BASALT FIBERS AS NEW MATERIAL FOR REINFORCEMENT AND CONFINEMENT OF CONCRETE

Conference Paper · November 2015

2 authors:

Eyþór Rafn Þórhallsson
Reykjavik University

Jónas Þór Snæbjörnsson
Reykjavik University

Sborník
ISBN 978-80-906097-0-9

BASALT FIBERS AS NEW MATERIAL FOR REINFORCEMENT AND CONFINEMENT OF CONCRETE


Eythor Rafn Thorhallsson [1]
Corresponding Author

Jonas Thor Snaebjornsson

Abstract

Basalt Fibre Reinforced Polymer (BFRP) is a new material in civil engineering and has shown to be a promising material for infrastructure strengthening: In comparison to carbon fiber, glass fiber and other composites, it has some advantages such as high-temperature resistance and low cost. At the Structural and Composite Laboratory at Reykjavik University (SEL) several research projects involving strengthening concrete beams and columns by using FRP materials have been on-going in recent years. These tests have shown improvements in strength and durability compared to unstrengthened concrete members. The benefit of using basalt fiber or other FRP material is that they are non-corrosive which is a good choice for reinforcing concrete structure exposed to de-icing salts, for examples in bridge decks and parking garage elements. Also for concrete exposed to marine environment, such as seawalls, water breaks and buildings or other structures located near a waterfront. Two research projects are presented in this paper; a test of prestressed concrete with internal basalt rods instead of steel and a test of columns strengthened by wrapping fibre-reinforced composite sheets around the columns to increase their strength and ductility. These experimental tests show increasing strength and ductile for both the beams and the columns.

Keywords: Basalt fiber, concrete beam, experimental work, prestress, shear strength, concrete strength, FRP strengthening, BFRP .

1. Introduction

Several research studies on the use of basalt fibre reinforcement polymer (BFRP) have been conducted at the Reykjavik University in collaboration with the Innovation Center Iceland.
Basalt rocks are the main material in basalt bars, basalt fabrics, chopped basalt fiber strands, continuous basalt filament wires and basalt mesh. Some of the potential applications of these basalt composites are: plastic polymer reinforcement, soil strengthening, bridges and highways,

floors, heat and sound insulation for residential and industrial buildings, bullet-proof vests and retrofitting and rehabilitation of structures [1] , [2]

Basalt fiber tendons (BFRP) have a tensile strength in order of 1000 - 1300 MPa. In comparison, regular steel reinforcement has tensile strength around 500 MPa. However, the elastic module of basalt fibers is much lower than that of steel or around 70 GPa. [1], [3]
The energy required for the production of basalt fibers is around 5 KWh/kg, while the production of carbon steel requires about 15 KWh/kg. The density of basalt is nearly three times less than the density of steel. For un-prestressed concrete, the energy gain for each kg of basalt bars instead of steel bars are then approximately 10 KWh/kg. By using strengthening technics with prestressed basalt fibers instead of steel the energy gain could be 30-50 kwh for each kg of basalt fibers used.

2. Prestressed basalt rods for reinforcing concrete beams

As stated, the elastic module for basalt rods are much lower than for steel. This disadvantage has lead to excessive deformations at service limit state compared to that of steel bars if bars of same cross sectional area are used. To overcome this disadvantage prestressed BFRP bars were tested in this study. [4], [5], [6]

2.1. Experimental procedure

Nine beams were prestressed using two 10 mm in diameter BFRP tendons each, delivered by Magma Tech Ireland with the trademark RockBar. The tensile strength for the rods are 1000+ MPa. Each tendon was prestressed to approx. 50% of its ultimate strength as recommended by the ACI guidelines for FRP (ACI, 2006). The concrete strength was determined as the mean concrete strength of 3 tests on concrete cylinders carried out 28 days after concrete casting.

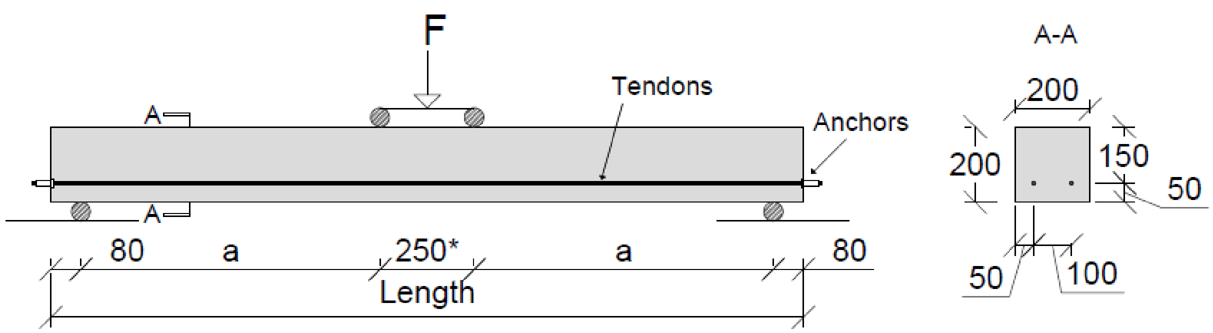

Fig. 1. Schematic drawing (not to scale) of the bending test setup and the beams cross section. The parameters Length and a, are listed in Table 1. The depth, d, was in all cases 150 mm.
* Note that the span length of the bracket was 500 mm instead of 250 mm for the 3860 mm long beam.

The experimental setup and the geometric dimensions of the beam are illustrated in figure 1. Table 1 provides information about beam size and the total length to the point at which the force (*F*/2) is applied. The prestressing force, P, is listed in column 5 of Table 1. All the tested beams

had the same equivalent amount of internal reinforcement and a cross-sectional area A_f = 156 [mm^2]. The bending ratio for all the beams was $\rho = 0{,}0052$. The maximum value of the shear force (F/2), measured in a given test before fracture of concrete, is referred to as V_{exp} in Table 1.
$M_{at\ failure}$ is the maximum bending moment in the beam mid-span calculated by multiplying V_{exp}. (factored) by a. $V_{craking}$. is the shear force at which macro cracking is initiated (in the four-point bending test).
Deflections (Δ [mm]) corresponding to failure load for each beam are listed in Table 1, in the second last column to the right.
Two strain gauges, 30mm in length each, were glued on the compression site (at the top of the section) in the mid-span of the beam. The maximum strains observed are listed in the last column of Table 1. The listed values are in promill (‰).

Table 1. Experimental tested beams. Size, properties and results.

Model	length	a	a/d	f'_c	P	A_f	ρ	V_{exp}.	$M_{at\ failure}$	$V_{craking}$.	Δ	*Strain at top face*
	[mm]	[mm]	-	[MPa]	[kN]	[mm^2]	-	[kN]	[kNm]	[kN]	[mm]	‰
3	1200	395	2,63	67,4	84	156	0,0052	77,5	30,6	33	17	3,25
				67,4	84	156	0,0052	72,0	28,4	38	15	2,90
1	2000	795	5,3	60,4	78	156	0,0052	29,5	23,5	16,5	33	-
				60,4	78	156	0,0052	33,5	26,6	17,5	35	-
				60,4	78	156	0,0052	29,0	23,1	16,5	33	2,60
4	2700	1145	7,63	61,7	84	156	0,0052	23,1	26,4	11,5	53	3,13
				61,7	84	156	0,0052	23,2	26,6	11,5	56	3,25
2*	3860	1600	10,67	57,1	78	156	0,0052	15,8	25,3	9	124	2,50
				57,1	78	156	0,0052	15,5	24,7	9	117	3,00
		Average		61,5	81				26,1			

Two extra beams of same size, with length 3860 mm were tested with external stirrups as shown on figure 2, to prevent diagonal tension shear failure which was observed for all the other beams.

Fig. 2. External stirrups on two extra beams of Model type 2, with length of 3860 mm.

2.2. Test results

Failure mode of the beams prestressed by BFRP tendons was due to bending-shear failure. As shown in Table 1 the failure moment was nearly constant for all of the beams (from 23,1 KNm to 30,6 KNm). The maximum shear force at failure was then different depending on the a/d ratio. This indicates that the a/d ratio has great effect on the shear capacity although most shear equations published in standards and guidelines don’t consider it directly. For the two beams clamped with external shear links, maximum shear at failure reached 17,9 KN instead of 15,7

KN for beams without shear links. These two beams failed at the compression side at the centre of the beam.

3. External basalt-mats for columns

Two groups of columns were prepared and tested. [7]. Group CA contained five columns with a corner radius of 20 mm and group CB contained five columns with a corner radius of 35 mm. All columns are same size 180 x 180 mm and of length 1400 mm. All the columns were reinforced in the same way, using one longitudinal bar of diameter 12mm of steel grade B500C in each corner and hoops of the same steel grade were spaced at 180 mm intervals. The same size of column specimen was used in previous research at SEL to investigate the ductility of reinforced concrete columns with different hoop spacing. Each group contained a column wrapped with one, two and three layers of BFRP jacket and one column without BFRP jacket as a reference. The basalt mats were from BASALTEX of type BAS UNI 600. The epoxy resin was from Sika.

1C, 2C, 3C and 1M indicates the locations of strain gages.

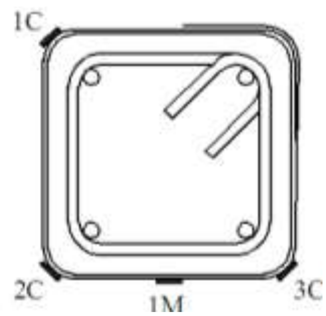

Fig 3: Plain reference column, column with rounded corners and prepared surface, wrapped column in process, BFRP confined column with one, two or three layers. A section cut is shown to the right.

3.1. Test results

The results are summarized by reporting the key parameters at milestones throughout the loading time in Table 2 and Table 3. Each table includes the unconfined axial load capacity at first peak Fc, for the unconfined columns, and Fcc_1 for the confined columns; the confined axial load capacity at the jacket rupture Fcc and the load capacity at the failure of the unconfined columns Fcu. Corresponding strain at Fc is ε_{c1}, at Fcu it is ε_{cu}, at Fcc_1 it is ε_{cc1}, and at Fcc it is ε_{cu}. For comparison of the performance of confined columns and the unconfined columns, the strength ratio Fcc/Fc and the strain ratio $\varepsilon_{cu} / \varepsilon_{c1}$ is shown.

All BFRP confined specimens had similar stress-strain behaviour in the ascending part of loading before the first micro cracks developed since the lateral dilation was very small. This behaviour confirms that the passive confining pressure provided by the BFRP jacket is negligible during the loading state up to the transition zone where micro cracks start to develop.

During the ascending part of loading up to the first peak load, no visual cracks were seen in the concrete cover of the reference columns CA0 and CB0 (columns without basalt strengthening). Right after reaching the peak load (see F_c in table 2), longitudinal bars buckled between hoops

ISBN 978-80-906097-0-9

and the concrete separated in an explosive way. Since there was no plastic deformation in the reinforcement the columns failed in a brittle manner giving no warning of failure. In both of these columns the failure occurred at the top of the centre region between the uppermost hoops spaced 180 mm apart.

Table 2 - Results of unconfined columns.

Column	n	F_c (kN)	F_{cu} (kN)	ε_{c1} (%)	ε_{cu} (%)
CA0	0	1129,5	1111,1	0,346	0,348
CB0	0	1076,7	1068,5	0,343	0,344

Table 3 - Results of confined columns.(n=number of layers)

Column	n	F_{cc1} (kN)	F_{cc} (kN)	F_{cc1}/F_c	F_{cc}/F_c	ε_{cc1} (%)	ε_{cu} (%)	$\varepsilon_{cu}/\varepsilon_{c1}$
CA1	1	1179,4	971,2	1,04	0,86	0,332	0,783	2,26
CA2	2	1169,7	1197,6	1,04	1,06	0,270	0,950	2,74
CA3	3	1310,1	1355,7	1,16	1,20	0,344	1,378	3,98
CA3	3	1310,1	1520,2	1,16	1,35	0,344	1,088	3,14
CB1	1	1127,1	1048,3	1,05	0,97	0,330	0,711	2,07
CB1	1	1127,1	1078,4	1,05	1,0	0,330	0,655	1,91
CB2	2	1193,2	1497,2	1,11	1,39	0,286	1,014	2,95
CB3	3	1209,5	1527,3	1,12	1,42	0,332	2,198	6,40

Increase in compressive strength at first peak was observed in all confined specimens. This is because the BFRP jacket activates in the transition zone where micro cracks developed and is fully activated after reaching the unconfined concrete strength where the cracks developed rapidly. After reaching the load capacity at first peak, all columns except CB3 exhibited a rapid descending slope while the BFRP jacket activated and started to resist the concrete dilation and buckling of the longitudinal bars. With the BFRP jacket fully activated, the load capacity started to increase again in stress-strain behaviour of a hardening response with increase in axial strain. Column CA1 however did not increase its load capacity after the BFRP jacket was fully activated but retained it constant until the jacket ruptured. Instead of a rapid descend slope after first peak, column CB3 kept the axial resistance at peak constant which then further increased until the jacket ruptured. Columns with increased axial load capacity generally reached a second peak where the maximum confined load capacity was reached and first rupture of fibres occurred. Load capacity at ultimate was where the BFRP jacket failed resulting in failure of the columns.

4. Conclusions

All the prestressed beam specimens failed in bending-shear, both for a/d ratio of 2,63 and for a shear span to depth (a/d) ratio of 10,67. In general the shear strength of FRP beams is less than that of steel-reinforced beams as the dowel actions are much lower for the FRP beams. Two

extra beams were reinforced with external steel stirrups to test the bending capacity. Those beams failed in flexure due to compression failure of the concrete and tendon rupture.
BFRP prestressed beams without shear reinforcement are vulnerable to transverse loading even if the a/d ratio is high. Comparison of these experimental results and relevant shear equations showed, that the shear force and bending moment distribution along the span or the slenderness of the beams needs to be considered to determine the shear capacity with accuracy to insure safety.
The experimental results from the confined reinforced concrete columns clearly demonstrated that BFRP wrapping can enhance the structural performance of concrete columns under axial loading. The confinement effect is directly related to the shape of the cross-section. The number of BFRP layers and the corner radius are the major parameters controlling the behaviour of the wrapped concrete columns. To enhance the confined behaviour, the stiffness of the BFRP jacket can be increased by applying additional layers as well as by increasing the corner radius of square columns.

5. Acknowledgements

The authors greatly appreciate the financial support from NordMin, a Nordic Network of Expertise for a sustainable mining and mineral industry funded by the Nordic Council of Ministers. The authors greatly appreciate also the support from the Innovation Center Iceland and Professor Olafur Wallevik. The work done on this topic by Bjorgvin Smari Jonsson, Sindri Gudmundsson, Arngrimur Konradsson and Andri Gunnarsson, a research students at Reykjavik University is acknowledged.

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ISBN 978-80-906097-0-9

Eythor Rafn Thorhallsson
✉ Menntavegi 1, 101 Reykjavik, Iceland
☎ +354 5886200
☺ eythor@ru.is
URL www.ru.is

Prof. Jonas Thor Snaebjornsson
✉ Menntavegi 1, 101 Reykjavik, Iceland
☎ +354 5886200
☺ jonasthor@ru.is
URL www.ru.is