several types of FRP reinforcing tendons. The purpose of this experimental work is to investigate prestressed basalt fibers reinforced polymer (BFRP) tendons as reinforcement for concrete structures.

Basalt fibers are made by melting basalt rock. This melted rock is then divided into small particles and from them fibers are manufactured. There are no additives in this procedure so it lowers the manufacturing cost. Basalt fibers have high strength and thermal stability (Bashtannik, Kabak, & Yakovchuk, 2003; Sim et al., 2005). The temperature interval of application of basalt fibers range from -270°C up to 700°C-900°C (N. N. Morozov et al., 2001). Basalt fibers have quite good chemical stability (Wei, Cao, & Song, 2010). If it is possible to reinforce concrete with material that does not oxidise and has high heat resistance it may replace steel reinforcement in some cases and perhaps it is possible to decrease the cover. FRP tendons have different mechanical properties than steel and therefore: *“Special care is required in the case of structural analysis, where the almost complete lack of ductility of the FRP reinforced concrete structures shall taken into account”* (*CNR-DT 203*, 2007, p. 10).

Prestressed concrete is one form of reinforced concrete, the compressive force is applied to a member through a tensioned steel tendon – or FRP tendon - which is anchored at the ends or have good bond to the concrete. This compression due to prestress causes stresses that reduces or nullifies the tensile stress in concrete which is caused by bending due to applied load (O´Brian & Dixon, 1995). If there is no tensile stress in the concrete the cross section will not crack and oxidation will not occur and bearing capacity will increase.

Several research studies on fibre reinforcement polymer (FRP) have been conducted at The Reykjavik University in collaboration with The Innovation Center Iceland. Following is a list of these reports and dissertations:

- Ester Rós Jónsdóttir and Grettir Adolf Haraldsson, 2007. Prófanir á bitum bentum með FRP stöngum ásamt viðeigandi steypuhönnun og kostnaðarathugunum (in Icelandic).
- Hannibal Ólafsson and Eyþór Þórhallsson, 2009. Rannsókn á styrk trefjastanga í steyptum þversniðum (in Icelandic).
- Hannibal Ólafsson and Eyrún Gestsdóttir, 2009. Steinsteyptir bitar styrktir með basaltstöngum (in Icelandic).
- Hannibal Ólafsson and Eythor Thorhallsson, 2009. Basalt fiber bar. Reinforcement of concrete structures.
- Rakel Magnúsdóttir, 2010. Basalttrefjar og basalttrefjastangir (in Icelandic).
- Eva Lind Ágústsdóttir and Sólrún Lovísa Sveinsdóttir, 2010. Prófanir á basalttrefjastöngum og basalttrefjamottum (in Icelandic).
- Ásdís Söebeck Kristjánsdóttir, 2010. Brunapróf steypu íblandaðri basalttrefjum (in Icelandic).
- Arngrímur Konráðsson, 2011. Experimental Research on BFRP Confined Concrete Columns.

The main findings are that FRP tendons are promising for reinforcing concrete members. Basalt fiber tendons (BFRP) have tensile strength about 1000 MPa. In comparison regular steel reinforcement has tensile strength around 500 MPa. Research on basalt sheets as a confinement material for strengthening older structures gave very promising results. This experimental study is in a sequel of these former studies where usages of basalt fibers as a reinforcing material for concrete structures are giving promising results and is a very interesting option especially here in Iceland. It could be possible to produce basalt fibers in Iceland, the main elements that are needed is basalt (rock) which is everywhere on the ground and cheap electricity for the melting process - and Iceland has a lot of sustainable energy. So maybe there could be an Icelandic BFRP reinforcement in the future. Still there is a long way to go with more research and testing to be done.

1.2 Statement of the problem

Basalt fiber bars have a rather low modulus of elasticity, which is only $^{1}/_{5}$ of the modulus of elasticity for steel. This fact is one of the reasons this experiment is taking place.

Because of this low elastic modulus, about 40-50 GPa, deflection of the beam will be over acceptable limits and the tensile strain in the concrete will be over acceptable limits and that will lead to very much cracking if beams are reinforced with BFRP without prestress. That is, if it is desired to utilise the high tension strength of BFRP. To prevent the cracks and use more of basalts fiber strength, prestressing the tendons is an optimal choice. By prestressing the concrete beam the whole cross section will be in compression in the serviceability limit state and the concrete will be without cracks. That increases it´s durability and load bearing capacity.

1.3 Aim of this research

The aim in this experiment is to test concrete beam reinforced with BFRP tendons and prestress that reinforcement tendons, to simulate for example precasted concrete slabs which are often reinforced with prestressed steel tendons. It is important to gather knowledge about the function of BFRP prestress tendons and how the function is if it is used as prestressing tendons for reinforced concrete members.

The main topic that will be investigated in this experimental work:

- Test ultimate force resistance of concrete beam reinforced with prestressed BFRP tendons.
- Estimate the relaxation of the prestressed BFRP tendons

1.4 Research methodology

This thesis is both a literature review and a report of the experimental work. The purpose of the literature overview is to see whether there is some similar research and it appears there is not much available research about BFPR and prestress. The review is therefore about characteristics of FRP in wider field, the mechanism of prestress and the two failure modes; shear and flexure. This summarizes the knowledge previously gathered and calculation methods from the literature can be used to estimate what result could be expected from the experiment.

The experimental work was extensive and is therefore quite well documented how it was done and what needed to be done in the preface. Then the procedure of the experimental work itself and the results are reported. The beams were tested under four-point bending.

The search for relevant literature was both conducted online and in the library at Reykjavik Uninversity. Web search was mainly in databases that are available for students at Reykjavík University, such as Web of Science, CSA Illumina, El Village, ScienceDirect and Whiley Online Library. Google Scholar was also used a lot and SpringerLink. The keyword used was for example; basalt, fiber and fibre, BFRP, FRP, BF (basalt fiber), prestress and pre-stress, tendon, rod, bar, rebar and strand, shear strength and flexural strength. It is inconvenient that there are so many names used for the same thing and it is also inconvenient that BFRP can both stand for basalt FRP and bamboo FRP.

There were limitations in the experimental work. Time frame of the work did not give a change to do many rounds of casting and breaking beams. The available tendons were also limited and the space for prestressing equipment was limited.

1.5 Scope of the work

This experimental work started in January 2011, this type of experiment has not been done before in this area so in the beginning a lot of equipment adjustments were needed.

In the beginning of February the setup for prestress and casting was ready so then the beams were casted. It was given about 20 days to harden. In the meantime the equipment to break the beam was adjusted to this work.

At the end of February the beams was broken and the work of analysing the data and writing this thesis could start.

The thesis is divided into chapters as follows:

- Chapter 1 is an introduction to the work and research aim is stated.
- Chapter 2 is a literature review where properties of FRP and prestress theory is viewed. The shear strength and flexural strength of members is also examined.
- Chapter 3 is about the experimental work and preparation of the experiment.
- Chapter 4 the results are revealed and discussed
- Chapter 5 is a discussion about the whole work
- Chapter 6 is a summary and conclusion.
- Chapter 7 is a recommendation for further research.
- Finally there comes bibliography and appendixes where technical data about the BFRP used in this experiment can be seen with other technical sheets and calculations.

Expected end of this work is in end of May 2011 and graduation is 18. of June 2011.

2 Literature review

2.1 Introduction

In this chapter a review of relevant literature will be introduced. First will be a review of fiber reinforced polymer (FRP) in general – and there is mainly referred to carbon, glass and aramid, then are especially findings about basalt fibers (BFRP). Then there is some review of prestressing methods. Shear strength and flexural strength is then viewed. Use of FRP composite material for strengthen and reinforce concrete is mainly done in three ways. First is externally bonded FRP which is either sheets or tendons (often square tendons), second is applying near surface mounted reinforcement (NSMR) which is often square tendons, third is internal reinforcement that is used instead of ordinary steel reinforcement. External fitting and NSMR is mainly used for retro fitting.

The main topic of this experimental research is to investigate prestressed BFRP tendons as an internal reinforcement. An exhaustive survey to find literature about this particular topic did not give many results. The best search results when looking for BFRP and prestressing tendons was from Chinese studies that are published online by CNKI which stands for *China National knowledge infrastructure* and is a key national online publishing project of China. There is though one big problem, all the studies found there are only published in Chinese. But because it seems to be the only literature found that is exactly about the same topic as this thesis, *prestressing BFRP tendons for reinforcing concrete beams,* an abstract of two Chinese papers will be introduced.

> *"Basalt fiber reinforced plastics (BFRP) is a kind of new compound material with high performance. In order to study the application of BFRP tendons on prestressed force structure, first of all, the fundamental theory of nonlinear finite element method used in concrete structure is discussed in this paper. Then ANSYS is used to analyze both the prestressed and non-prestressed concrete beam with BFRP tendons. According to the results such as deflection-load curve, characteristic load value and crack condition, the prestressed beam is compared with non-prestressed one and the result shows that the prestressed concrete beam with BFRP tendons has higher resisting cracking degree and stiffness degree, but has relatively poorer deformability than the non-prestressed one. Under the situation of equal tendon rates, the ultimate bearing capacity is almost equal to that of non-prestressed beam".* (Gan Yil, Jiang, Fei Weil, Sun, & Li Bing, 2009)

And the latter abstract seems to be from research that is very similar as the one that are published here in this thesis, unfortunately it was not available for review in English.

"A new composite material named Fiber Reinforced Polymer is used in the strengthening field of civil engineering widely. This new strengthened technology using prestress (called prestressed reinforcement) can availably develop the strong capacity of FRP. In order to know the mechanical characteristics of RC beam in bending capacity, experimental study and theoretical analysis are combined in the process of research. Twelve concrete beams reinforced by CFRP bar, BFRP bar and strand were made. The reinforced effects including cracking load, yielded load, ultimate load, development of crack, improvement on rigidity and destructive mode are compared and researched" (Shangjian, Haibo, Shengdeng, & Meiguang, 2008).

2.2 FRP

Composite materials have been used almost as long as men have been building structures. First straws were used to strengthen mud bricks in ancient communities. Concrete made of portland cement and rocks are other signification of composite material. Fiber reinforced concrete is composite material that is high strength both in compression and tension (Mamlouk & Zaniewski, 2006). Fibers are both used as small particles that are randomly mixed into concrete or as more fabricated fibers that are in the form of sheets or tendons. Both sheets and tendons are used as external strengthening material but tendons are also used as internal reinforcement.

To manufacture FRP composite material the main materials that are required are; the reinforcing fibers and a polymer resin matrix. Continuous (very long) fibers are used when tendons and sheets are manufactured but short fibers often 10-50 mm are used when fibers are sprayed up. The volume of continuous fibers often varies from 20-60% of the total volume of composite material. (Bank, 2006).

Fibers are very small and have various mechanical properties. The filament diameter can vary from manufacturer to manufacturer as other properties can also vary. The diameter of basalt fibers can be about 12-18 µm, the tensile strength vary from 1650-4650 Mpa and elastic modulus is often 71-89 Gpa. The melting temperature is around 1450°C. The diameter of glass fibers are 3-24 µm but for structural engineering fibers around 17 µm are often used. The tensile strength is from 2300-4600 MPa, depending on type of glass fiber, the elastic modulus of glass fiber is often from 70-90 Gpa. The melting temperature to produce glass fibers are around 1400°C. Carbon fibers have diameter about 5-10 µm, the tensile strength varies from 2400-4800 Mpa and elastic modulus from 200 Gpa (similar as steel) up to ultrahigh modulus of 800 Mpa. Carbon fibers are produeced at temperature from 1200°C-2400°C. Aramid fibers were used many years ago to produce prestressing tendons but are not common product for structural engineering in present days. Aramid is also known as Kevlar trade mark. Aramid fibers are rather expensive and have high moisture absorptions and low melting temperature, around 425°C. Aramid fibers are therefore not as attractive for structural engineering applications as the other fiber types mentioned. The tensile strength is quite high from 3400 Mpa to 4100 Mpa and elastic modulus is from 70-125 GPa. (Bank, 2006; Novitskii & Efremov, 2011; Wei et al., 2010).

Carbon fibers were at first very expensive and the high price limited the use of CFRP in concrete but since the mid 1980´s lower cost carbon fibers have been produced with petroleum and coal pitch. Carbon fibers are very light and have high tensile and flexural strength. The elastic modulus can be as high as for steel and up to three times stronger. (Ramakrishnan, Tolmare, & Brik, 1998).

The polymer resin which is "the glue" that combines the fibers when composite material is fabricated can be of various types. Unsaturated polyester resin, epoxy resin and vinylester resin are the most common for FRP product used for structural engineering (Bank, 2006; Novitskii & Efremov, 2011; Wei et al., 2010).

Commercially fabricated FRP tendons have not been well qualified for use in architectural structures because of low heat resistant compared to steel. The heat resistance of FRP tendons are strongly dependant on the heat resistance of the matrix resin. Therefore new types of matrix resin that are high heat resisting for fabrication of FRP bars are desired. Sumida, Mutsuyoshi, & Pandey published an article (2007) where they describe research and development of a high heat resisting resin that gave promising result for heat resistance of FRP tendons. Beams tested in bending reinforced with carbon fiber tendons with this new resin had similar heat resistance as steel tendons. The future work in this field is to develop very high heat resistant resin.

FRP tendons or sheets for strengthening beams or slabs have been used in practice both in Europe and the United States for many years. It is very common to strengthen bridges that have been damaged for example because of corrosion in the reinforcing steel. One of the methods that have been tried and is giving pretty good result is what is called NSMR (near surface mounted reinforcement) (Nordin & Taljsten, 2005). This is possible if the concrete cover is quite big. Then grooves are sawed in the concrete cover and the strengthening material is fitted in the grooves. Nordin and Taljsten did an experiment (2005) where they prestressed CFRP tendon to strengthen RC-beams. Their result was that NSMR is better than external fitting because of peeling material while the strengthening material has been prestressed. The test shows that prestessing CFRP does increase the load that the beams resist before cracking, the serviceability limit state load (SLS) increases. Smaller cracks at SLS can be very important for the construction´s durability, it can prevent bad freeze – thaw influence and prevent corrosion in the steel.

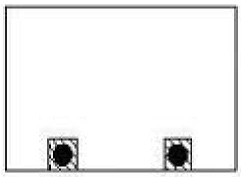

Figure 2-1: Schematic sketch of NSMR

As mentioned before there is very little research available about prestressing BFRP tendons and in general BRFP seems to be a rather new structural material. It looks like most of the recent research work about prestressing BFRP tendons has been done in China and only published in Chinese. But basalt fibers have though been investigated for quite long time and used in other fields than structural engineering. It looks like that both basalt and glass fibers have been investigated in the USA in the 1950s-70s but the glass industry in the USA have focused on glass fibers and basalt fibers were abandoned (Ross, 2006). That is probably the reason why basalt fiber is not mentioned in American books and codes.

FRP reinforced concrete has been investigated and used in the United States since the 1950s. First 20-30 years research focused on very small-diameter glass FRP-tendons, both for surface reinforcement and prestress. Later, in the 1980s, FRP tendons were used instead of steel tendons in highway bridge decks and other constructions were corrosion had been a problem. Most current applications today are in bridge decks and in underground tunnels. Very common fiber reinforcement is glass fibers because it is rather cheap in fabrication. (Bank, 2006).

GangaRao, Taly and Vijay (2007) say that FRP internal reinforcement has both advantages and limitations. FRP does not rust, it has high longitudinal strength, high fatigue endurance (that depends on type of FRP), is not magnetic, is very light in comparison to steel and has low thermal and electric conductivity. On the other hand, FRP is brittle so it does not yield before breaking. FRP has low elastic modulus relative to steel, low transverse strength and low shear strength. It also has reduced durability in moist, acid or salt and alkaline environments. Thermal expansions perpendicular to the fibers is high relative to concrete. Some types of FRP have low fire resistance both depending on the fiber type and type of matrix resin.

A fiber is anisotropic material and therefore the strength in transverse direction is very low in comparison to the very high tensile strength in the longitudinal direction. Because of this low transverse strength, ordinary steel anchors to fasten the ends of the tendons are not suitable (Adhikari, 2009; Bank, 2006). Though many manufactures have tried to develop specialized anchors for FRP tendons it is still the Achilles heel of prestressing FRP. It is both because of very high price and other technical problem (Bank, 2006). Specialized anchor is needed to prestress FRP and will be described in the chapter about experimental work.

An American standard recommendations about prestressing concrete structures with FRP have been published. There they give following recommendations: *"FRP tendons are produced from a wide variety of fibers, resins, shapes, and sizes. Only aramid and carbon fibers, however, are recommended in this document. Glass fibers have poor resistance to creep under sustained loads and are more susceptible to alkaline degradation than carbon and aramid fibers"* (*ACI 440.4R-04*, 2004, p. 9). BFRP is not mentioned, probably because it is a rather new material for structural engineering and the Americans seems to focus more on aramid, glass and carbon.

FRP has both limitations and advantages, and it could be said that more initial cost than steel reinforcement and a lower elastic modulus are the biggest lacks of FRP. Parreti et al. (2007) did a comparison on FRP (GFRP) and steel reinforced bridge deck. The bridge deck had the same cross section and the difference in designing was use of GFRP reinforcement or steel reinforcement. The main finding was that more reinforcement area (A_f) was needed when using GFRP than if steel (A_s) was used. Among other things it was a need to add GFRP shear stirrups, because of lower shear resistance of the concrete reinforced with FRP, but stirrups were not needed if steel reinforcement was used. Higher cost per unit and more units lead up to 30-40% more initial cost of reinforcement for this bridge deck. Despite this more initial cost, FRP is considered very appealing when taking into account maintenance cost of bridge decks reinforced with steel because of corrosion and waterproofing barriers over the whole lifetime of the bridge (100 years).

2.2.1 BFRP

Basalt is a very common type of rock. It´s therefore very convenient to use basalt to produce material to replace other materials that are not sustainable and are quite expensive. BFRP is environmentally and ecologically harmless and does not cause health hazard (Patnaik, 2009). Basalt does not have bad reaction to air or water and it is not flammable. When basalt gets in contact with other chemicals it does not produce chemical reaction that can be harmful for health or environment (Lopresto, Leone, & De Iorio, 2011). Production of basalt fibers is rather cheap, compared to other fiber types, and also much easier to produce than glass fibers. (Bashtannik et al., 2003)

As noted before BFRP tendons are rather new as structural material. In an experimental research from the year 1998, testing BFRP tendons among other fibers, the literature review about BFRP tendons, says: "*An exhaustive literature survey showed no data on basalt rod reinforced concrete*" (Ramakrishnan et al., 1998, p. 3). Basalt fibers are older and there was conducted basalt research in the old Soviet Union. Not many of them have been published in English, though a couple of them were found published in English (some translated to English).

Ramakrishnan et al (1998) found out from their tests on basalt fiber tendons that the bond strength was not good and the bars slipped when ultimate load was reached, their recommendations was to roughen the bars. Over the years great improvement on the surface have been made and for example the bars used in this research are like sandpaper to increase the bond strength. Other manufacturers have made the tendons with ribs, similar as steel tendons.

Brik (2003) did a research that was following Ramakrishnan et. al. (1998) to evaluate the characteristics and bond of so called modified BFRP tendons, which were tested as reinforcement for concrete beams. For comparison plain BFRP tendons were also tested. His conclusions was that the bond between concrete and modified basalt tendons is good. There was no slip of the tendons. The test indicated that the ultimate moment was much higher than the first crack moment. The plain BFRP tendons had considerably lower bond strength and did fail due to slip in a pull-out test.

In a research done by Lopresto et al. (2011) the possibility to replace glass fibers for basalt fibers was studied. Comparing mechanical and characteristic difference between glass fiber and basalt fibers reinforced plastic laminates. The main finding was that basalt fibers have overall better qualities than glass fibers and could be used to replace glass fibers as a filler in epoxy matrix application, where glass fibers are widely used now. They suggest that further investigation about this rather new material, basalt fibers, should be done.

Another research comparing basalt and glass fiber was done were the researcher tested tensile behaviour of basalt and glass fibers after chemical treatment. Sodium hydroxide (NaOH) and Hydrochloric acid (HCl) solutions were used to check the chemical resistant of the two fiber types. Both fiber types showed damages while treated with the solutions but overall the chemical stability of basalt fibers, and especially in acidic environment, was shown to be better than for glass fibers. Basalt fibers could be a good material compared to other better known fibers in a chemical environment for long time service (Wei et al., 2010).

Despite much evidence about how good basalt fibers are and promising results about mechanical properties there have also been published articles where questions are raised about the quality of BFRP. In a technical report done for the New England Transportation Consortium (Parnas, Shaw, & Liu, 2007) it is stated that the tests conducted indicated that there is a wide inconsistency between properties of basalt fibers from the literature (manufacturers) and the test results. They did many repeated measurements and used glass fibers to control the experimental method. According to Parnas et al. (2007) the only recommended investigations for the civil engineering community is to verify the properties of basalt and use of basalt should be done with great care! It should however be noted that this experimental work was performed in the year 2007 and since then lot of experiments have been carried out.

2.3 Prestress

The main aim of prestressed concrete sections is to limit tensile stress and also limit flexural cracks under service load (Mosley, Bungey, & Hulse, 2007). The tendons, most commonly steel but in this experiment BFRP-tendons, are pulled and then anchored on both ends. Then concrete is cast in the formwork around the tensioned tendons. When the concrete has reached enough strength to bond the tendon, the tendons are unfasten from the anchor device. The tension force in the tendon will transfer into the concrete and squeeze the concrete together.

In non-prestressed member that is resisting bending moment, only part of the cross-section can be considered as an effective section. The part of the cross section that is under the neutral axis is in tension and therefore cracked and is considered inactive. By prestressing members all the cross-section is in compression and all the section is available to resist the applied load in SLS and in ULS the neutral axis is lower than in non-prestressed beam. Prestressed beams are without cracks under service load (McCormac & Brown, 2009).

After releasing the anchor the member is supposed to go up in the middle of the span (see Figure 2-2) and it is necessary to check, by calculation, the tension stress on the upper side of the beam, it may not be over tensile strength of the concrete.

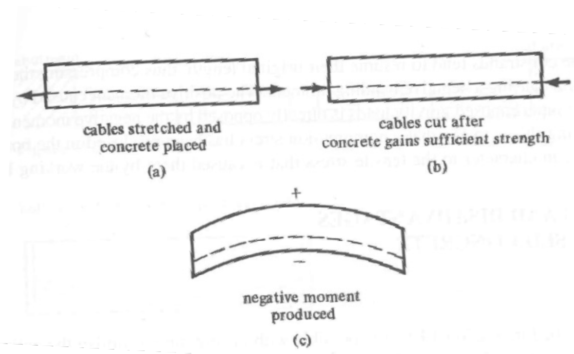

Figure 2-2: Prestress (McCormac & Brown, 2009, p. 559)

The total prestress force that is applied to the tendon will not transfer to the concrete beam. There will be losses which is a combination of various things. In normal concrete beams with prestressed steel tendons losses take place because of;

- elastic shortening of concrete that occur immediately after applying the force
- creep of concrete which is time dependent factor
- relaxation of steel which is also time dependent
- shrinkage of the concrete and is influenced by curing and environment
    - (Mosley et al., 2007).

The reasons for losses in prestressed FRP are the same, according to fib 14 (2001). The elastic shortening, creep and shrinkage of the concrete can be estimated in the same way but instead of relaxation of steel there is FRP and it depends on the type of FRP how much relaxation will occur. Further investigations of the behaviour of BFRP relaxation are badly need.

„Since the modulus of elasticity of FRP tendons is typically lower than a corresponding steel tendon, losses for prestressed FRP tendons due to elastic shortening, creep, and shrinkage of concrete will be less than for prestressed steel tendons" (El-Hacha & Couture, 2007, p. 12).

Normally concrete of higher strength is used for prestressed members than for normally reinforced members. The main reason for this high strength concrete is higher modulus of elasticity. That leads to lesser elastic shortening (McCormac & Brown, 2009).

Research and development of FRP tendons for prestressing concrete were done in the early 1980s in Holland (aramid), Germany (glass) and Japan (carbon and aramid). The main motivation was to reduce corrosion in prestressed concrete members. This was both internal and external prestressed FRP reinforcement. Products from this research were used in number of demonstration project through the 1990s, manly bridges, in Europe and the United States but continued use has not occurred (Bank, 2006). High price and problems with anchors seems to be the main reason.

According to American code (*ACI 440.4R-04*, 2004) for prestressing FRP in concrete structures, the typical prestress force for steel tendons is 85% of their yield strength or about 0.005 strain. However the typically allowed stresses in FRP tendons are from 40 to 65% of their ultimate strength due to stress rupture limitations.

Eurocode 2 (*EN 1992-1-1*, 2004) gives formulas to estimate the relaxation loss of prestressed steel tendons. The applied factor represent 1000 hours relaxation loss (in %) which are either provided by the manufacturer or value recommended by the standard. The long term relaxation (final) may be estimated for a time 500,000 hours (approximately 57 years). There is no available information about relaxation loss for BFRP tendons.

Experimental investigation of relaxation of prestressed FRP (Mehdizad Taleie, Vatani Oskouei, & Moghaddam, 2007) were carbon and aramid FRP strips were prestressed gave a result for relaxation after 48 hours and 1000 hours. The AFRP relaxation after 48 hours was approximetly 15.7% and after 1000 hours 22.4%. For CFRP the relaxation was much lower or 4.6% after 48 hours and 6.7% after 1000 hours. Many factors affect relaxation of FRP, for example the type of resin matrix, how much of ultimate tension force the prestress level is, types of fibers and size of specimen. The authors recommend more investigations about relaxation of FRP. The ACI prestress guide (*ACI 440.4R-04*, 2004) expressed result from relaxation test on CFRP (CFCC special type of CFRP strands) and steel strands where both materials were prestressed to 80% of their ultimate load. After 100 hours the relaxation had reduced by 2% for CFCC and 8% for steel. Another test result, expressed in same ACI guide, tested at 50%, 65% and 80% prestress force of ultimate load resistance gave 0.48% to 0.96% relaxation for CFCC and 1.02% to 7.35% for steel after 100 hours.

2.4 Shear strength of FRP reinforced beams

In an American approach to shear design (*ACI 445R-99*, 1999) it is stated that axial compression due to prestressing or other applied load increases the shear resistance of beams. But also that it is not well understood how much the shear resistance is influenced by axial load.

Shear strength of reinforced concrete is very complex. There are three main mechanisms that have an effect on shear reaction in beam that is reinforced with longitudinal bars and is subjected to bending. It is dowel action, aggregate interlock and the resistance of the concrete in the uncracked zone of the beam (Figure 2-4). The uncracked zone is what is above the neutral axis. How this mechanism works is still a debate (*fib 40*, 2007; O´Brian & Dixon, 1995).

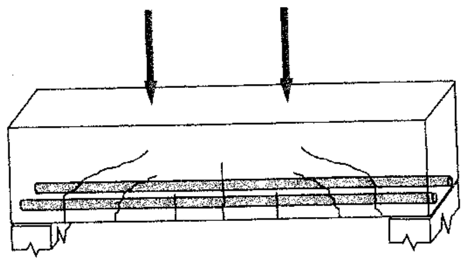

Figure 2-3: Shear cracks in beam with longitudinal reinforcement

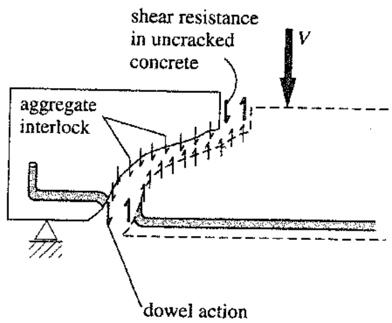

Figure 2-4: Mechanisms of shear transfer

The Shear strength of concrete members reinforced with FRP longitudinal reinforcement and without shear reinforcement (stirrups) cannot be calculated using the same equations as for steel reinforced members. There is a need to take into account the effects of material properties of the dowel. In an experimental research by Zou (2005) where ten prestressed beams were investigated, some were reinforced with steel tendons and some with CFRP tendons. The ones that were reinforced with steel tendons failed in bending as supposed but the ones reinforced with CFRP tendons failed unexpectedly due to shear. The conclusion, that the author drew from this, is that the dowel action of CFRP is less than for steel due to lower transverse shear strength of CFRP and he recommends further investigation in this field.

"*The shear strength of flexural concrete members with FRP longitudinal reinforcement and no shear reinforcement has indicated a lower shear strength than a similarly steel-reinforced member without any shear reinforcement*" (GangaRao et al., 2007, p. 248).

The American concrete institute published guidance for shear strength of concrete structures. There it is mentioned that prestressed beams have typically much lower angle of inclined cracks when shear failure occur than beams that are not prestressed. For beams that are shear reinforced with stirrups prestressed beams need fewer stirrups (*ACI 445R-99*, 1999).

The American building code for concrete (*ACI 318-08*, 2008) gives formulas to calculate shear resistance of the concrete in members without shear reinforcement (Equation 2-1) and according to (McCormac & Brown, 2009, p. 221) the shear strength of concrete can be calculated in SI-units by equation 2.

$$V_c = 2 \cdot \lambda \cdot \sqrt{f'_c} \cdot b_w \cdot d$$

Equation 2-1: Shear strenght ACI in U.S units

$$V_c = \left(\frac{\lambda \cdot \sqrt{f'_c}}{6} \right) \cdot b_w \cdot d$$

Equation 2-2: Shear strength ACI in SI-units

Were the factor λ is equal to 1 for normal weight concrete, it is lower for light weight concrete (McCormac & Brown, 2009). This is the normal American shear strength formula for concrete members with longitudinal steel reinforcement.

These above formulas are simple practical formulas just as the following that are introduced by EC2 (*EN 1992-1-1*, 2004) for shear strength of concrete without shear reinforcement (Equation 2-3 and Equation 2-4) where the latter equation is for the minimum value of shear resistance.

$$V_{Rd,c} = (C_{Rd,c} \cdot k(100 \cdot \rho_I \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp}) b_w \cdot d$$

Equation 2-3: EC2:2004-shear resistance of concrete

$$V_{Rd,c} = (v_{\min} + k_1 \cdot \sigma_{cp}) b_w \cdot d$$

Equation 2-4: EC2:2004 minimum value of shear resistance

But there is one big difference and that is that the ACI code does not take into account the effects from the prestress, the compression. EC 2 does not suggest any FRP reinforcement modifications to these formulas. But considering following references there should be some recommendations about it in EC2.

The American concrete institute has published guide for design of concrete with FRP bars. Cross section which is reinforced with FRP longitudinal bars has lower axial stiffness than steel reinforced cross section. Therefore after cracking, the depth of the neutral axis is smaller. That is affected by the modulus of elasticity of the tendon. Then the compression zone is smaller and the shear resistance provided by the concrete gets smaller (*ACI 440.1R-03*, 2003). The guide for designing with FRP bars gives the following equations for shear strength of concrete reinforced with FRP bars, see equation 2-5 and rewritten equation 2-6. V_c is as expressed in ACI 318.

$$V_{c,f} = \left(\frac{A_f \cdot E_f}{A_s \cdot E_s} \right) \cdot V_c = \left(\frac{\rho_f \cdot E_f}{\rho_s \cdot E_s} \right) \cdot V_c \leq V_c$$

Equation 2-5: Shear strength of FRP reinforced concrete

$$V_{c,f} = \frac{\rho_f \cdot E_f}{90 \cdot \beta_1 \cdot f_c'} \cdot V_c$$

Equation 2-6: Shear strength of FRP reinforced concrete

It should be noted that these equations are in U.S units but can be used for SI-units if V_c is calculated from the formula for SI-units according to design examples in ACI 440.1R-03 guide. The factor $\beta_1 = 0.85$ for concrete strengths up to 27.5 Mpa (4000 psi), for strength over 27.5 the value reduces at a rate of 0.05 for each 6.9 MPa (1000 psi) of strength increasing to a minimum value of 0.65. That gives minimum value of β_1 for 60 Mpa concrete.

These formulas were reconsidered and in later edition of the ACI-FRP guide (ACI440.1R-06) empirical modifications have been done on these previously introduced formulas for FRP (Bank, 2006)

$$V_c = 5 \cdot \sqrt{f_c'} \cdot b_w \cdot c$$

Equation 2-7 - ACI.440 1R-06:9-1

Or written as:

$$V_c = \frac{5 \cdot k}{2} (2 \cdot \sqrt{f_c'} \cdot b_w \cdot d) \text{ and in SI units } V_c = \frac{5 \cdot k}{12} \cdot \sqrt{f_c'} \cdot b_w \cdot d$$

Equation 2-8: ACI 440.1R-06:9-1a. US units and SI units

Where the factors c and k are calculated as follows:

$$c = k \cdot d$$
$$k = \sqrt{(\rho_f \cdot n_f)^2 + 2 \cdot \rho_f \cdot n_f} - \rho_f \cdot n_f$$
$$n_f = E_f / E_c$$

Here is the ratio of elastic modulus between fiber and concrete taken into account and the factor c is reduced size of the factor d that is commonly known to be used. The factor c is depth of the neutral axis and is considered to be smaller in FRP reinforced beams than in steel reinforced beams because of lower elastic modul of the longitudinal reinforcement (Bank, 2006).

All previously introduced ACI code formulas do not take into account effects from prestress and do not address prestressing effects. One ACI code (*ACI 440.4R-04*, 2004) about prestressed FRP concrete structures has been published. It should be noted that the code was published 2004, that is before the revised FRP guide, ACI 440.1R, which was published 2006. This prestress code introduces nothing new to V_c strength, it gives a formula for FRP stirrups and the prestress factor V_p is taken into account. V_p is the vertical component of the prestress force. This is only relevant if the tendon is curved. The total shear strength for prestressed FRP reinforced concrete according to ACI-440.4R-04 is then as follows:

$$V_n = V_c + V_{frp} + V_p$$

Equation 2-9: Prestress, ACI440.4R-04:(5-1)

Where V_c is suggested to be calculated by the formula introduced by ACI318. It is probably valuable to calculate V_c pursuant to ACI.1R-06. Here in this prestress formula the effect of axial compression is totally overlooked.

An Italian code, called CNR-DT 203, is a guide for designing FRP reinforced concrete structures. In the introduction it is noted that principles and practical rules in the code can´t be used for prestressed structures which are using FRP bars. This code will nevertheless be introduced here because it is dealing with FRP with reference to EC2. It is also mentioned that prestress applications need specific guidance. (*CNR-DT 203*, 2007). It is pointed out that for members that do not need shear reinforcement, the minimum longitudinal FRP reinforcement in tension shall satisfy the equation $\rho_I = A_f/(b \cdot d) \geq 0.01$ which is a new one from EC2 where the reinforcement ratio is only limited as $\rho_I = A_f/(b \cdot d) \leq 0.02$. This Italian code tries to find out reasonable approximation for the EC2 formula (EN 1992-1-1:1992) for shear strength of concrete without shear reinforcement. It was considered to be more practical for engineers at the time this was presented to use the older version of EC2 rather than try to modify the formula presented in the final EC2:2004. (Fico, Prota, & Manfredi, 2008).

$$V_{Rd,c} = \tau_{rd} \cdot k \cdot (1{,}2 + 40 \cdot \rho_I) \cdot b_w \cdot d$$

Equation 2-10: CNR-DT 203 (EN 1992-1-1:1992 old version) without effect from axial force

$$V_{Rd,ct} = 1{,}3 \cdot \left(\frac{E_f}{E_s} \right)^{1/2} \cdot V_{Rd,c}$$

Equation 2-11: CNR-DT 203 to take FRP into account

Fico et al. (2008) did a survey on many experiments which was used to determine the EC2 modification made in CNR-DT 203. Data for 88 beams and slabs with no shear reinforcement were gathered and investigated. They compared experimental results to calculated results according to Japanese code, ACI code, Canadian code and this new Eurocode-like equation expressed in the Italian guide CNR-DT 203.

The factor 1.3 in Equation 2-11 was applied after evaluating all the experimental data the authors had collected and analysed. Using the ratio between elastic modulus of fiber and steel is a method similar to that introduced in ACI formulas. Equation 2-11 is suggested by the authors as a valuable and a reasonable approximation to shear strength of FRP reinforced concrete while more information about how behaviour of FRP reinforcement varies from steel reinforcement. Once more information is available, updated equations will be published.

The Federation for Structural Concrete published a bulletin (*fib 40,* 2007) were modifications to some of previously introduced code formulas were introduced. For both EC2 and ACI-318 it is recommended to add factors that are the ratio between elastic modulus of fiber and steel and ratio between allowed yield strain of FRP reinforcement and steel reinforcement. These recommendations are though, like the other, not taking into account effects from prestress.

$$V_{Rd,c} = C_{Rd,c} \cdot k \cdot \left(100 \cdot \rho_I \cdot \frac{E_f}{E_s} \cdot \phi_\varepsilon \cdot f_{ck} \right)^{1/3} \cdot b_w \cdot d$$

Equation 2-12: fib 40 - modified Ec2:2004

Where $\varphi_\varepsilon = \varepsilon_f/\varepsilon_s$ is ratio between allowed strain, recommended ε_f=0.0045 and ε_s=0.002. Similar modifications are recommended for ACI-318.

$$V_{cf} = V_c \cdot \left(\frac{E_f}{E_s} \cdot \phi_\varepsilon\right)^{1/3}$$

Equation 2-13: fib 40 - modified ACI-318

2.4.1 Calculations

Based on the formulas that were looked at in the literature review about shear strength, calculations were made to compare the result. It should be noted, as mentioned before, that some of these equations are not for FRP reinforced concrete. Here the shear strength is calculated as recommended by the relevant code, for example with safety factors. In the table below (Table 1), first the shear strength is shown in KN, then whether the formula is for steel or FRP reinforcement, then from which code or guide it is and last the formula for each case is shown. The last formula is a modification to fib 40, done by the author of this work, where effects from prestress are added to the formula in the same way as it is done in EC2:2004. These results are for the cross section that will be introduced in the experimental work.

	Shear strength	Steel/FRP	Code	Formula
	KN			
V_c	38,7	Steel	ACI 318-08	$V_c = \left(\frac{\lambda \cdot \sqrt{f'_c}}{6} \right) \cdot b_w \cdot d$
V_c	33,3	Steel	EC2:2004	$V_{Rd,c} = (C_{Rd,c} \cdot k(100 \cdot \rho_l \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp}) b_w \cdot d$
$V_{c,min}$	33,6	Steel	EC2:2004	$V_{Rd,c} = (v_{min} + k_1 \cdot \sigma_{cp}) b_w \cdot d$
V_c	22,7	Steel	EC2:2004	$V_{Rd,c} = (C_{Rd,c} \cdot k(100 \cdot \rho_l \cdot f_{ck})^{1/3}) b_w \cdot d$
$V_{c,min}$	23,0	Steel	EC2:2004	$V_{Rd,c} = v_{min} b_w \cdot d$
V_{cf}	2,9	FRP	ACI 440.1R-03	$V_{c,f} = \frac{\rho_f \cdot E_f}{90 \cdot \beta_1 \cdot f'_c} \cdot V_c$
V_{cf}	10,9	FRP	ACI 440.1R-06	$V_c = 5 \cdot \sqrt{f'_c} \cdot b_w \cdot c$
V_{cf}	13,7	FRP	CNR-DT 203	$V_{Rd,ct} = 1{,}3 \cdot \left(\frac{E_f}{E_s} \right)^{1/2} \cdot V_{Rd,c}$
V_{cf}	18,5	FRP	fib40(EC2)	$V_{cf} = 0{,}12 \cdot (1 + \sqrt{\frac{200}{d}}) \cdot (100 \cdot \frac{A_f}{b_w \cdot d} \cdot \frac{E_f}{E_s} \cdot \phi_\varepsilon \cdot f_{ck})^{1/3} \cdot b_w \cdot d$
V_{cf}	31,4	FRP	fib40(ACI)	$V_{cf} = V_c \cdot (\frac{E_f}{E_s} \cdot \phi_\varepsilon)^{1/3}$
V_{cf}	**29**	**FRP**	**EC2(fib 40)**	$V_{Rd,c} = (C_{Rd,c} \cdot k \left(100 \cdot \rho_l \cdot \frac{E_f}{E_s} \cdot \phi_\varepsilon \cdot f_{ck} \right)^{1/3} + k_1 \cdot \sigma_{cp}) b_w \cdot d$

Table 1: Comparing shear strength of concrete beams without shear reinforcement.

Note: the last equation is author's modification to the fib40 equation based on the EC2:2004 equation.

2.4.2 Discussion

The calculations above show how great the difference is between calculated shear strength of the beam when using different formulas. Equations that are meant to be for steel reinforcement give much higher shear strength than equations that have been modified to use for FRP reinforcement. While looking at these results, it can be seen that the oldest FRP equation (ACI440.1R-03) is giving very low value of shear strength. The

FRP equations that have been published since are giving higher values and hopefully getting closer to the actual shear strength.

All studies that have been reviewed about shear strength of FRP reinforced beams neglect the effect of axial compression, it is only the EC2 formula for steel reinforcement that take axial compression into account. None of these studies are about BFRP, it is mostly about carbon-, aramid- or glassfibers. It seems to be an urgent need to investigate shear strength of prestressed FRP reinforced beams and also study whether there is some difference between BFRP and other types of FRP. It is not very well understood how much shear strength increases due to applied axial load or prestressing force (*ACI 445R-99*, 1999).

The code that European engineers are supposed to use nowadays is Eurocode 2 (*EN 1992-1-1*, 2004) and the shear strength of concrete is calculated there. Marí and Cladera (2007) did a research to estimate the accuracy of the new Eurocode 2 (2004) equation for shear strength of concrete. They studied many experiments and calculated shear strength using equations from many codes and they concluded that EC2 (2004) is a step forward for practical engineers because of its simplicity but can be inaccurate in some cases. Nevertheless it is probably most adequate for European practical engineers if there will be introduced modified FRP shear strength formula for concrete that is with reference to the formula which is introduced in Eurocode 2:2004.

2.5 Flexural strength

When reinforced concrete is designed and the strength of reinforcement is fully utilised the cross section is considered to be ”under-reinforced”. When the reinforcement does not reach its full strength before the concrete crushes in the compression zone the cross section is considered to be “over-reinforced”. When designing reinforced concrete members so called “balanced condition” is often assumed. In balanced cross sections the concrete compressive strength and reinforcement tensile strength is considered equal (*fib 40*, 2007).

Internally FRP reinforced beams are beams were steel tendons are replaced with fiber tendons. Flexural strength of such beams is calculated similar as the one with steel tendons but it must be recognized that FRP reinforcing tendons differ from steel. Mechanical properties are different, for example very high tensile strength in the longitudinal direction only. FRP reinforcement is considered to be unsuitable as a compression reinforcement and therefore only single reinforced beams are discussed in most literature. The compressive resistance of the tendons is considered to be zero (GangaRao et al., 2007). When applying the method of balanced design which utilise the full tension strength of the tension reinforcement and using FRP tendons a large proportion of the cross section will be in tension. It leads to much more deflection and higher strain in the compression zone than if similar steel reinforcement had been used (*fib 40*, 2007). Bank (2006) note that since the FRP bars do not yield, as can be shown in Figure 2-5 where stress-strain relationship of various types of FRP and steel are shown, the ultimate strength of FRP replaces the yield strength of steel when calculating the ultimate flexural resistance.

Zou (2005) carried out an experimental research where CFRP tendons were prestressed at the range of 40%-60% of guaranteed ultimate strength and compared to prestressed steel tendons. In this research cracking load, deflection and ultimate load were investigated on ten full scale beams. The main findings are that the experimental values of cracking load are close to theoretical values. In other word, the cracking load can be calculated with adequate accuracy. By increasing the level of prestress force the cracking load did increase but the ultimate moment carrying capacity was not affected. The deflection did diminish.

Bank (2006) says that when steel is used for balanced design the balance state is at the yield point of the steel and after that the section can still carry load until it fails totally. But when using FRP reinforcement the ultimate strength is used and therefore catastrophic collapse will occur. However Zou (2005) did state, from his investigation on prestressed beams that the deflection of the beams were so great before collapse that it does give sufficient warning.

Reinforced concrete beams or slabs are designed to resist bending moment at two limit states. Ultimate limit state (ULS) and serviceability limit state (SLS). At SLS deflection, which must be within acceptable limits, is considered and cracks are controlled. Cracks must be limited to prevent moisture to reach the reinforcement and cause corrosion (in steel) and prevent freeze-thaw effects. It is assumed that stress/strain relationship in SLS is linear. At ULS the safety of beam or slab is considered and the capability to resist moment is compared to ultimate moment from applied load. Stress/strain relationship is non-linear and high stresses occur (O´Brian & Dixon, 1995)

FRP tendons under tension load are linear elastic to failure while steel has yield point (Figure 2-5). FRP tendons fail in a brittle manner but steel has a ductile failure. (Bank, 2006). The linear elastic behaviour until FRP tendons fails can lead to catastrophic collapse of members.

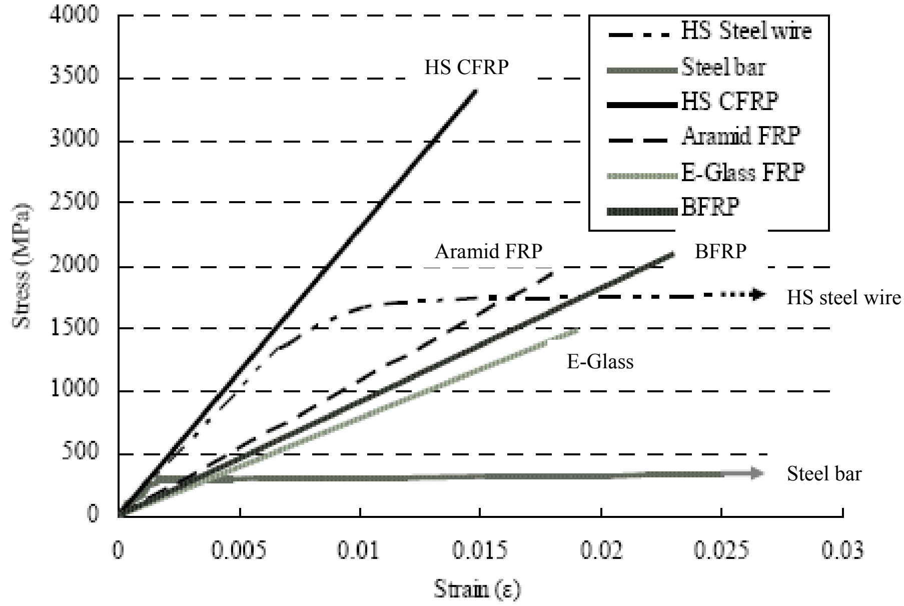

Figure 2-5: Short-term mechanical behaviour (Z. Wu, Wang, & G. Wu, 2009)

When member carries a load that cause bending, it gives compression on one side and tension at the other, there are two failure modes that can occur. One is the failure of the concrete in the compression zone (crushing) and the other is the rupture of the FRP (*ACI 440.1R-03*, 2003).

Design guidelines for moment resistance members reinforced with FRP longitudinal tendons are quite similar to the one where steel reinforcement is used.

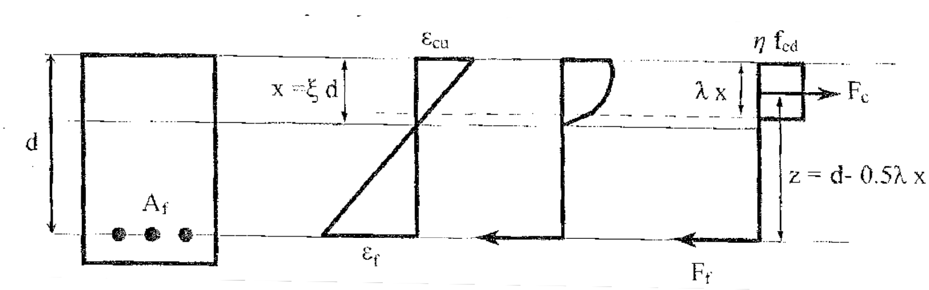

*Figure 2-6: Simplified stress block proposed for FRP reinforced concrete members (*fib 40*, 2007).*

Method provided by fib 40 (*fib 40*, 2007) where they adopt the framework from Eurocode 2 (*EN 1992-1-1*, 2004) and the Italian guide CNR- DT203 (*CNR-DT 203*, 2007) is used to estimate the flexural strength of the beams. The methods introduced in American guidelines and textbooks are similar to this.

3 Experimental work

3.1 Introduction

The experimental work was in fact pretty simple! Just cast several beams with prestressed tendons. But to do that extensive modifications of equipment were needed because work like this has not been performed at Reykjavik University before and quite many things were needed to be taken care of in the preface.

BFRP tendons that were used in this experiment were imported by Innovation Centre Iceland from Magmatech in the UK. Technical information can be seen in appendix A.

Figure 3-1: BFRP bars with sanded surface from Magmatech - Rockbar

3.2 Preface – equipment adjustments

It was decided to use the laboratory at Reykjavik University (RU) for prestressing and casting. It was considered to get a company that specializes in precasting concrete elements involved to this work. After a brief thought, it was decided to use the facility at RU. Because the test setup is unlike and smaller than typically used in factories it was considered better to modify the equipment at RU than trying to modify the equipment in some factory. Other benefit conducting the experiment at RU is that other students can later on have opportunity to use the setup for similar experiments. It also gives the researcher more space to work and investigate than if it would have been in a factory where fabrication is going on.

It was predetermined to cast three beams at the same time. A solution to prestress tendons for three beams was therefore needed. Placing the beams in one line (end to end, Figure 3-3) and use only one jack to prestress gave the best result and exactly the same prestress force in all the beams. Casting the beams at same time has many practical advantages; the main reason was to save time but also to have three beams with similar properties; same concrete, same curing time etc.

The length of each beam was decided to be 2.0 m because each BFRP tendon was only 2.5 m long and it was necessary to have free space at each end for joining of the tendons into one 7.5 m long tendon.

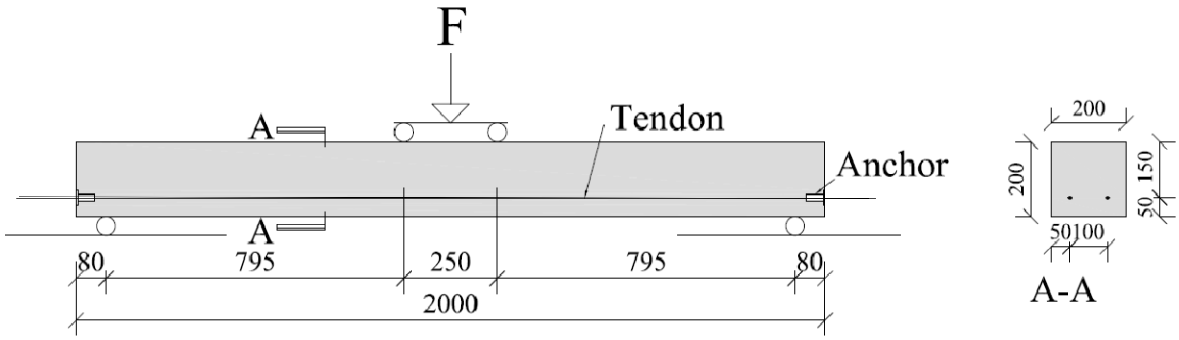

Figure 3-2: Schematic drawings of bending test setup and beam cross section

The test setup was decided in the preface to estimate the expected failure strength and see how the cross section of the beam should be. It was the plan that the beam would fail in bending so the cross section was therefore designed to be 200x200 mm. First it was calculated as h=250 mm and b=150 mm. But that gave very high bending strength and relatively low shear strength, and then wider and lower cross section was decided. Good space for the anchors at the ends was needed so very narrow beam was not eligible. Calculation in this first stage can be seen in chapter 3.2.1

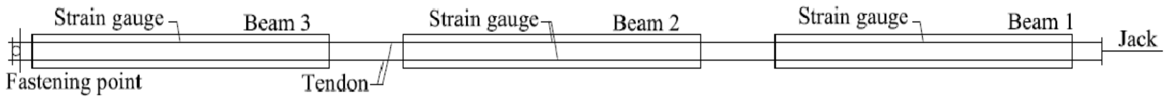

Figure 3-3: Schematic drawing of the prestressing setup

Connecting the BFRP tendons and making anchor on the ends was also “homemade”. Hot rolled circular pipe, with outside diameter 22 mm and inside diameter 13 mm, was threaded on the inside for better traction. Glue for plaster-bolts from the manufacturer Hilti, called HIT-RE-500 (see appendix B) was used to glue the BFRP-tendons inside the steel pipes. Similar technique had been used in previous experiments (Adhikari, 2009; Ramakrishnan et al., 1998). To check the tension capability of this combination, a small test was performed. BFRP tendon was glued inside two 15 cm long steel pipes and then pulled until the tendon failed (Figure 3-4) and that was at approximately 87 KN or almost twice as much as the intended prestress force.

Figure 3-4: Testing Hilti glue

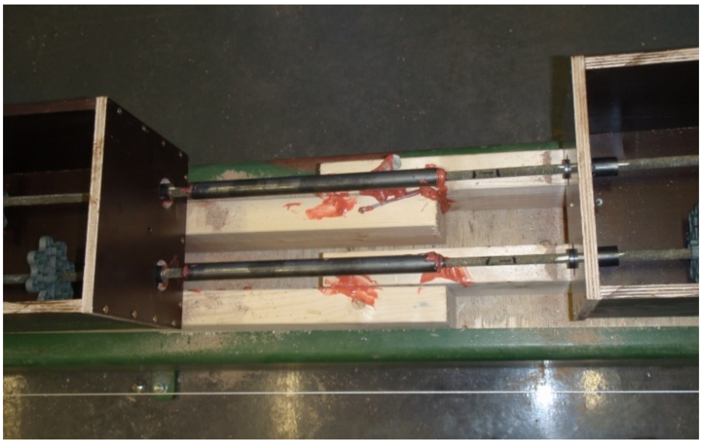

Figure 3-5: Combination of tendons

Figure 3-5 show the combination of tendons where 30 cm long steel pipe was filled with Hilti glue and then the end of each tendon was pressed into the middle. It can be seen that the steel pipe is closer to the formwork on the left side, that is because when the tendons are pulled it was expected to be drawn in to the hydraulic jack approximately 70 mm so there was need to have space for that movement. Short steel pipe (50 mm) welded to 5x50x200 mm anchor end plate (see also Figure 3-11) was glued after pulling the tendons so it could be positioned at right place after all the movement had occur. Small hole was drilled in the middle of the anchor steel pipe for adding the glue.

A lot of adjustment was needed in the laboratory. In the laboratory the stressing rig consisted of a long RHS-profile with a jack connected to it on the other end and it was used as a "ground" for the casting and prestressing face. This equipment has the name Hallgerdur langbrok (Figure 3-6). The length of the RHS-profile was 8.0 m so there was enough space for three beams, 2 m each, and a space between them to connect the tendons. On the other end of Hallgerdur a fastening point was set down and there was a kind of joint (Figure 3-8) to split the force, that was given by the hydraulic jack on the other end, into two equal parts to the tendons. Then the formwork could be custom built.

Plywood was bought at a local hardware store, sawed in correct size and the formwork was custom made in the laboratory. The height of the formwork needed to be adjusted very accurately, relative to the hydraulic jack, to get the tendons at right height in the concrete cross section.

Figure 3-6: Hallgerdur langbrok

Figure 3-7: Eirikur raudi

It was also necessary to prevent the hydraulic jack to loose its prestressing force over time. To fasten the hydraulic jack, strong steel bars were welded on each side of the jack (see Figure 3-9) and right after prestressing it was welded to the endplate of the cylinder.

Figure 3-8: Joint on the fastening point of the tendons

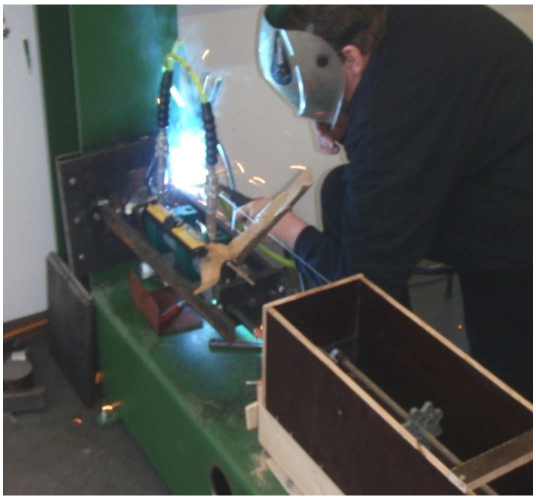

Figure 3-9: Fastening the jack to prevent loss of force

It was necessary to use a jack to break the beams. The available jack, called Eirikur raudi, (which was mainly built for breaking columns) did not have the capacity of breaking 2 m long beams, so adjustment to the jack were needed. (Figure 3-7).

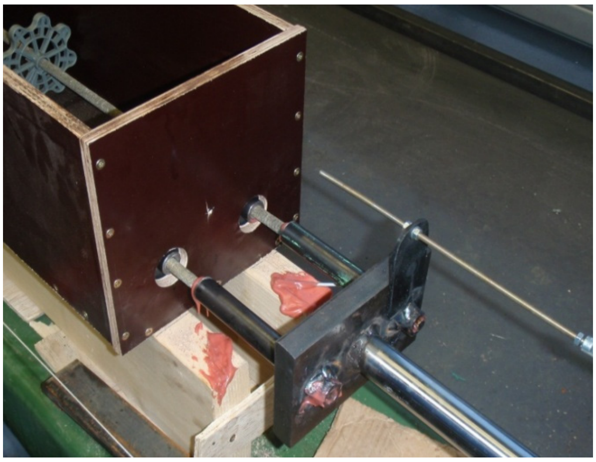

Figure 3-10: Prestressing jack

Figure 3-11: Anchor inside the formwork

Figure 3-10 shows how the end of the hydraulic jack was modified with thick steel plate and steel pipes into which the BFRP tendons were glued. Transducer was also fastened to the steel plate. Figure 3-11 shows the anchor at each end of the beams. It was combined of steel plate 5x50x200 mm and steel pipe which is 50 mm long with tread on the inside for better traction. The pipes were welded to the plate afterwards. This was all loose when the prestressing was taking place, then the glue was injected to the pipe through a hole which was drilled in the middle of the pipe. The glue was injected just before casting and therefore the pipes were welded to the plates after several days so the concrete and the glue had a good time to cure.

Prestress force was decided about 47 KN, which gave around 600 Mpa stress in the BFRP tendons. This is approximately 50% of the ultimate tensile strength of the tendons (see appendix A). This is in the recommended zone for prestressing FRP according to American concrete institute (*ACI 440.4R-04*, 2004). There it is recommended to prestress FRP tendons 40%-65% of ultimate tensile strength. While steel tendons are normally tensioned up to 85% of their yield strength.

The strain in tendons was measured by strain gauges that were placed on the middle of the tendons, at the constant moment zone. The available computer hardware does only support four strain gauges at the same time. Therefore the strain was only measured in one tendon in beam 1 and 3 but in both tendons in beam 2 while prestressing. Then when breaking the beams both tendons were measured in each beam. Figure 3-12 shows how the stain gauges were placed, the sandy surface of the tendons was polished with sandpaper. The strain gauges were glued on small aluminium plate and the plate then “glued” to the tendon with steel repair filler. The wires that are connected to the gauges were protected from the concrete with small tubes and then this was all carefully wrapped with insulating tape (Figure 3-13). It is very important to prevent water from reaching the strain gauges.

Figure 3-12: Strain gauges

Figure 3-13: Strain gauges, carefully wrapped

3.2.1 Calculations – expected resistance

The size of the beams was limited to about 2 m because the BFRP tendons were only 2.5 m long and it was necessary to have free ends. With the length limited, it was tried to find a cross-section that would break in bending. In the preface simple calculations based on EC2:2004 were done. The shear strength was calculated according to previously introduced equation for member without shear reinforcement. See chapter 6.2.2 in EC2:2004. The flexural resistance was determined using simple stress distribution, see schematic stress distribution on Figure 3-14, taking into account the compression force and eccentricity of the prestressed tendons. The prestress stress was decided to be 600 Mpa, 50% of ultimate tension resistance of the BFRP tendons, and the ultimate flexural resistance estimated.

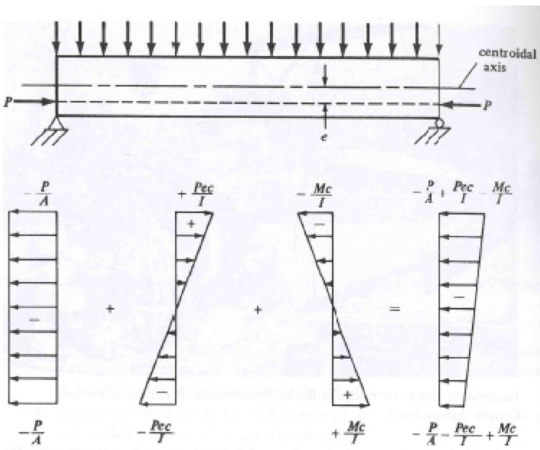

Figure 3-14: Schematic figure of stresses due to applied load and prestress

These first calculations, based on well known methods where steel tendons are used for longitudinal reinforcement but here the ultimate tension strength for BFRP tendon was used, gave following results:

- $F_{ult} = 71.1$ KN
- $M_{ult} = 28$ KNm
- $V_{req} = 35.6$ KN $= F_{ult}/2$
- $V_{Rd,c} = 42.3$ KN

With these result it was decided to use previously introduced cross section 200x200 mm. And it was expected that the beam would fail due to flexure.

3.3 Concrete

Concrete for the prestressed beams was fabricated at the Testing and Research Lab at Mannvit Engineering, with pretty good equipment to manufacture concrete for small specimens. But afterwards it came clear that the volume of concrete needed, 0.32 m^3, was three times of the mixer´s capacity. The whole procedure took a very long time and resulted in a different workability between mixes. This was probably because of different humidity in the gravel. In appendix D the properties of concrete mix can be seen. For the prestress beam it was desired to have a rather high strength concrete, or at least 50 Mpa. Two reasons were for that; precast factories use very often concrete of that high strength and this experiment was supposed to simulate precasted slab and on the other hand it was necessary to get good bond to the tendon in a few days´ time to minimise the risk of losing the prestress force due to slippage of the tendons. Advice was sought to concrete experts at Mannvit for the concrete mix.

The beam which was not prestressed was cast with concrete from the supplier BM-Valla and brought to the lab at RU in concrete truck. Expected strength of that concrete was 25 Mpa. Beam 4 was cast 2 days later than the prestressed beams.

Figure 3-15: Concrete mixer at Mannvit

Figure 3-16: Concrete moved to RU-lab in tub

The concrete was moved to HR lab (Figure 3-16) where the formworks were. All this procedure, mixing and moving, took very long time.

3.4 Cylinder specimen

Cylinder specimen was cast in standard steel formwork, size 150x300 mm, according to standard procedure (ÍST EN 12390-3). There were cast 2x3 specimens, 3 from mix number 1 and 3 from mix number 3. These cylinders were stored in humidity room at standard atmosphere in the Testing and Research Lab at Mannvit Engineering.

The concrete which was used for the beam that was not prestressed was cast in standard steel form at the size 100x200 mm. This was done at the RU lab and the specimens were stored there, carefully wrapped in vacuumized plastic to prevent too quick drying.

All the specimens were axially loaded in universal testing machine at Mannvit laboratory. This machine is standardised and calibrated according to standard. The load speed is 600 kpa/s. The drying time was 20 days for the concrete used in un-prestressed beam and 22 days for the concrete used in prestressed beams.

Figure 3-17: Cylinder specimen 150x300 mm

Figure 3-18: Cylinder after testing

The C50 cylinders did explode while testing but the C25 cylinders just cracked a little bit.

The results from testing the cylinders were as the following table (Table 2) shows.

Concrete group	Days	Average size b, (mm)	h, (mm)	Test result (KN)	(Mpa)
C25-A	20	99,8	199,7	180,1	23,0
C25-B	20	99,8	200,4	184,0	23,5
C25-C	20	99,9	200,1	180,4	23,1
				Average	**23,2**
C50-1A	22	149,9	300,2	1102,1	62,4
C50-1B	22	150,2	300,3	1095,0	61,8
C50-1C	22	149,8	301	1075,7	61,0
				Average	61,7
C50-3A	22	150,1	300,3	1080,9	61,1
C50-3B	22	150,0	300,6	1012,5	57,3
C50-3C	22	150,0	300,3	1035,3	58,6
				Average	59,0
				C-50 Average	**60,4**

Table 2: Cylinder test, strength and size

More details about the specimens can be seen in appendix D.

3.5 Casting

As noted before the concrete was moved to RU lab and placed into the formwork. Because of the long time it took to mix all the concrete and move it, the first mix that was at the bottom of the tub was getting quite stiff when it finally got into the formworks. It was quite a hard work to get these beams well cast and vibrating the concrete was done by picking it with a stick and hammering the formwork on the outside. The strain gauges are very delicate and therefore it was decided not to use electric vibration while placing the concrete. Afterward it can be seen that this was not the best procedure and it would have been better to get the concrete from a supplier in a concrete truck. Right after casting the beams the formwork was carefully wrapped into plastic to hinder too much drying. A hygrometer and thermometer were placed under the plastic.

The humidity was 96-97% for most of the time and the heat was steady after day 2, about 22°C (similar to the room temperature). On the second day the heat was up to 32°C and it was noticed on the strain gauge measurements (see chapter 4.2).

The anchors were welded together, that is the pipes were welded to the plate on day 9. The formworks were opened then for the first time (Figure 3-19).

Figure 3-19: Welding the anchors

3.6 Data gathering and analysis

The hydraulic jack that was used was connected to computer with software that had been developed by students and teachers at RU. This equipment had been calibrated by specialists. The software logged applied force and distance which is measured with transducer mounted to the jack.

The strain is measured with specialized strain gauges and related computer software.

Figure 3-20: Strain gauges

The strain in the tendons was measured constantly from the start of prestressing and until the day when the beams was broken. The time interval for taking measurements was 9.92 seconds. There came up one problem while gathering the strain data. The computer stopped the log approximately at every 28 hours and was restarted. It was probably because of overflow of data into the file. So the data collection for these 22 days had gaps but the time logging is very accurate and the software gave exact time for the beginning and numbers of data in each file so it was possible to calculate how long the time gaps were and then put all the files together with a correct time scale. Then it was possible to plot the strain over all the time period. Frequent logging was needed in the beginning while the stressing procedure is on, but after fastening the hydraulic jack it would have been more convenient to have the time interval bigger. The strain in tendons was also measured while breaking the beams. The time interval for taking measure point was then 0.5 s.

Both data about strain and force in the hydraulic jack were copied into excel files and plotted using excel.

4 Results of the experiment

4.1 Breaking beams

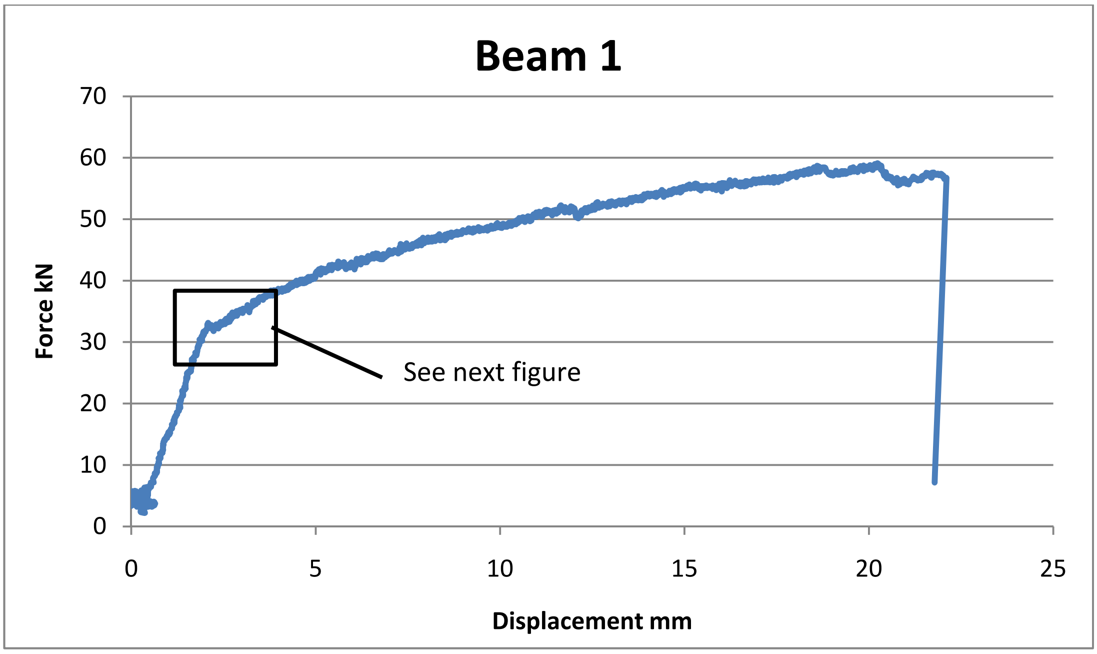

Figure 4-1: Force-displacement relationship, beam 1

This figure shows the force vs. displacement for beam 1. Maximum force was 59 KN. This was a shear failure. The displacement is around 22 mm.

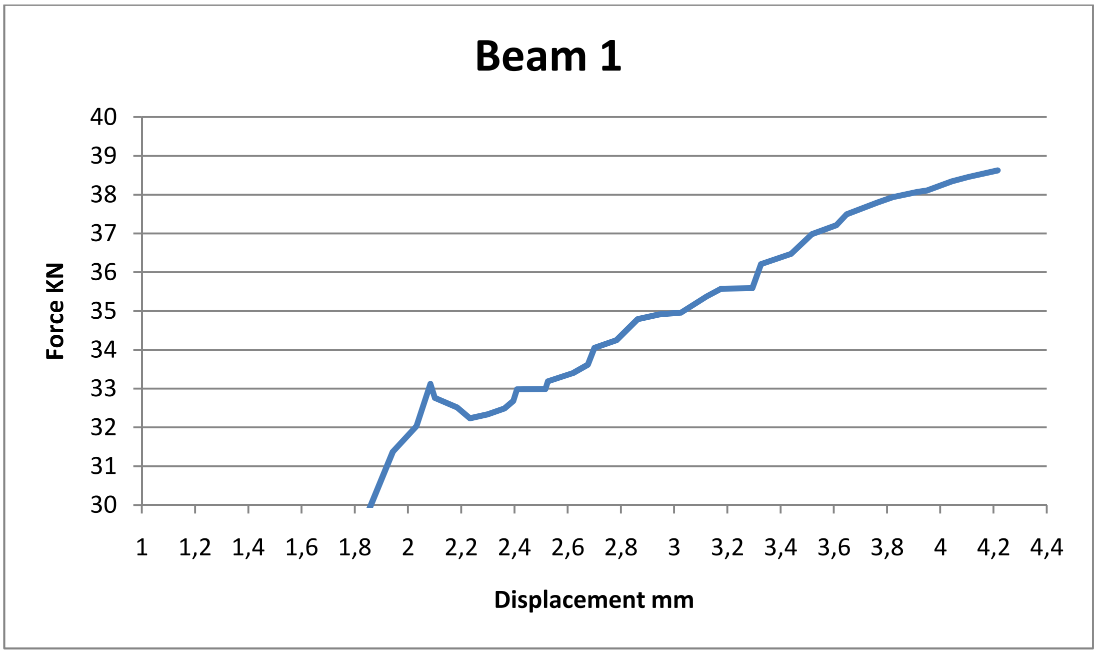

Figure 4-2: Force-displacement relationship, beam 1, zoomed up

Here is the figure zoomed up around the point where the concrete first cracks. As can be seen it is at 33 KN force. This is then considered to be the service load and the deflection is 2.1 mm.

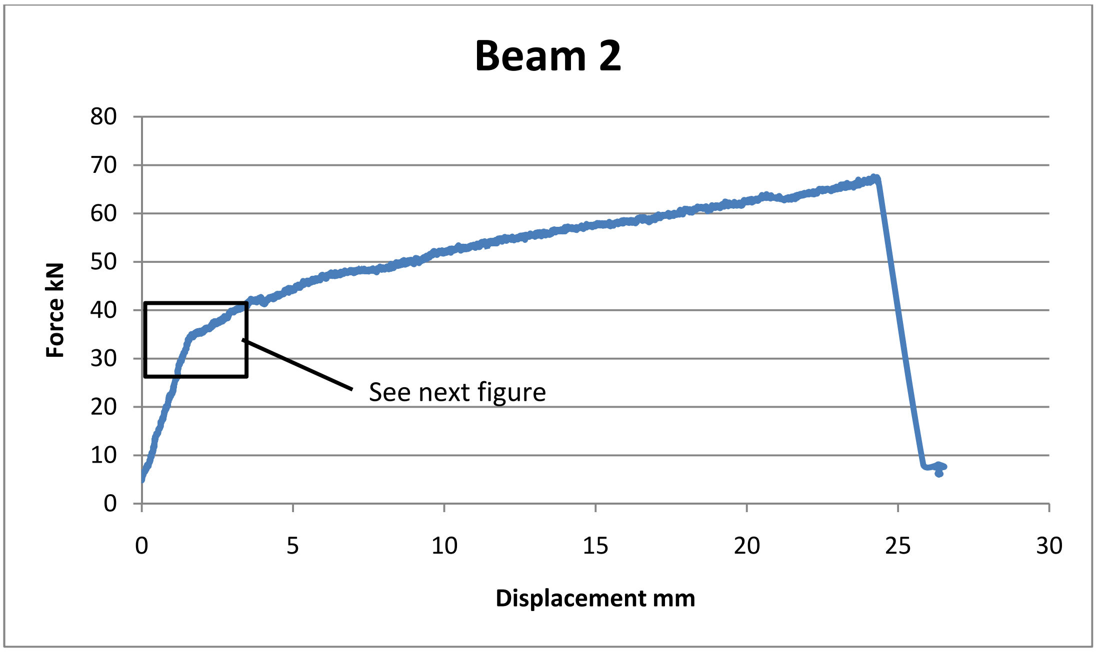

Figure 4-3: Force-displacement relationship, beam 2

This figure shows the force vs. displacement for beam 2. Maximum force was 67 KN. This was a shear failure. The displacement is around 24 mm.

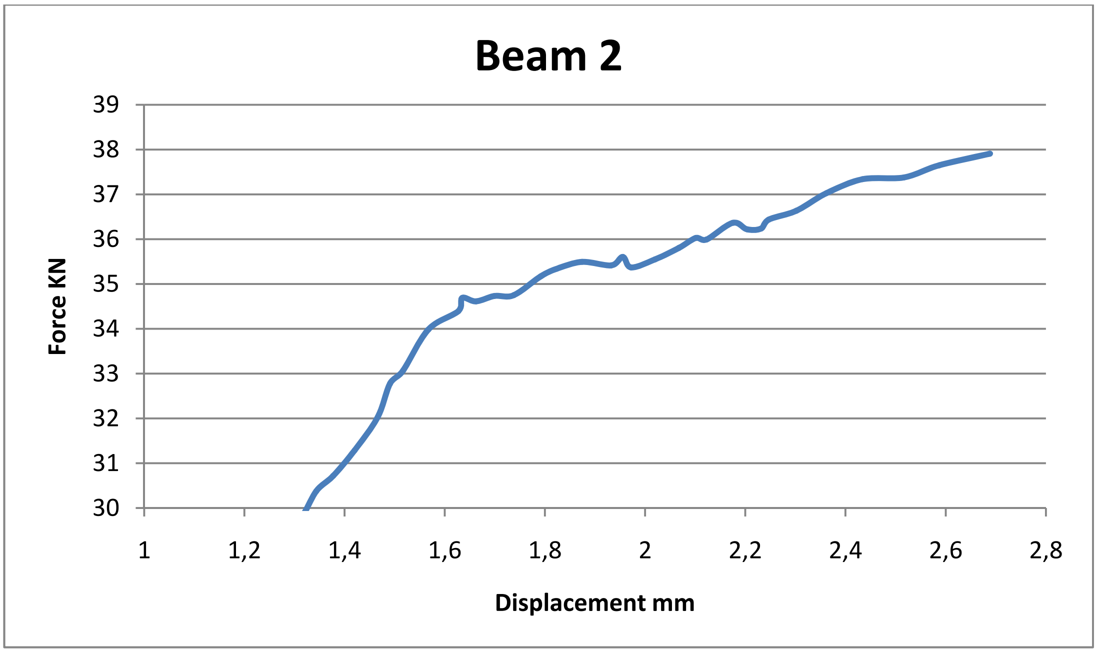

Figure 4-4: Force-displacement relationship, beam 2, zoomed up

Here is the figure zoomed up around the point where the concrete first cracks. As can be seen it is 35 KN force. This was the strongest beam. This is then the service load for this beam and the displacement at this stage is around 1.8 mm.

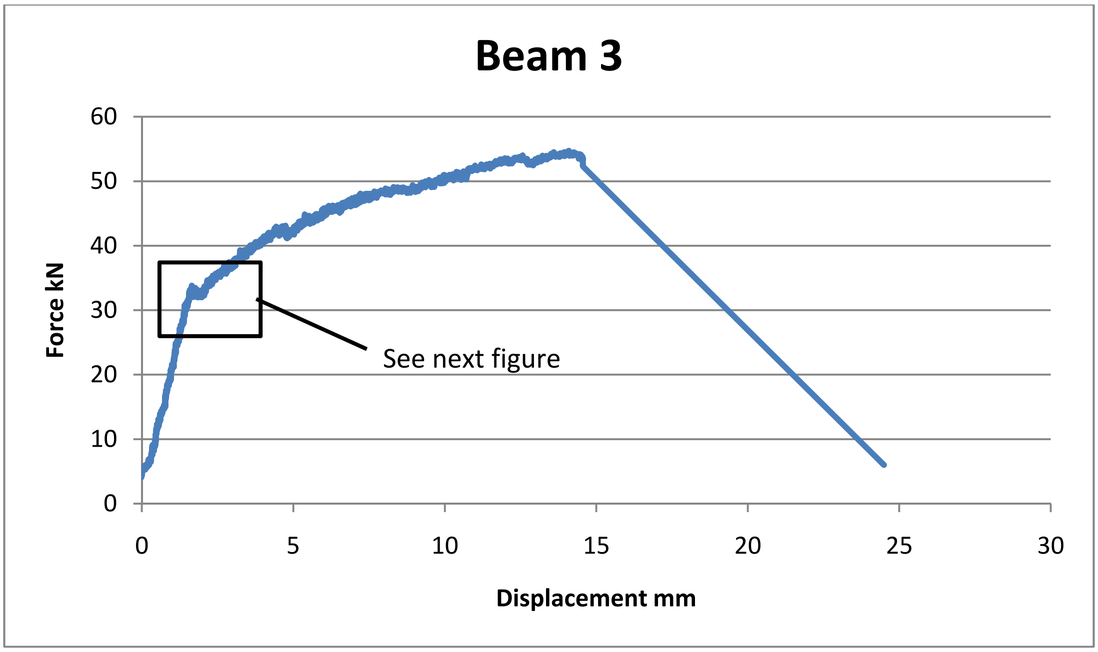

Figure 4-5: Force-displacement relationship, beam 3

This figure shows the force vs. displacement for beam 3. Maximum force was 54 KN. This was a shear failure. The displacement is around 15 mm.

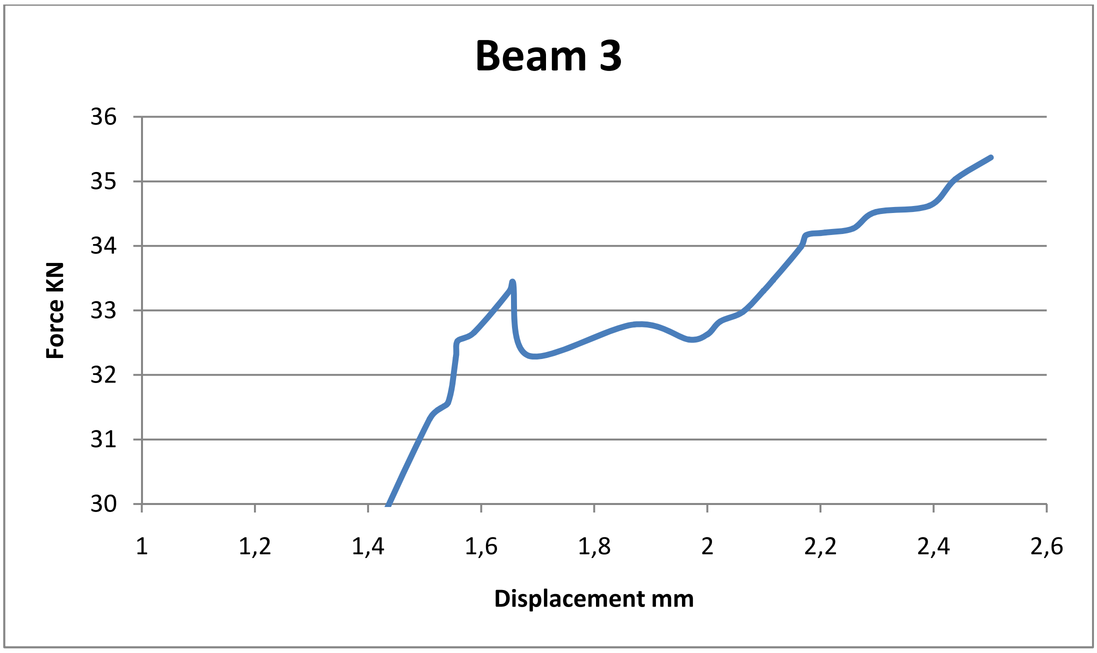

Figure 4-6: Force-displacement relationship, beam 3, zoomed up

Here is the graph zoomed up around the point where the concrete first cracks. As can be seen the force is between 33-34 KN. This is then the service load for beam 3 and the displacement is 1.7 mm.

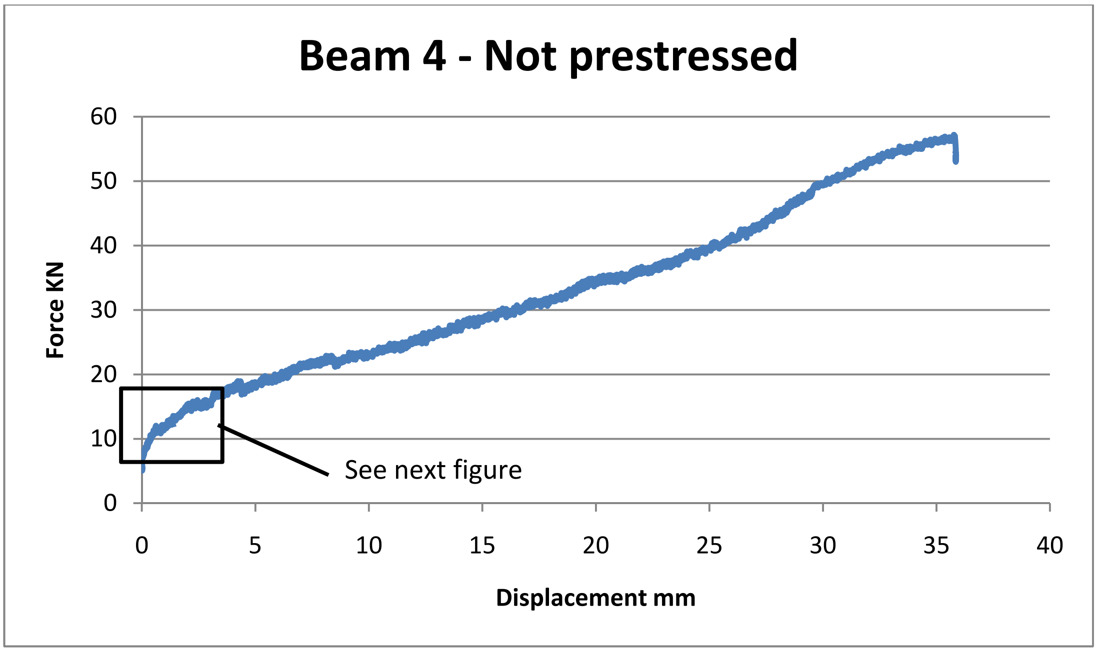

Figure 4-7: Force-displacement relationship, beam 4

Beam 4 was not prestressed and it did not collapse totally like the others. The maximum force is 58 KN and then the beam was at maximum displacement available. The failure was first in bending and later there came shear cracks.

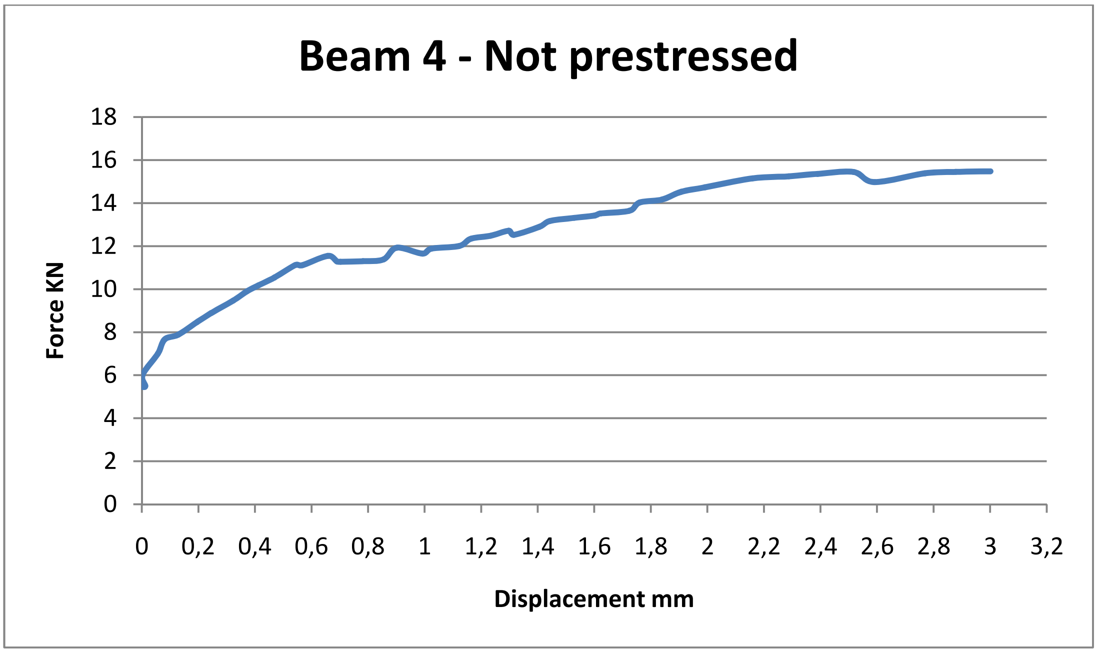

Figure 4-8: Force-displacement relationship, beam 4, zoomed up

Here is the graph zoomed up around the point where the concrete cracks. As can be seen it is little bit less than 8 KN force when it first cracks. Another crack can clearly been seen at 12 KN, that is almost same force (13 KN) as was noted while breaking the beam, then clear bending crack was marked on the middle of the span. The deflection in SLS is

0.2-0.8 mm. The ultimate displacement is far more in this beam than the prestressed ones. Here it is about 36 mm. The beam did not collapse, it bent a lot and the cross section got very cracked though it did not totally collapse. The elastic lengthening in the tendons were too much for the concrete.

Figure 4-9 shows force vs. displacement for all the beams on one graph.

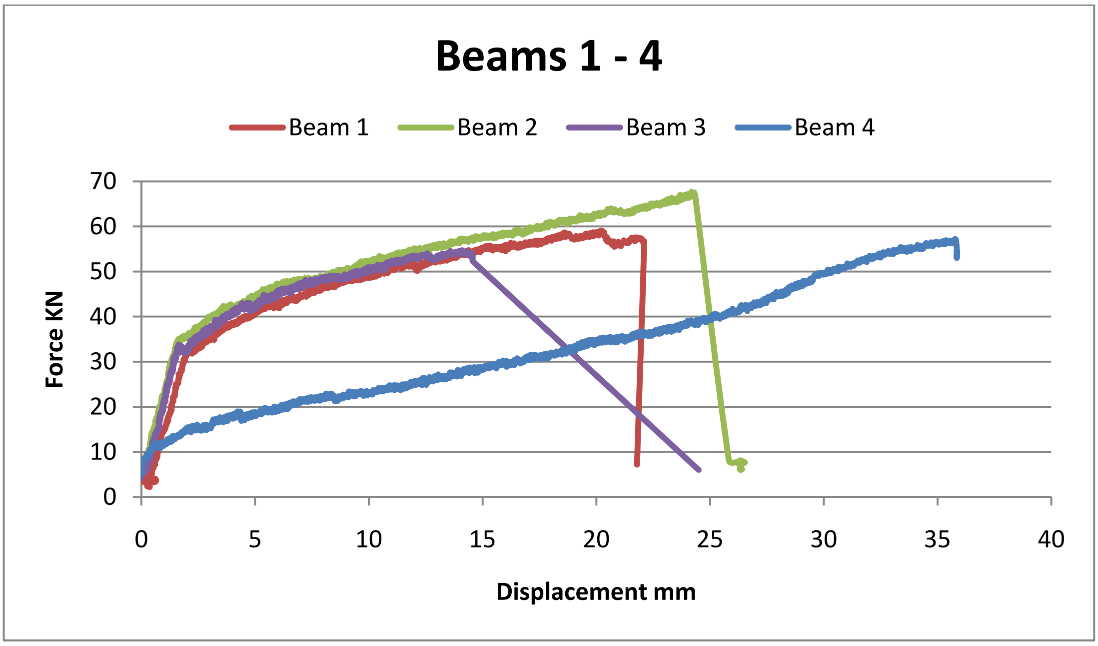

Figure 4-9: Force-displacement relationship, beams 1-4

Here are pictures of the tested beams.

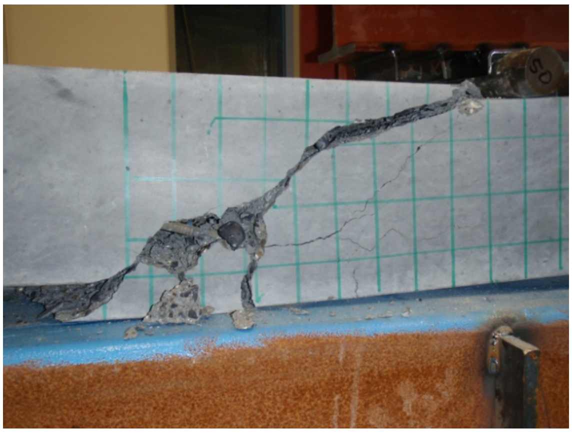

Figure 4-10: Beam 1

Figure 4-11: Beam 2

Figure 4-12: Beam 3

Figure 4-13: Beam 4, un-prestressed

4.1.1 Discussion

These tests show that the service load resistance does increase if the tendons are prestressed. It also reduces the deflections. The ultimate load is similar, though it should be mentioned that the concrete in the un-prestressed beam was totally failed even though the beam did not collapse. Considering the Icelandic building regulations where the maximum displacement for beams is limited to L/400 for the combination of dead and live load. Then the maximum displacement, for these beams with span 1840 mm, is 4.6 mm. From that it can be seen that the prestressed beams are stiff enough because the displacement at SLS was around 2 mm in all the beams. And the ultimate deflection was around 10 times the SLS displacement and 5 times the allowable displacement so there should be enough warnings before collapse of a structure even though the tendons don't have a definite yield point. This is in harmony with the previously introduced research done by Zou (2005). It is recommended to do further studies on flexural strength of beams with prestressed BFRP tendons and then the cross section need to be lower or the span longer to get clear flexural failure.

Stress distribution from; axial prestress, eccentricity of prestress and bending, should give tension in the bottom face of the beam equal to the mean tensile strength of the concrete (Figure 4-14). The mean tensile strength of the concrete is calculated according to table 3.1 in the European standard (*EN 1992-1-1*, 2004), f_{ctm} is 4.4 Mpa for C60 and 2.4 for C23.

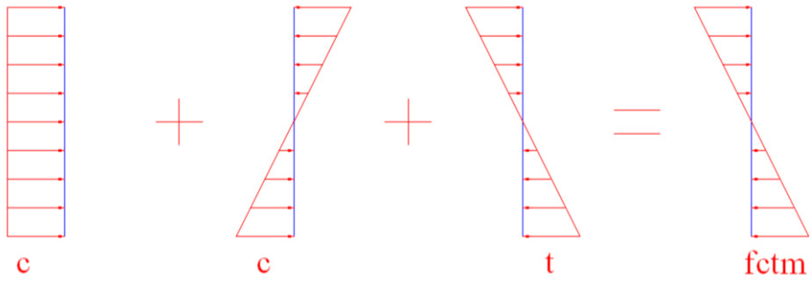

Figure 4-14: Stress in beam due to; compression force, tendon eccentricity, applied load = mean tensile strength of concrete

It is assumed that maximum service load is when the concrete cracks. These calculations give 30.2 KN force for the prestressed beams and 8.2 KN for beam 4 which was not prestressed. Calculating SLS for prestressed beam with concrete C23, max force is 25.7 KN. That is close to 3 time higher SLS resistance of the prestressed beam than of the un-prestressed beam.

According to the calculations (appendix C) the ultimate moment for un-prestressed beam (beam 4) is 9.1 KNm and that gives 11.4 KN force.

Table 3 shows the test results and calculated values for these four beams.

	Test, F		Theory, F			Test, displacement		L/400 limit
	SLS, KN	ULS, KN	SLS, KN	ULS, KN	ULS, KN	SLS, mm	ULS, mm	mm
Beam 1	33	59*	30.2	71.1	84.6***	2.1	22	4.6
Beam 2	35	67*				1.8	24	
Beam 3	33	54*				1.7	15	
Beam 4	8 - 12	58**	8.2	11.4		0.2-0.8	36	

Note: * shear failure
** beam was pushed to the foundation, both shear and flexural failure
*** F due to expected shear strength of concrete

Table 3: Test result, force and displacement

Calculated shear strength varies a lot depending on which method is chosen to calculate the shear strength of concrete. ULS: F=84.6 KN is what was expected at the beginning, but considering the result of shear strength presented in Table 1 and the tests results, the shear strength was overestimated. The shear strength of beams 1-3 varies from 27-33.5 KN ($F_{Test}/2$).

4.2 Strain gauge – stressing and relaxation

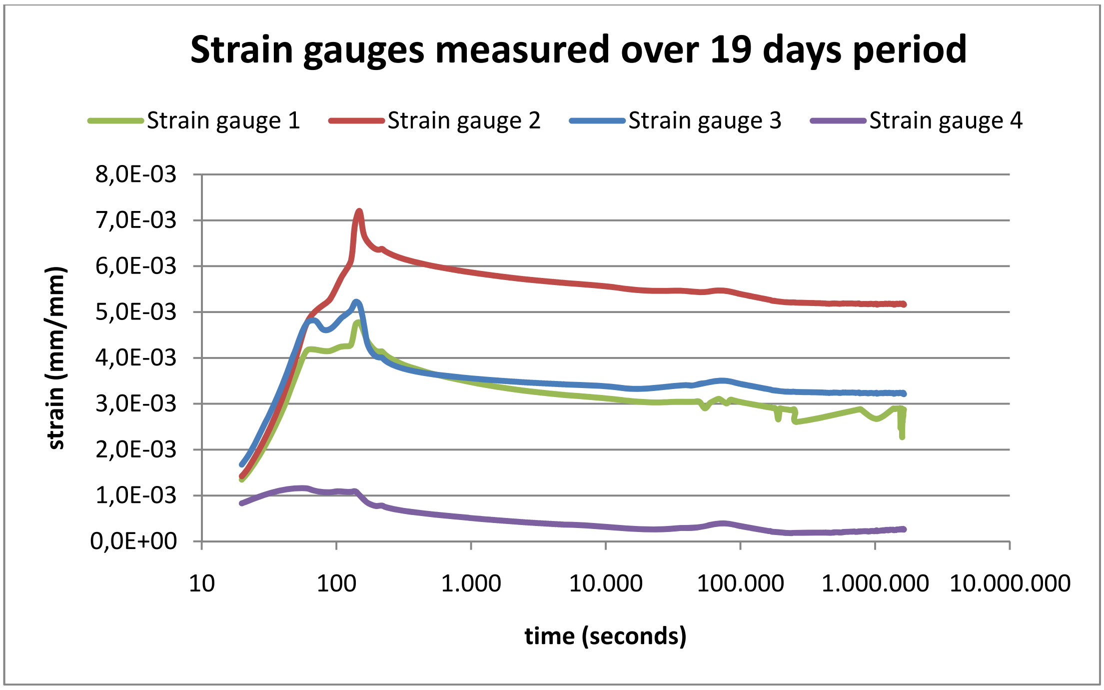

Figure 4-15: Strain in tendons

Here are the strain measurements on the prestressed basalt tendons over 19 days (Figure 4-15). Strain gauge 1 was in beam 1, strain gauges 2 and 3 were in beam 2 but gauge nr. 3 was the only one on the tendons on the other side. Strain gauge 4 was in beam 3. (Strain gauges 1, 2 and 4 were on the same combined tendon.) Strain gauge 4 was all the time with values that was much lower than other gauges, although it seems to follow similar curve. It was only gauges 2 and 3, both in beam 2, which measured all the time without noticeable errors. The values from gauge 1 and 4 showed very much error and were considered not usable even though it looks similar as the other on the graph.

The strain loss is calculated from 300 s. (ca. 5 minutes) and to the end of this log period. Strain gauge 1 showed 25% loss, strain gauge 2 showed 15.8% loss, stain gauge 3 showed 13.9% loss and strain gauge 4 showed 60% loss. For further investigation and speculations only gauges 2 and 3 were used.

4.2.1 Discussion

Trying to simulate the long-term strain loss by calculating trendline for these curves gives some clue about how much the loss will be. But it is necessary to measure strain loss over longer period and it is also recommended to prestress basalt tendon and measure the strain without casting concrete around the tendon. Heat from the concrete and the risk of damaging the gauge while casting can lead to errors in the measurements. As can be seen gauge 1 has a “cracked” curve after ca. 1 day with some errors over the time period but it measured “correctly” occasionally. The time scale is logarithmic and there can be seen that after about 100.000 seconds or ca. 1 day, when the heat in the concrete is at the highest level because of curing, the curve in all cases goes a little bit up and then goes to a similar level when the temperature drops.

Because gauges 1 and 4 showed both some errors the measurements were not used for further investigation. The data from gauges 2 and 3 were used to estimate the long-term relaxation for prestressed BFRP tendon. As can be seen on Figure 4-15 the strain drops quite much on the first seconds after the pulling stopped. That could be because of the relaxation in the hydraulic jack before it was welded. Therefore the estimation is based on the curve after the top and to the end of the logging period. Excel was used to find a equation that can describe the plotting curve.

For gauge 2: $y=-8\cdot10^{-5}\cdot\ln(x)+0.0064$ with $R^2=0,9874$ which is quite good. Where x is the time in seconds and y is the strain after x time. Using x=3.000.000.000 seconds (approximately 95 years) the strain loss is assumed to be 20.6%. (~21%)

For gauge 3: $y=-4\cdot10^{-5}\cdot\ln(x)+0.0037$ with $R^2=0,9222$ which is not as good as the other. For the same time, 95 years, the stain loss is assumed to be 20.4% (~20%).

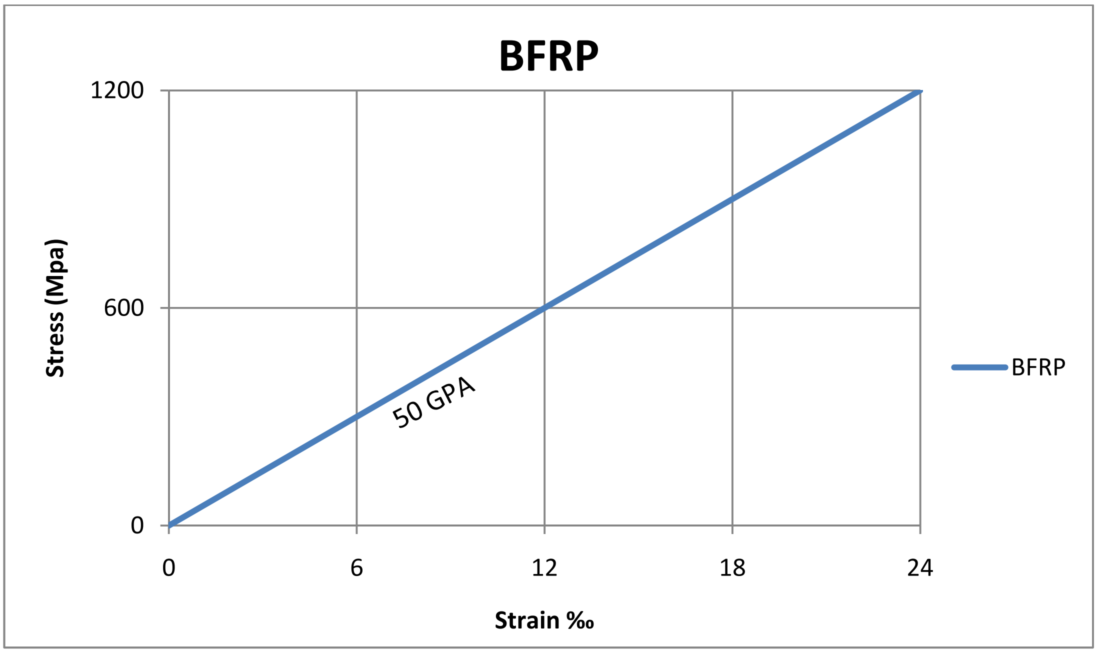

Figure 4-16: Theoretical stress/strain relationship for BFRP used in this experiment

In theory with the Elastic modulus 50 GPa as the manufacturer guarantees, the strain at 600 Mpa should be 12 ‰ (Figure 4-16). As can be seen on Figure 4-15 the measured values were lower (~5-7‰). Reasons for that are not clear but the strain gauges were glued to an aluminium plate, which was then fastened to the tendon with steel “glue” and these can be factors that cause loss or inaccuracy. There are several factors that can cause that the strain gauges are not measuring from the “same zero point”, for example if the aluminium plate bend a little bit then the gauge is in negative strain in the beginning. Then it is possible that the steel "glue" caused some loss. As can be seen on Figure 4-15 there is a crack in all the strain curves in similar area while the stress is still increasing. That could be because the steel "glue" cracked.

Calculated strain according to the 70 mm lengthening of the tendons measured while prestressing is 10.6‰. That is taking the length 7500 mm minus 900 mm which is the connections where the tendons were glued inside the steel pipes, that part is considered to be stiff. Further study on the relaxation of BFRP tendons are recommended and find out work procedure to minimize the possibilities of errors.

4.3 Strain while cutting the tendons

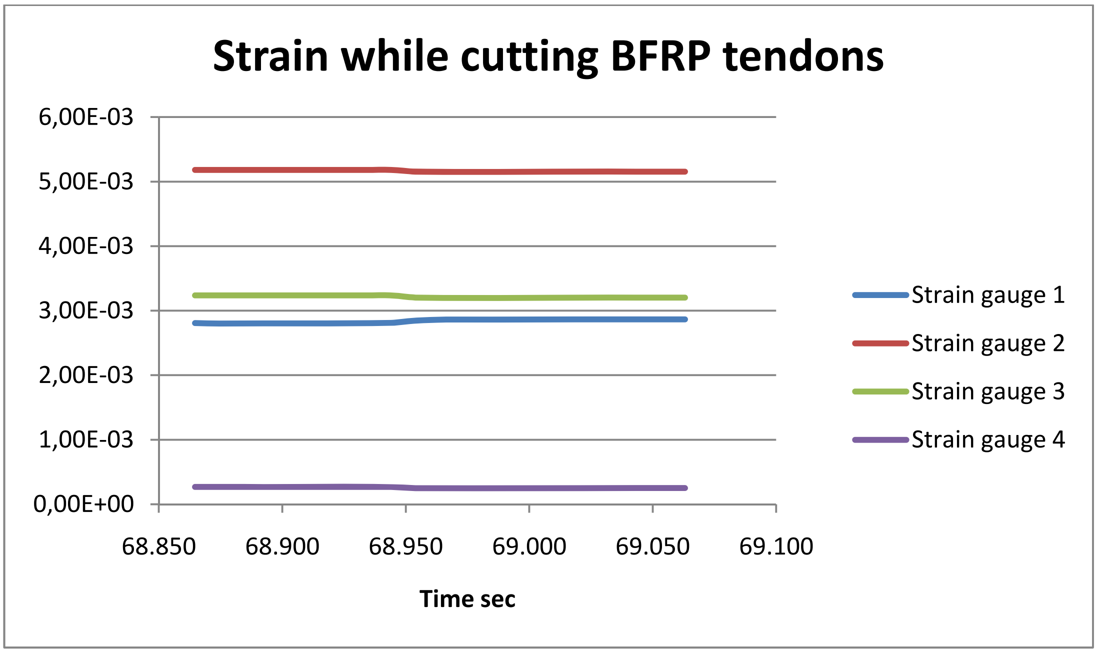

Figure 4-17: Strain while cutting tendons

Strain measured while cutting the BFRP tendons on the 19th day since the prestressing and casting took place, two days before breaking the beams. The time scale is in seconds but it is not measured from beginning, it is relative time in that log session.

On next two figures the strain drop for gauges 2 and 3 is zoomed up. The percentage of strain loss in gauge 3 (Figure 4-18) is maximum 1.27% and after about 300 seconds (5 minutes) when the strain curve has recovered the strain loss measured is 0.98%. The percentage of strain loss in gauge 2 (Figure 4-19) is maximum 0.59% and after about 300 second (5 minutes) when the strain curve has recovered the strain loss measured is 0.45%.

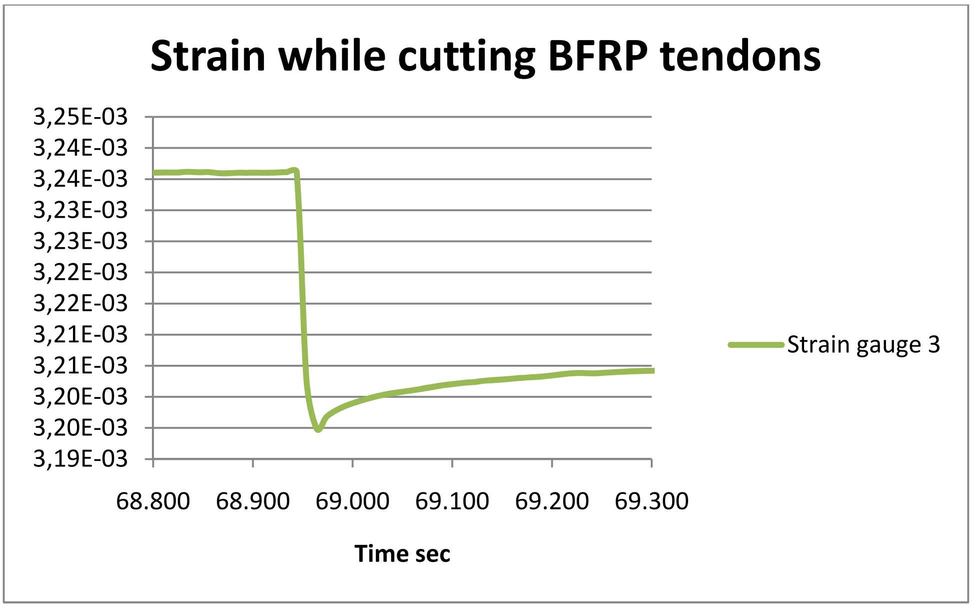

Figure 4-18: Cutting, strain gauge 3

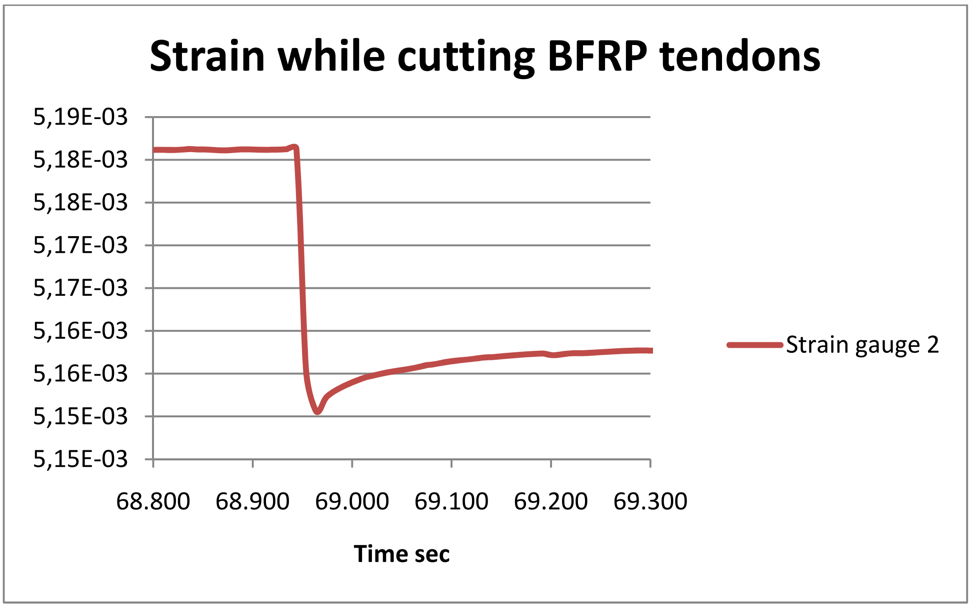

Figure 4-19: Cutting, strain gauge 2

4.3.1 Discussion

At the time when the tendons where cut there was some drop in the strain measurements. From that it can be seen that there were elastic shortening in the tendons. But it is not very much. It was tried to measure how much the beam bend upwards in the middle but it was not noticeable. The beams "jumped" apart (Figure 4-20) and the measure equipment was disturbed. The first cut was between beam 1 and 2, beams 2 and 3 were therefore still stuck together and as can be seen on Figure 4-21 the fastening plate (the joint) on the end moved ca. 2 cm backward.

Figure 4-20: Cut tendons

Figure 4-21: Fastening point moved back

On Figure 4-20 can be seen how much the beams moved apart when the prestressd tendons were cut.

Figure 4-22: Cutting the prestressed tendons

4.4 Stain while breaking beam

Here are the strain gauge measurements when breaking the beams. Because of technical problems the only beam where both the tendons and the concrete were measured was beam 3. The values on horizontal axis of the figures below are number of measuring values and on vertical axis is the strain (mm/mm). It was desired to combine in one figure; force measurements from the hydraulic jack and the strain measurements but since the force and strain are measured with two individual computers it was unfortunately not possible to synchronize these two measurements.

Figure 4-23 shows the strain in tendons and the strain on the tension zone of the concrete (bottom).

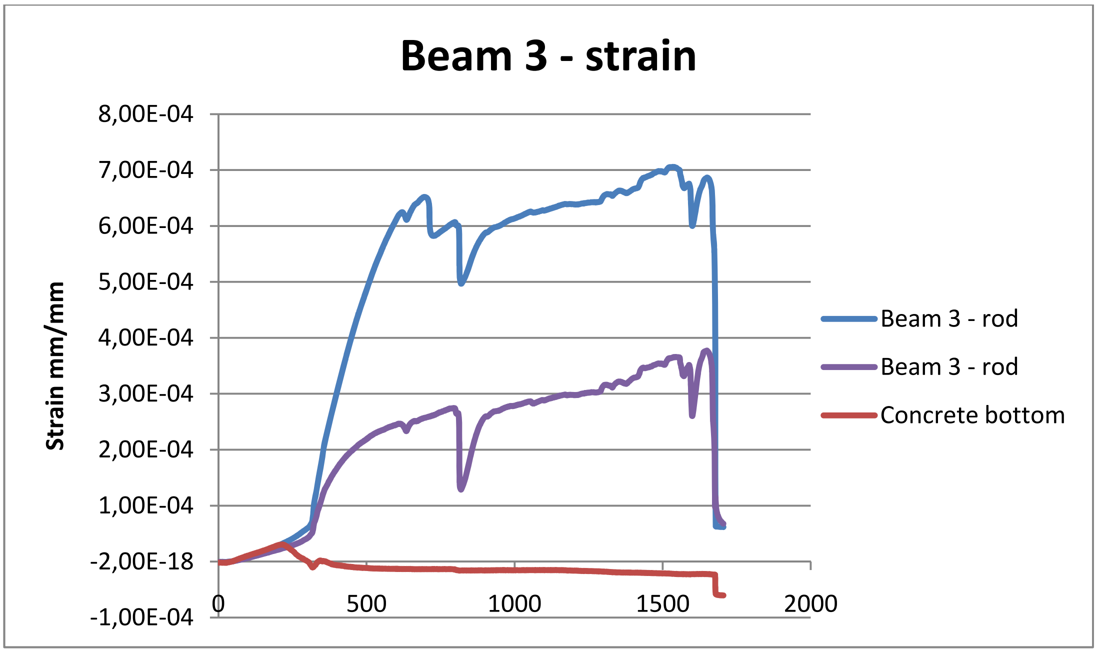

Figure 4-23: Strain when breaking beam 3

On Figure 4-24 is the strain on the compression zone of the concrete (upper part), where it can be seen that the compression strain increases almost linearly up to 2.6‰ when the beam fail.

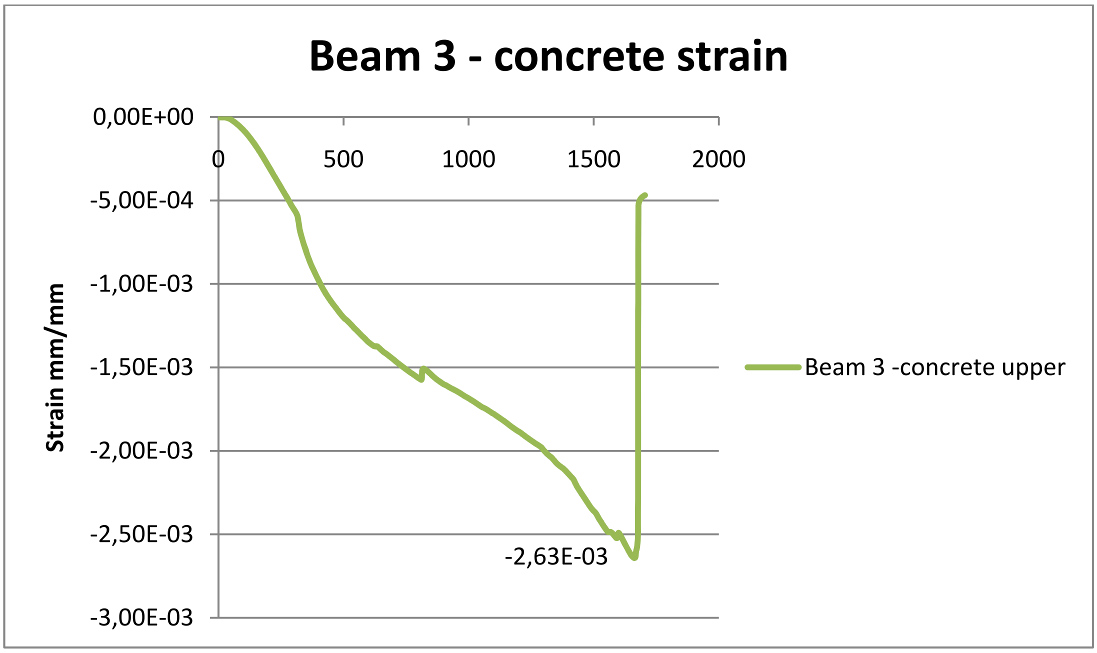

Figure 4-24: Beam 3, strain in compression zone

Strain increase in tendons in beam 2 is 0.7‰ in one tendon and in the other 1‰.

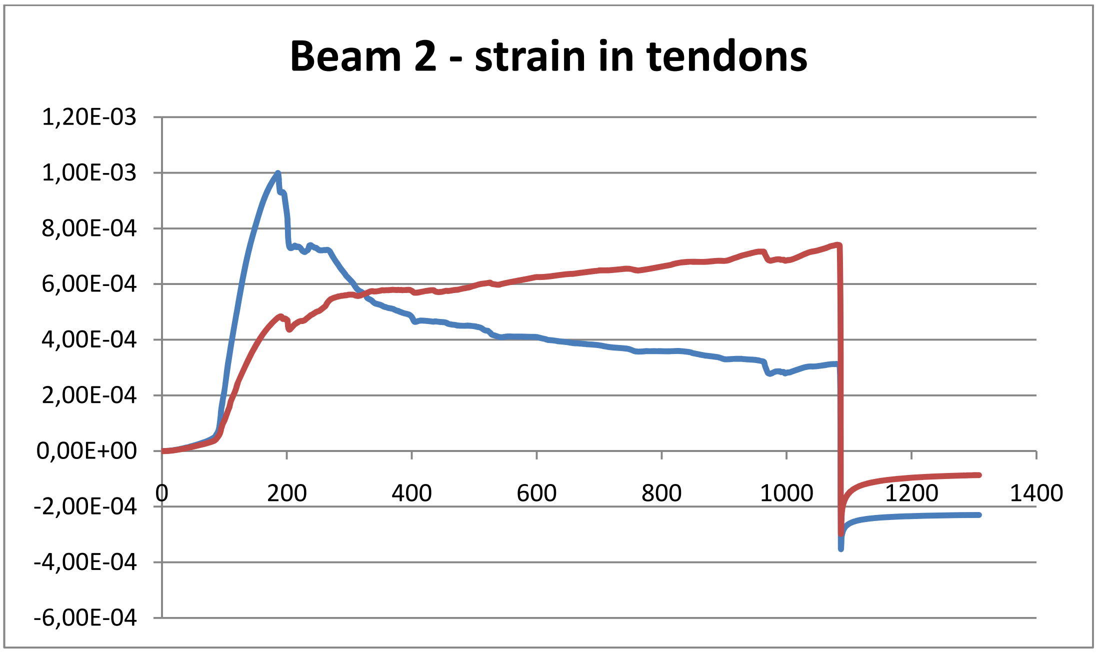

Figure 4-25: Beam 2, strain in tendons

The strain was just plotted for one tendon in beam 1, the other strain gauge failed and did not measure.

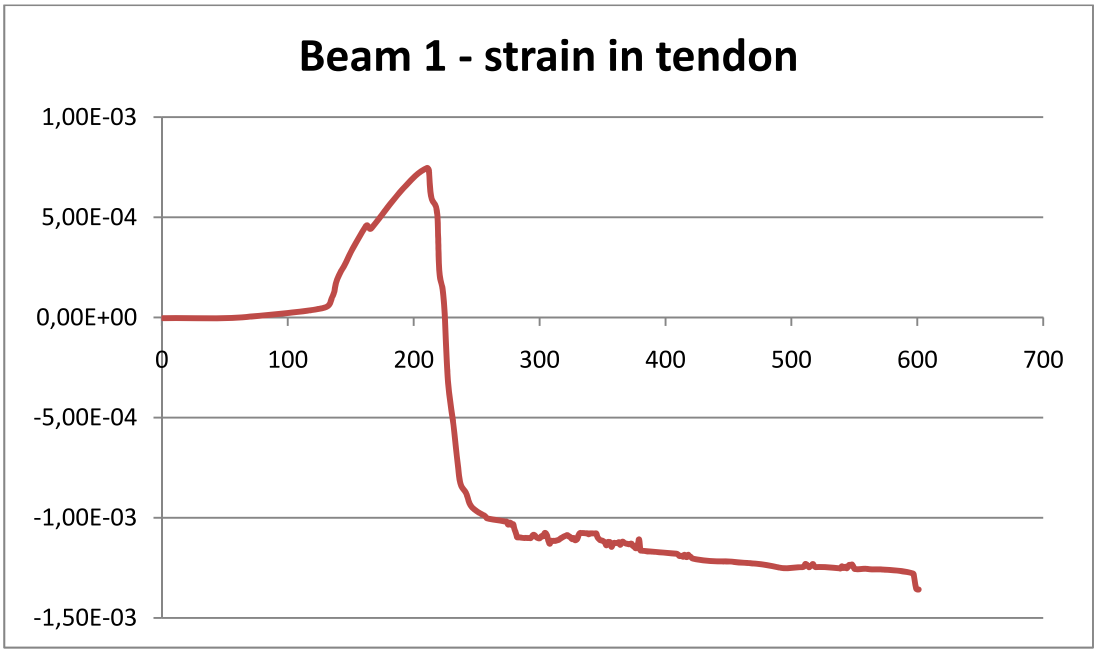

Figure 4-26: Beam 1, strain in one tendon

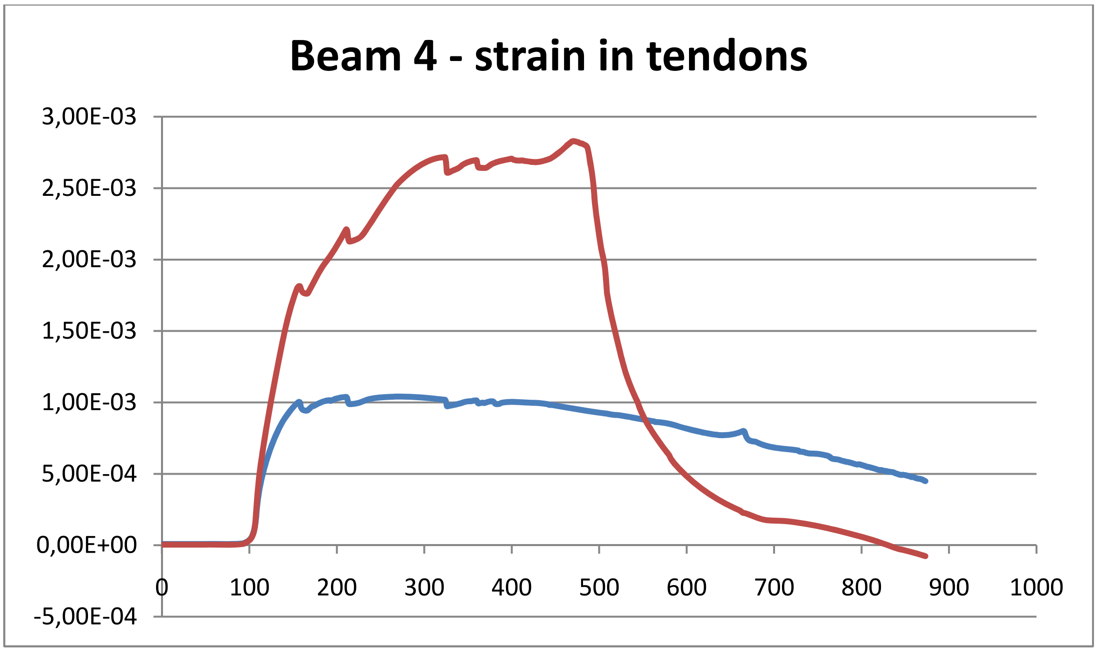

Figure 4-27: Beam 4, strain in un-prestressed tendons

4.4.1 Discussion

Unfortunately it was only possible to measure the strain in the concrete in one beam. The maximum compressive strain in concrete was 2.6‰ that is a little under (very close though) the ultimate compressive strain according to EC2 for C60 concrete. This measurement is considered to be reliable. The concrete had not crushed in the compression zone when the beam failed due to shear failure.

As can be seen on Figure 4-23 the strain in concrete at the bottom of the beam follows the same curve as the tendons in the beginning. Then probably the strain gauge loosens or is damaged because of tension cracks. The strain in the tendons increases until the beam begins to fail. The tendons in beams 1-3 were all prestressed, so the strain that is shown on the figures above is additional strain. Considering the strain because of prestress and adding the strain when beaking the beams it is giving ~10‰. That is only half of the ultimate theoretical strain according to Figure 4-16. And even though the prestress strain is considered 12‰ (Figure 4-16 at 600 Mpa) the additional strain, because of flexure while breaking, is very low.

On Figure 4-27 is the tension strain in un-prestressed tendons plotted. The blue curve seems to be disturbed, but the red curve is showing the strain 2.7‰. Calculating the strain gives values between 2-2.7‰, according to the length of the tendon and deflection of the beam and estimated lengthening of the tendon because of buckle. According to Figure 4-16 where theoretical strain of BFRP tendon is shown, the un-prestressed beam could have deflected much more before reaching maximum strain capacity of the tendons. But the concrete did not have much strength reserve.

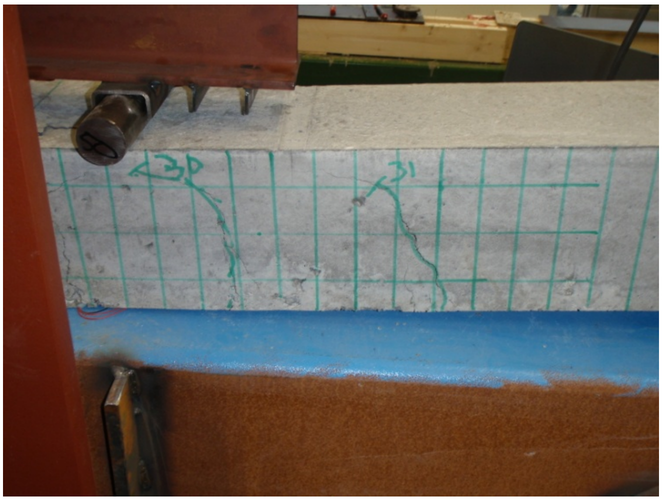

Figure 4-28: Beam 4-Shear, flexure and compression cracks

4.5 Shear strength

The prestressed beams broke in shear failure (Figure 4-29). Shear strength in beam nr. 1 was 29.5 KN, shear strength in beam nr. 2 was 33.5 KN and shear strength in beam nr. 3 was 27 KN. The angle of the shear cracks was measured. In beam 1 the angle was approximately 21°, for beam 2 the angle was 20° and for beam 3 the angle was 20°.

Figure 4-29: Beam 1, shear failure

4.5.1 Discussion

It was unexpected that the beam failed due to shear failure. Afterwards, after studying the literature, evidence was found about reduction of concretes shear strength if the longitudinal reinforcement is FRP tendon instead of steel. All the prestressed beams had similar shear cracks and the angle of the cracks that caused the collapse was in all cases close to 20°. The literature talks much about linear elastic behaviour of FRP failure and that FRP have a brittle failure. The literature does also mention that the so called dowel action of FRP tendons is less than steel tendons because of lower transverse resistance. Therefore it was quite unexpected that the tendons did not fail when the concrete failed. As can be seen on Figure 4-30 the tendons are still holding the two parts together. In the cracked area the tendons seems to have sprained, similar to steel tendon that have yielded.

The unstressed beam did fail in combination of shear and flexural failure. The angle of the shear cracks in beam 4 was much bigger. It varies from 40°-48°. This difference of angle of shear cracks in prestressed and un-prestressed beams is in harmony with what was presented in the literature review (chapter 2.4).

Considering all the methods for calculating shear strength that were introduced in the literature review, the equation that were mentioned there as authors´ modification which is based on EC2:2004 and fib 40 is giving promising results of shear strength of concrete beams without shear reinforcement and with prestressed FRP longitudinal reinforcement.

$$V_{Rd,c} = (C_{Rd,c} \cdot k\left(100 \cdot \rho_I \cdot \frac{E_f}{E_s} \cdot \phi_\varepsilon \cdot f_{ck}\right)^{1/3} + k_1 \cdot \sigma_{cp})b_w \cdot d$$

Equation 4-1: Authors' modification based on EC2 and fib40 equations

This modified equation is used to calculate shear strength of the beams and is giving following results for C23 and C60 concrete, assume the prestress loss is 0% and 20% and then un-prestressed beam.

V_{cf}	Prestress loss		Untensioned	
Concrete	0%	20%		
C23	24.0	21.9	13.4	KN
C60	29.0	26.9	18.4	KN

Table 4: Calculated shear strength of concrete - V_{cf}

The strain loss over the curing period is approximately 15%, Calculating shear strength for prestressed C60 beams with that loss gives V_{cf} =27.5 KN. Comparing to test result 27-33.5 KN

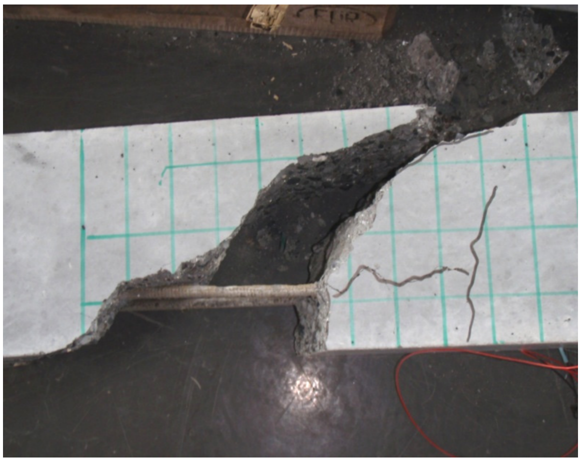

Figure 4-30: Shear failure - tendons

Figure 4-31: Shear failure - tendons bent

5 Discussion in general about the whole work

Failure mode of the beams prestressed by BFRP tendons was due to shear failure. The first cracks were due to bending in the constant maximum moment zone. The beam which was not prestressed failed due to combination of bending and shear. First cracks occurred at the constant moment zone and then came shear cracks. The un-prestressed beam did deflect a lot and the displacement capacity of the equipment was not enough for the beam to collapse, but the cross section cracked a lot and was totally ruined. The tendon has so much elastic lengthening capacity that when the load was released the beam did go up in the middle almost at horizontal level again.

One of the problems that concrete structures suffer from is corrosion of reinforcing steel. It reduces the durability of concrete structures and cost high amount of money every year. FRP tendons can replace steel and there is BFRP very strong candidate because of good mechanical properties. (Brik, 2003)

One factor that was not studied here is the cost of BFRP reinforcement. Many factors can be taken into account when considering the cost of fiber reinforcement compared to steel reinforcement. Manufacturers state that 1 ton of basalt tendons provide the performance of 9.6 ton of steel rebar because of higher tensile strength and lesser density. This is perhaps little bit overestimated by manufactures, at least if additional shear reinforcement is needed. Despite these speculations BFRP can replace several tons of steel reinforcement. A brief survey shows the price of steel per ton was 740 US dollar in January 2011 and was rising all the year before. In January 2010 the price was about 550 US dollar per ton. So the price increase was 35% over the last year. BFRP cost 6000-8500 US dollars per ton. That is approximately 10 times higher per ton than steel but considering the statement that 1 ton of BFRP tendon can replace 9.6 tons of steel, it can be stated that the price is comparable and steel price is still increasing but BFRP price would probably decrease if more of it would be fabricated. Other factor that should be considered when cost analysis is done is the maintenance cost when steel begins to rust. Shipping cost is another factor that can be considered, fewer tons of reinforcement for particular structure reduces the shipping cost. These cost studies need more research since they are relevant when taking decision about using BFRP instead of steel reinforcement.

The Chinese abstract (Gan Yil et al., 2009) introduced in the beginning presented that SLS for prestressed beam is higher than for un-prestressed beam but ULS is equal. That is similar to the results from this experimental work. The ultimate force applied was similar in both cases. It is though not easy to say when the un-prestressed beam had reached its ultimate load. It did not collapse but it was ruined. So it can be argued when the beam has reached its ultimate force. The load that caused the first crack is the primary limit state when designing prestressed beam. It is desirable to have the cross section without cracks. The first crack load was three times higher for the prestressed beam than for the un-prestressed one. For prestressed beams like the one that was investigated here the sercive load should be limited a little under the first crack load. The ultimate load was around twice as high as the first crack load. So the global safety factor is close to two.

6 Summary and conclusions

This experiment was very challenging for me as a student and I have learnt many things about structural design, about research work both during the experimental work and while writing the thesis. Many questions have risen and some of them have been answered, others are still hanging out there and the enthusiasm to do research work increased while this work was in progress.

The main topic this study was supposed to answer was: How much is the ultimate resistance of a prestressed BFRP reinforced concrete beam? And estimate the relaxation of the prestressed BFRP tendons.

The conclusions that can be drawn from this study:

- Ultimate bearing resistance of a beam with prestressed BFRP tendons is not much higher than of un-prestressed beams but the SLS bearing resistance is much higher and the deflection is smaller
- The long-term relaxation (100 year) is estimated around 20%. That is comparable with aramid fibers but much higher than for steel and carbon fibers.
- Special care should be taken when designing members without shear reinforcement.

The SLS bearing capacity of the prestressed beams was triple compared to un-prestressed beam.

It is suggested to use Eurocode 2/fib 40 modified equation to calculate shear resistance of concrete $V_{Rd,c}$ in members that are prestressed

$$V_{Rd,c} = (C_{Rd,c} \cdot k\left(100 \cdot \rho_I \cdot \frac{E_f}{E_s} \cdot \phi_\varepsilon \cdot f_{ck}\right)^{1/3} + k_1 \cdot \sigma_{cp})b_w \cdot d$$

In fib 40 the ratio between both Elastic modulus and strain of FRP and steel is considered. This equation is a modification, using the prestressing factor according to EC2. With reference to the literature about shear strength further studies and research on shear strength of prestressed beams are recommended. There seems to be a debate in the academic community how to calculate the shear strength of concrete.

7 Recomendations

7.1 Further reaserch

While doing this experimental research and writing this thesis a few questions have been answered and other questions have risen. Here are few topics suggested as future experimental research:

- The long term behaviour of prestressed BFRP tendons and the loss of prestress force. It would be interesting to prestress tendons and not cast concrete around to minimize the factors that can affect the strain gauges.
- Shear strength of prestressed BFRP reinforced beams or slabs without shear reinforcement.
- Designing anchorages for BFRP tendons so it will be possible to prestress the tendons. Because of the low transverse strength it is difficult to clamp the ends of FRP tendons.
- Investigate the bond between concrete and BFRP tendons and find out whether it is necessary to have anchor plates to fasten the tendon at the ends of the beams.

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9 Appendix

A. Rockbar informations

Tecnical information from the manufacturer of the BFRP-tendons that were used in the experimental work.

RockBar

Corrosion resistant basalt fibre reinforcing bars

RockBar is a range of basalt fibre composite reinforcing bars for use in Concrete, Mortar and Cast Stone

The properties of **RockBar** include:

- Excellent chemical and corrosion resistance
- 3.7 times lighter than steel and stainless steel
- 2.5 times stronger in tensile strength than steel and stainless steel
- Over 60 times less thermally conductive than steel and over 20 times less thermally conductive than stainless steel
- Non magnetic
- Electrically non-conductive

Environmental performance of **RockBar** includes:

- 40% lower global warming impact that stainless steel
- No waste production during manufacture
- Basalt is one of the most common rock types in the Earth's crust

MagmaTech

RockBar technical information	
Length	Stock lengths are 2.5m and 4m. Cutting to required lengths is possible.
Nominal Diameters	3mm, 4mm, 5mm, 6mm, 7mm, 8mm, 10mm, 12mm Other diameters available on request.
Stock	Currently in stock in the UK: 3mm diameter bar (1000m / pack of 2.5m length) 5mm diameter bar (400m / pack of 2.5m and 4m length) 8mm diameter bar (150m / pack of 2.5m length, 180m / pack of 4m length) 10mm diameter bar (125m / pack of 2.5m length, 120m / pack of 4m length) 12mm diameter bar (75m / pack of 2.5m length, 80m / pack of 4m length)
Composition	Basalt fibre reinforced polymer (BFRP) bar with a sanded finish to aid bonding to mortar.
Tensile strength	1200 MPa
Elastic Modulus	50 GPa.
Bond Strength	10 – 30% higher bond strength than ribbed stainless steel rebar in pull out tests from Cast Stone.
Durability	Durability tests which model the alkali environment of concrete have been completed at Sheffield University The report concludes that; “The estimated environmental strength reduction factor for a period of 100 years wet concrete conditions is 1.25 which corresponds to a strength retention of 79.6 %”.
Sustainability	A life cycle analysis has been conducted at Imperial College London. The report concludes that; “The production of stainless steel bars emits ~170% more CO_2 than the BFRP bars”.
Coefficient of Thermal Expansion	2 x 10^{-6} 1/k (in the longitudinal direction)
Thermal Conductivity	0.7 W/K·m

MagmaTech

www.magmatech.co.uk
contact: Ben Williams
Tel: +44(0)7916155822
Fax: +44(0)8701236392
ben.williams@magmatech.co.uk
PO Box 59162 ▪ London ▪ NW2 9HD

B. HILTI

Hilti HIT-RE 500
with HIT-V / HAS

Hilti HIT-RE 500 with HIT-V / HAS

Injection mortar system

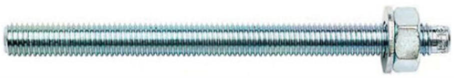
Hilti
HIT-RE 500
330 ml foil pack

(also available as 500 ml and 1400 ml foil pack)

Statik mixer

HAS rod

HAS-E rod



HIT-V rod

Benefits

- suitable for non-cracked concrete C 20/25 to C 50/60
- high loading capacity
- suitable for dry and water saturated concrete
- under water application
- large diameter applications
- high corrosion resistant
- long working time at elevated temperatures
- odourless epoxy
- embedment depth range:
  from 40 … 160 mm for M8
  to 120 … 600 mm for M30

Concrete | Small edge distance and spacing | Variable embedment depth | Fire resistance | Corrosion resistance | High corrosion resistance | European Technical Approval | CE conformity | Hilti anchor design software

Approvals / certificates

Description	Authority / Laboratory	No. / date of issue
European technical approval a)	DIBt, Berlin	ETA-04/0027 / 2009-05-20
Fire test report	IBMB, Braunschweig	UB 3565 / 4595 / 2006-10-29 UB 3588 / 4825 / 2005-11-15
Assessment report (fire)	warringtonfire	WF 166402 / 2007-10-26 & suppl. WF 172920 / 2008-05-27

a) All data given in this section according ETA-04/0027, issue 2009-05-20.

C. Calcutations

Cross section

	h (mm)	b (mm)	A (mm^2)	W (mm^3)
Beam 1	198,2	199,4	39.514	1.305.074
Beam 2	199,25	198,75	39.601	1.315.081
Beam 3	200,75	198,75	39.899	1.334.956
		Ave :	39.671	1.318.370
Beam 4	200,25	199,25	39.900	1.331.656

$$\sigma = \frac{M}{W} \Rightarrow M = \sigma \cdot W$$

Calculating SLS – for prestressed beams and un-prestressed beam. F is the load from jack.

Beam 1-3, service load

f_{ck} =	60,4	Mpa
f_{cm} =	68,4	Mpa
f_{tcm} =	4,4	Mpa > C50/60
ΔP =	20%	loss of prestress force
P =	94	KN, prestress force
A =	39.671	mm^2, average
e =	50	mm
W =	1.318.370	mm^3, average
σ =	1,9	Mpa, P/A
σ =	2,9	Mpa, Mp
σ =	9,1	Mpa, Mb
F =	30225	N
a =	795	mm
F =	**30,2**	**KN**

Beam 4, service load

f_{ck} =	23,2	Mpa
f_{tcm} =	2,44	Mpa < C50/60
W =	1.331.656	mm^3
a =	795	mm3
F =	8175	N
F =	**8,2**	**KN**

If the concrete in prestressed beams is considered as C23, F = 25.7 KN, close to 3 times higher than for un-prestressed beam.

Calculating the ultimate bending strength of a FRP reinforced beam. These calculations are according to ACI440.1R-03 and the book *Reinforced concrete design with FRP composites* (GangaRao et al., 2007, pp. 236-238). Mn calculated is Maximum moment that the beam can resist. This calculations are not taking prestress into account

ACI-4401R-03 Guide

f_{fu}	1200	N/mm^2	ACI440	
C_E	1	Table 7.1	ACI440	
f_{fu}*	1200	Mpa	ACI440	
$f_c´$=	60	Mpa		
d =	150	mm		
h_{beam} =	200	mm		
b_{beam} =	200	mm		
cover	35	mm		
stirrup	10	mm		
Ø =	10	mm		
A_f =	157	mm^2		
n_{rod} =	2			
ρ_f =	0,005236			
ε_{cu} =	0,0029			
E_f =	50000	Mpa		
f_f =	888	N/mm^2	f_f<f_{fu}	Í lagi
β1 =	0,65			

ρ_{fb} =	0,0030	ρ_f <ρfb	FALSE
1,4ρ_{fb} =	0,0042	ρ_f >1,4ρfb	TRUE
1,4ρ_f =	0,0073	Then ϕ	0,7
ϕ =	0,7	ACI 440 eq. 8-7	

Mn = 20,0 KNm

ϕMu = 14,0 KNm

For C23 concrete Mu is 9.1 KN.

This gives F (load): For C23 – F=11.4 KN and for C60 – F=17.6

D. Concrete

Properties of the concrete from Mannvit Engineering Research laboratory.

Hverjir prófsteyptu:

MANNVIT
VERKFRÆÐISTOFA

Forskrift
(rakt fylliefni)

Forskrift nr.
HR-C50-01

Verkefni
Basalttrefjaverkefni
Verkkaupi
HR
Nafn
Eyþór R Þórhallsson / Björgvin
Vegna
C50/60 fyrir súlur

Verk nr.
7.009.285
Dagsetning
28.01.2011
Framkv. af
SvSv

Hönnunarforsendur				**Efniseiginleikar**		v/s-hlutfall	0.36
Umhverfisflokkur	XC3/ Tæring vegna kolunar			Þrýstiþol (sív.) [MPa]	57.4	Loft [%]	5.0
Styrkleikaflokkur	C50/60	Sigmálsfl.	S3	Bindiefni [kg/m³]	450	Sigmál [mm]	140
Lágmark sement [kg/m³]	400	Loft [%]	5.0	Sement [kg/m³]	450	Hitastig [°C]	16
Hámark v/s-hlutfall	0.55	Klórflokkur	Cl 0,2	Vatn [kg/m³]	162	K.bol.S [MPa]	34.0

Athugasemdir

Blöndunarhlutfall — **Blöndu stærð [m³]** **0.117**

Bindiefni	Flokkur	Jafngild Sement	Rúmþyngd [kg/m³]	Blöndunarhlutfall [w. %]	Blöndunarhlutfall [kg/m³]	**Blanda [kg]**	
Aalborg Rapid	CEM I	1.00	3200	100.0	450	**52.7**	52,7

Vatn	158	**10.1**

Fylliefni	Raki [w.%]	ASS [m²/kg]	Mettivatn [%]	Rúmþyngd [kg/m³]	Blöndunarhlutfall (SSD) [v. %]	Blöndunarhlutfall (SSD) [kg/m³]		**Blanda (rakt) [kg]**	kg	ctm
Víkurfjara 0/4mm	10.20	4.42	1.20	2850	45	829	4	**105.7**	5,454	105,7
Víkurfjara 4/32 mm	1.40	0.38	1.30	2900	35	656	3	**76.9**	4,047	76,9
Bj.perlumöl 545-13	2.50	0.44	3.20	2800	20	362	2	**42.1**	2,87	42,1

Íblöndunarefni	Þurrefni [w. %]	Rúmþyngd [kg/m³]	Blöndunarhlutfall [w.% b.]	Blöndunarhlutfall [kg/m³]	**Blanda [kg]**	
Kemloft K19	4.50	1010	0.01	0.03	**0.00**	6
Kemflot KKl 20	22.00	1046	1.00	4.50	**0.53**	

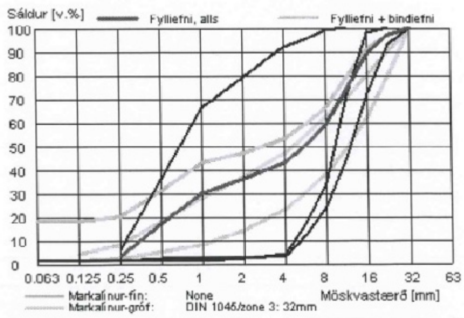
Klukkan:
Vatn í sement: ____________ sigmál mælt kl. ____________________

2461 **287.9**

Sigmál: 19,5 cm Hiti: 22,8°

Loft: 4.0%

Rúmþ.: 24020



Eiginleikar

Sem. jafngildi	1.000
Vatn/bindiefni - hlutfall	0.36
Efja + loft [v.%]	35.3
Fínefni <0.25 mm [kg/m³]	510
WRI	0.863
ASS (áætlað) [m²/kg]	2.25
ASS (raun) [m²/kg]	2.21
Heildar raki [%]	8.43

ComPose5.01
MANNVIT/KI

Skrá :
HR501-C50-01.cpm

Prentað:
28.1.2011 - 13:33

Örbylgjuþurrku

Athugasemdir: Bætt við 3 ltr. vatn
— — 337gr. flot

Bakki: ____________

Bakki+rök steypa: ____________

Bakki+þurr steypa: ____________

Hverjir prófsteyptu: ______ ______

MANNVIT VERKFRÆÐISTOFA

Forskrift
(rakt fylliefni)

Forskrift nr.
HR-C50-02

Verkefni **Basalttrefjaverkefni**		Verk nr. **7.009.285**
Verkkaupi **HR**		
Nafn **Eyþór R Þórhallsson / Björgvin**		Dagsetning **28.01.2011**
Vegna **C50/60 fyrir súlur**		Framkv. af **SvSv**

Hönnunarforsendur				**Efniseiginleikar**		v/s-hlutfall	0.36
Umhverfisflokkur	XC3/ Tæring vegna kolunar			Þrýstiþol (siv.) [MPa]	57.4	Loft [%]	5.0
Styrkleikaflokkur	C50/60	Sigmálsfl.	S3	Bindiefni [kg/m³]	450	Sigmál [mm]	140
Lágmark sement [kg/m³]	400	Loft [%]	5.0	Sement [kg/m³]	450	Hitastig [°C]	16
Hámark v/s-hlutfall	0.55	Klórflokkur	Cl 0,2	Vatn [kg/m³]	162	K.bol.S [MPa]	34.0

Athugasemdir

Blöndunarhlutfall — **Blöndu stærð [m³]** **0.117**

Bindiefni	Flokkur	Jafngild Sement	Rúmþyngd [kg/m³]	Blöndunarhlutfall [w. %]	Blöndunarhlutfall [kg/m³]	**Blanda [kg]**
Aalborg Rapid	CEM I	1.00	3200	100.0	450	**52.7**

Vatn		
	158	**10.1** → 13.1

Fylliefni	Raki [w.%]	ASS [m²/kg]	Mettivatn [%]	Rúmþyngd [kg/m³]	Blöndunarhlutfall (SSD) [v. %]	Blöndunarhlutfall (SSD) [kg/m³]		**Blanda (rakt) [kg]**		
Víkurfjara 0/4mm	10.20	4.42	1.20	2850	45	829	4	**105.7**	5,588	105,7
Víkurfjara 4/32 mm	1.40	0.38	1.30	2900	35	656	3	**76.9**	3,841	76,92
Bj.perlumöl 545-13	2.50	0.44	3.20	2800	20	362	2	**42.1**	2,613	42,1

Íblöndunarefni	Þurrefni [w. %]	Rúmþyngd [kg/m³]	Blöndunarhlutfall [w.% b.]	Blöndunarhlutfall [kg/m³]	**Blanda [kg]**
Kemloft K19	4.50	1010	0.01	0.03	**0.00** 6
Kemflot KKI 20	22.00	1046	1.00	4.50	**0.53** → 0,867

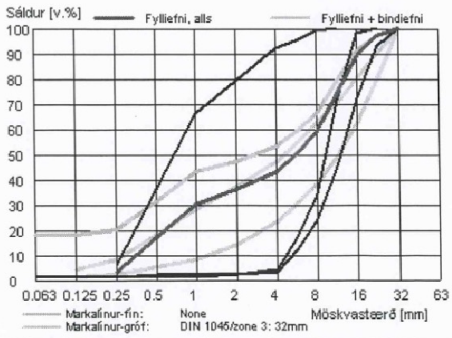
Klukkan:
Vatn í sement: __________ sigmál mælt kl. __________

2461 **287.9**

Sigmál: __________ Hiti: ______

Loft: __________

Rúmþ.: __________



Eiginleikar

Sem. jafngildi	1.000
Vatn/bindiefni - hlutfall	0.36
Efja + loft [v.%]	35.3
Fínefni <0.25 mm [kg/m³]	510
WRI	0.863
ASS (áætlað) [m²/kg]	2.25
ASS (raun) [m²/kg]	2.21
Heildar raki [%]	8.43

ComPose5.01 MANNVIT/KI — Skrá : HR501-C50-01.cpm — Prentað: 28.1.2011 - 13:35

Örbylgjuþurrku

Athugasemdir: __________

Bakki: __________

Bakki+rök steypa: __________

Bakki+þurr steypa: __________

Hverjir prófsteyptu: ________

MANNVIT VERKFRÆÐISTOFA

Forskrift
(rakt fylliefni)

Forskrift nr.
HR-C50-03

Verkefni
Basalttrefjaverkefni
Verkkaupi
HR
Nafn
Eyþór R Þórhallsson / Björgvin
Vegna
C50/60 fyrir súlur

Verk nr.
7.009.285
Dagsetning
28.01.2011
Framkv. af
SvSv

Hönnunarforsendur				**Efniseiginleikar**			
Umhverfisflokkur	XC3/ Tæring vegna kolunar			Þrýstiþol (siv.) [MPa]	57.4	v/s-hlutfall	0.36
Styrkleikaflokkur	C50/60	Sigmálsfl.	S3	Bindiefni [kg/m³]	450	Loft [%]	5.0
Lágmark sement [kg/m³]	400	Loft [%]	5.0	Sement [kg/m³]	450	Sigmál [mm]	140
Hámark v/s-hlutfall	0.55	Klórflokkur	Cl 0,2	Vatn [kg/m³]	162	Hitastig [°C]	16
						K.bol.S [MPa]	34.0

Athugasemdir

Blöndunarhlutfall — **Blöndu stærð [m³]** **0.117**

Bindiefni	Flokkur	Jafngild Sement	Rúmþyngd [kg/m³]	Blöndunarhlutfall [w. %]	Blöndunarhlutfall [kg/m³]	**Blanda [kg]**
Aalborg Rapid	CEM I	1.00	3200	100.0	450	52.7

Vatn		
	158	~~10.4~~ 13,1

Fylliefni	Raki [w.%]	ASS [m²/kg]	Mettivatn [%]	Rúmþyngd [kg/m³]	Blöndunarhlutfall (SSD) [v. %]	Blöndunarhlutfall (SSD) [kg/m³]		**Blanda (rakt) [kg]**		
Víkurfjara 0/4mm	10.20	4.42	1.20	2850	45	829	4	105.7	5,65	105,7
Víkurfjara 4/32 mm	1.40	0.38	1.30	2900	35	656	3	76.9	4,17	76,9
Bj.perlumöl 545-13	2.50	0.44	3.20	2800	20	362	2	42.1	2,71	42,1

Íblöndunarefni	Þurrefni [w. %]	Rúmþyngd [kg/m³]	Blöndunarhlutfall [w.% b.]	Blöndunarhlutfall [kg/m³]	**Blanda [kg]**	
Kemloft K19	4.50	1010	0.01	0.03	0.00	6
Kemflot KKI 20	22.00	1046	1.00	4.50	0.53	0,862

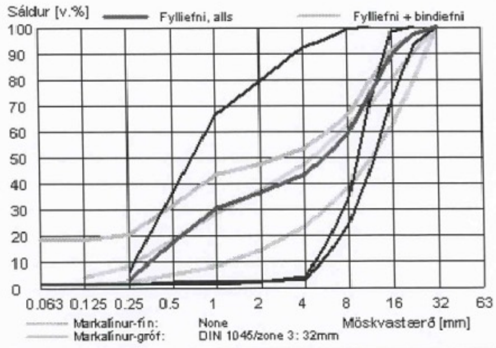
Klukkan:
Vatn í sement: ________ sigmál mælt kl. ________

2461 **287.9**

Sigmál: 25,0 cm Hiti: ______

Loft: 3,0 %

Rúmþ.: ________



Eiginleikar	
Sem. jafngildi	1.000
Vatn/bindiefni - hlutfall	0.36
Efja + loft [v.%]	35.3
Fínefni <0.25 mm [kg/m³]	510
WRI	0.863
ASS (áætlað) [m²/kg]	2.25
ASS (raun) [m²/kg]	2.21
Heildar raki [%]	8.43

ComPose5.01
MANNVIT/KI

Skrá :
HR501-C50-01.cpm

Prentað:
28.1.2011 - 13:35

Örbylgjuþurrku

Athugasemdir: ________

Bakki: ________

Bakki+rök steypa: ________

Bakki+þurr steypa: ________

Þrýstistyrksprófun Sívalninga í CONTROL'S PRESSU

Verkkaupi: Háskólinn í Reykjavík
Verkbeiðandi : 0
Mannvirki: 0
Byggingarhluti: 0

ÞVERMÁLS- OG HÆÐARMÆLINGAR
Aðeins þarf að skrifa í fyrstu dálkana fyrir standardsívalninga. Þegar prófhlutir eru ekki standard sívalningar eða borkjarnar skal mæla þvermáli og hæð skv. ÍST EN 12390-3:2001 Annex B.

Verk-ranns. nr. Basaltverkefni

Dags.: 24.feb dags þegar síðustu sívalningarnir eru brotnir !

Rannsóknaðarmaður, settu stafina þína í grænu reitinn !

Okkar merking	Prófunar-aldur	Steypu-dagur	Dags. prófunar	Þungi í vatni (gr)	Þungi í lofti (gr)	Þvermálsælingar Efri flötur (mm)	(mm)	(mm)	neðri flötur (mm)	(mm)	(mm)	Meðal þvermál (mm)	Hæðarmæling c hlið 1 (mm)	hlið 2 (mm)	hlið 3 (mm)	Meðal hæð (mm)	D-max (mesta stærð)	Álestur á pressu (kN)	(MPa)	Aldur (dagar)	Rúmþyngd (kg/m3)	H/Þ	Flatar-mál (mm²)	Þrýstistyrkur reiknaður (MPa)	Leið-rétti-stuðull	Leiðrétt þrýstistyrkur (MPa)
C25-1	20 daga styrkur	4.02.11	24.02.11	1850	3409,0	99,9	100,0		99,5	99,8		99,8	199,7	199,6		199,7		180,1	23,0	20	2187	2,00	7.823	23,0	1,00	23,0
C25-2		4.02.11	24.02.11	1861	3426,0	100,4	100,0		99,4	99,5		99,8	200,2	200,5		200,4		184,0	23,5	20	2189	2,01	7.827	23,5	1,00	23,5
C25-3		4.02.11	24.02.11	1823	3390,0	100,0	99,6		99,8	99,8		99,8	200,1	200,1		200,1		1804,0	23,1	20	2163	2,01	7.823	230,6	1,00	23,1
	Gert (rannsóknarmaður) :			BSJ																Meðaltal :	2180					23,2
1-A	22 daga styrkur	2.02.11	24.02.11	7795	13088,0	150,1			149,7			149,9	300,0	300,3		300,2		1102,1	62,4	22	2473	2,00	17.648	62,4	1,00	62,4
1-B		2.02.11	24.02.11	7835	13137,0	150,1			150,3			150,2	300,2	300,3		300,3		1095,0	61,8	22	2478	2,00	17.719	61,8	1,00	61,7
1-C		2.02.11	24.02.11	7870	13178,0	149,4			150,1			149,8	301,3	300,6		301,0		1075,7	61,0	22	2483	2,01	17.613	61,1	1,00	61,0
	Gert (rannsóknarmaður) :			BSJ																Meðaltal :	2478					61,7
3-A	22 daga styrkur	2.02.11	24.02.11	7620	12946,0	150,1			150,0			150,1	300,0	300,6		300,3		1080,9	61,1	22	2431	2,00	17.683	61,1	1,00	61,1
3-B		2.02.11	24.02.11	7650	12976,0	150,3			149,7			150,0	300,2	300,9		300,6		1012,5	57,3	22	2436	2,00	17.672	57,3	1,00	57,3
3-C		2.02.11	24.02.11	7585	12873,0	150,1			149,8			150,0	300,3	300,3		300,3		1035,3	58,6	22	2434	2,00	17.660	58,6	1,00	58,6
	Gert (rannsóknarmaður) :			BSJ																Meðaltal :	2434					59,0
	28 daga styrkur																									

Þrýstistyrksprófun Sívalninga í CONTROL´S PRESSU

Verkkaupi:
Verkbeiðandi : 0
Mannvirki: 0
Byggingarhluti: 0

ÞVERMÁLS- OG HÆÐARMÆLINGAR
Aðeins þarf að skrifa í fyrstu dálkana fyrir standardsívalninga. Þegar prófhlutir eru ekki standard sívalningar eða borkjarnar skal mæla þvermáli og hæð skv. ÍST EN 12390-3:2001 Annex B.

Verk-ranns. nr.

Dags.: dags þ

Rannsóknaðarmaður, settu stafina þína í

Okkar merking	Prófunar-aldur	Steypu-dagur	Dags. prófunar	Þungi í vatni (gr)	Þungi í lofti (gr)	Þvermálsælingar Efri flötur (mm)	(mm)	(mm)	neðri flötur (mm)	(mm)	(mm)	Meðal þvermál (mm)	Hæðarmæling c hlið 1 (mm)	hlið 2 (mm)	hlið 3 (mm)	Meðal hæð (mm)	D-max (mesta ...rnastær)	Álestur á pressu (kN)	(MPa)	Aldur (dagar)
C25-1	7 daga styrkur	4.02.11	24.02.11	1850	3409,0	99,9	100,0		99,5	99,8		99,8	199,7	199,6		199,7		180,1	23,02	20
C25-2				1861	3426,0	100,4	100,0		99,4	99,5		99,8	200,2	200,5		200,4		184,0	23,52	
C25-3				1823	3390,0	100,0	99,6		99,8	99,8		99,8	200,1	200,1		200,1		180,4	23,06	
	Gert (rannsóknarmaður) :																	Meðaltal :		
1-A	28 daga styrkur	2.02.11	24.02.11	7795	13088,0	150,1			149,7			149,9	300,0	300,3		300,2		1102,1	62,43	22
1-B				7835	13137,0	150,1			150,3			150,2	300,2	300,3		300,3		1095,0	61,79	
1-C				7870	13178,0	149,4			150,1			149,8	301,3	300,6		301,0		1075,7	61,04	
	Gert (rannsóknarmaður) :																	Meðaltal :		
3-A	7 daga styrkur			7620	12946,0	150,1			150,0			150,1	300,0	300,6		300,3		1080,9	61,08	
3-B				7650	12976,0	150,3			149,7			150,0	300,2	300,9		300,6		1012,5	57,3	
3-C				7585	12873,0	150,1			149,8			150,0	300,3	300,3		300,3		1035,3	58,58	
	Gert (rannsóknarmaður) :																	Meðaltal :		
	28 daga styrkur																			
	Gert (rannsóknarmaður) :																	Meðaltal :		
	Gert (rannsóknarmaður) :																	Meðaltal :		
	Gert (rannsóknarmaður) :																	Meðaltal :		

Mannvit hf.
Rannsóknarstofa
Grensásvegur 1, 108 Reykjavík

Útgáfa: 5.0
Áb.maður: I-Hló

Þrýstistyrksprófun Sívalninga í CONTROL´S PRESSU

Verkkaupi:
Verkbeiðandi : 0
Mannvirki: 0
Byggingarhluti: 0

ÞVERMÁLS- OG HÆÐARMÆLINGAR
Aðeins þarf að skrifa í fyrstu dálkana fyrir standardsívalninga. Þegar prófhlutir eru ekki standard sívalningar eða borkjarnar skal mæla þvermáli og hæð skv. ÍST EN 12390-3:2001 Annex B.

Verk-ranns. nr.

Dags.: dags þ

Rannsóknaðarmaður, settu stafina þína í

3426
3390

Okkar merking	Prófunar-aldur	Steypu-dagur	Dags. prófunar	Þungi í vatni (gr)	Þungi í lofti (gr)	Þvermálsælingar Efri flötur (mm)	(mm)	(mm)	neðri flötur (mm)	(mm)	(mm)	Meðal þvermál (mm)	Hæðarmæling c hlið 1 (mm)	hlið 2 (mm)	hlið 3 (mm)	Meðal hæð (mm)	D-max (mesta ...rnastær)	Álestur á pressu (kN)	(MPa)	Aldur (dagar)
C25-1	28 daga styrkur		24-2	1850	3409	99,9	100		99,5	99,8			199,7	199,6						
C25-2			–	1861	3426	100,4	100		99,4	99,5			200,2	200,5						
C25-3			–	1823	3390	100,0	99,6		99,8	99,8			200,1	200,1						
	Gert (rannsóknarmaður) :		BSJ															Meðaltal :		
1-A	28 daga styrkur	2-2	24-2	7795	13088	150,1			149,7				300,7	300,0	300,3					
1-B				7835	13137	150,1			150,3				300,2	300,3						
1-C				7870	13178	149,4			150,1				301,3	300,6						
	Gert (rannsóknarmaður) :																	Meðaltal :		
3-A	28daga styrkur			7620	12946	150,1			150,0				300,0	300,6						
3-B				7650	12976	150,3			149,7				300,2	300,9						
3-C				7585	12873	150,1			149,8				300,3	300,3						
	Gert (rannsóknarmaður) :																	Meðaltal :		
	28 daga styrkur																			
	Gert (rannsóknarmaður) :																	Meðaltal :		
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	Gert (rannsóknarmaður) :																	Meðaltal :		

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