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Influence of steel-polypropylene fibers on fracture parameters of high performance concrete

PIOTR SMARZEWSKI, DANUTA BARNAT-HUNEK

Lublin University of Technology Faculty of Civil Engineering and Architecture,
40 Nadbystrzycka St., 20-618 Lublin, Poland, p.smarzewski@pollub.pl

Abstract. The purpose of this article is to determine the influence of the type of fibers on fracture parameters of high performance concrete (HPC). In this study there were two types of coarse aggregate used: granite and granodiorite with a grain size of about 2/8 mm. Experimental tests were performed on cubic samples, cylinders and notched beams. In the concrete of FRC type, 0.5 and 0.75% of steel fibers and 0.5 and 0.25% of polypropylene fibers respectively, were added. Mean strengths and standard deviations for compression, splitting tensile strength, mean static modulus of elasticity and mean fracture energy were determined. Experimental studies in the 1st fracture model showed that the HPC without the addition of fibers (C), was characterized by brittleness, and fiber concrete (FRC) was more ductile. Fibers were bridging the cracks during loading, delayed hairline cracks and prevented the notched beams from breaking. The shape of the descending curve of the load-deflection depended on the geometry and mechanical properties and the quantity of the fibers used, and in the case of HPC without fiber, on the type of coarse aggregate. In the case of granodiorite aggregate, better mechanical parameters of concrete were observed.
Keywords: high performance concrete, steel fibers, polypropylene fibers, coarse aggregate, fracture parameters
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1. Introduction

High performance concrete (HPC) is a material of not only high compressive strength, but also of excellent durability, so important in terms of sustainable construction. However, the brittleness of concrete increases with increase in its strength. The behaviour of concrete is improved after mixing fibers of high tensile strength, excellent toughness, of short length and of slender shape. It has been proved that fibers having a high modulus of elasticity e.g. steel ones can effectively increase the tensile strength and toughness of concrete [1]. On the other hand, fibers with a low modulus of elasticity e.g. polypropylene ones are good in inhibiting cracks caused by early plastic shrinkage and in reducing shrinkage strains during drying. In order to improve the resistance to cracking and mechanical properties during the entire period of use, a new type of concrete is i.e. hybrid-fiber reinforced concrete is applied, which constitutes a mixture of different fibers.

Many studies have shown that steel and polypropylene fiber hybrid in concrete may benefit from manifold advantages derived from the properties of both fibers, it can effectively improve the condition of the contact between cement and aggregate, and reduce the occurrence and development of hairline cracks in concrete and increase the continuum of the material [2-4]. Such a material has good prospects for future use in underground watertight structures, in engineering of roads and bridges, and structures subjected to seismic actions.

This study evaluated the effect of the type of fiber on the mechanical properties and fracture of HPC. For this purpose, a three-point bending test on notched samples was carried out and experimental fracture energy values calculated up to the limit threshold criteria were compared.

2. Materials and test program

2.1. Details of materials, mixtures design, and sample specifications

The experimental examination was carried out on HPC cubes, cylinders, and notched prismatic beams with a variable content of steel-polypropylene fibers (FRC) or without fibers (C). The following tests were conducted: compressive strength, splitting tensile strength, static modulus of elasticity, and three-point bending tests to determine the effect of two types of fiber and its content on the compressive, tensile behaviour, deflection and fracture parameters of HPC made with or without fibers.

The following components were used in recipes of the concrete mixtures: Portland cement CEM I 52.5 N-HSR/NA, condensed silica fume, coarse aggregates — granodiorite from Ukraine or granite from Graniczna, quartz sand, water, and superplasticizer. The maximum grain size of both coarse aggregates was 8 mm. In order to attain the same workability, superplasticizer based on polycarboxylate ethers was used in four concrete mixtures. A detail characteristic of fibers was shown in Table 1.

Table 1

Detail fiber characteristic

Fiber Type	Symbol	Density [g/cm^3]	Length [mm]	Diameter [μm]	Elastic modulus [GPa]
Steel fiber	ST	7.8	50	1000	200
Polypropylene fiber	PP	0.9	12	25	3.5

The mixtures were prepared using a concrete mixer with a capacity of 100 l. The mixing procedure was as follows: quartz sand and coarse aggregate were homogenized together and mixed with half quantity of water. Then, cement, silica fume and the remaining water were added and finally superplasticizer. After the components had been thoroughly mixed, in FRC mixtures, steel and polypropylene fibers were gradually added to obtain homogeneous and workable consistence. Mixed materials per cubic meter of concrete used in experimental program were shown in Table 2.

Table 2

Mixture proportion

Material	Symbol, unit	C1	C2	FRC1	FRC2
Cement	c [kg/m^3]	670.5			
Quartz sand 0/2 mm	s [kg/m^3]	500			
Granodiorite 2/8 mm	a_1 [kg/m^3]	990	–	990	–
Granite 2/8 mm	a_2 [kg/m^3]	–	990	–	990
Silica fume	sf [kg/m^3]	74.5			
	sf/c [%]	11			
Superplasticizer	sp [l/m^3]	20			
	$sp/(c+sf)$ [%]	2.7			
Water	w [l/m^3]	178			
	$w/(c+sf)$ [–]	0.24			
Steel fiber	w_{fs} [kg/m^3]	–	–	58.5	29.25
	V_{fs} [%]	–	–	0.75	0.5
Polypropylene fiber	w_{fp} [kg/m^3]	–	–	2.25	4.5
	V_{fp} [%]	–	–	0.25	0.5

Molds coated with anti-adhesive oil were filled with a concrete batch and compacted on a vibrating table. After compacting, the samples were covered with foil to minimize the loss of moisture. All samples were stored at temperature of about 23°C until the time of removing them from the molds after 24 hours. They were then placed in a water tank for 7 days to cure. After this period the samples were removed from the tank to cure in laboratory conditions up to 28 days. Sample specifications used in the test program were shown in Table 3.

TABLE 3

Sample specifications

Label	C1	C2	FRC1	FRC2
Fiber type	–	–	Hybrid (S+P)	Hybrid (S+P)
L_f [mm]	–	–	50 /12	50 /12
V_f [%]	–	–	0.75+0.25	0.5+0.5
Number of tested samples				
Compression test	3	3	3	3
Splitting tensile test	3	3	3	3
Modulus of elasticity test	3	3	3	3
Three point bending test	3	3	3	3

2.2. Test equipment and solutions

Compressive and splitting tensile tests were carried out after 28 days on 100 × 100 × 100 mm cubes, according to standards [5, 6]. A servo-hydraulic closed-loop test machine was used.

Test method for static modulus of elasticity of concrete in compression was performed on cylinders of 150 mm diameter and a height of 300 mm, measuring the deformation stress of the sample in the range from 0.5 MPa to 30% of the concrete compressive strength. The examination was conducted by means of a press and using a modulus measuring device with an extensometer according to the recommendations of ASTM [7].

A flexure test under centrically applied load was also performed after 28 days on Axial/Torsional Test System machine, according to RILEM TC 89-FMT [8], using 80 × 150 × 700 mm prismatic samples. In the mid-span of each beam sample, a single notch was done with a thickness of 3 mm and a depth of 50 mm, in order to locate the crack. Before testing, the samples were provided with plaques for fixing the clip gauge thereon. In order to measure the crack mouth opening displacement (CMOD), a strain gauge consisting of elastic plates separated by means of a non-conductive cube (Fig. 1) was used. The tests were conducted by imposing a displacement rate of 0.05 mm/min.

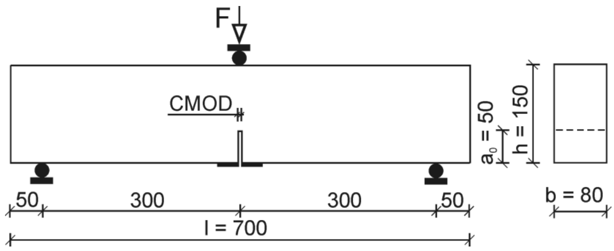

Fig. 1. Sample notched beam geometry and dimensions under centrically bending load

3. Results and discussion

2.1. Compressive and splitting tensile strength

The cube mean values and standard deviation of compressive and splitting tensile strength were given in Fig. 2. The cube compression strength was insignificantly affected by adding ST-PP fibers. It should be noted that in the case of the addition of fibers (ST 0.75% + 0.25% PP), in FRC1 concrete made form granodiorite aggregate, there was a drop in strength by 4% compared to the standard concrete C1. On the other hand, the granite aggregate based concrete showed an increase in compressive strength by 3% after the addition of fibers (FRC2) in an amount of 0.5% ST and 0.5% of PP as compared to concrete without reinforcement C2. The results obtained were highly influenced by greater adhesive strengths between the granite aggregate, cement paste and the fibers. On the other hand, granodiorite aggregate was characterized by higher crushing strength and therefore all mechanical parameters of C1 and FRC1 concretes were respectively higher than the properties of C2 and FRC2 concretes.

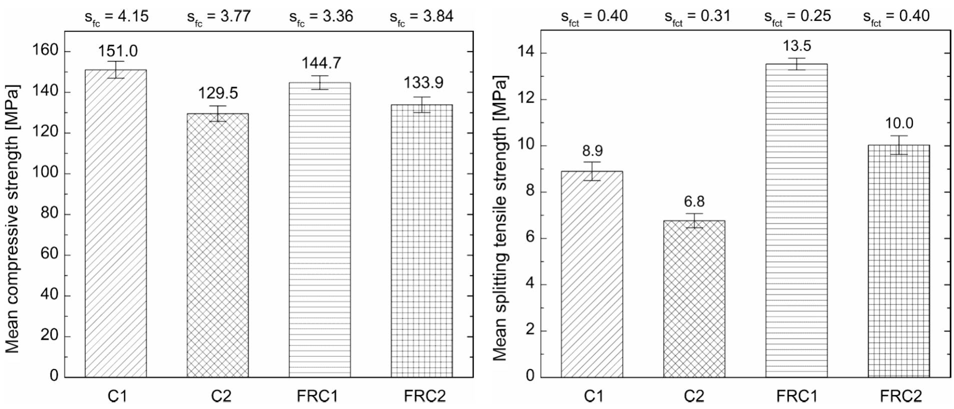

Fig. 2. Cube compressive and splitting tensile strength

The addition of fibers affected significantly the cube splitting tensile strength, however a greater increase was observed at a higher percentage of steel fibers added. An increase in strength was observed for FRC1 by about 52% compared to C1. In the case of concrete FRC2 an increase in splitting tensile strength by 47% was observed compared to that of C2 concrete. C1 and FRC1 concretes made from granodiorite aggregate were characterized by the higher strengths by 31% and 35%, respectively, in relation to granite aggregate based concretes C2 and FRC2.

2.2. Flexural tensile behaviour

The behaviour of HPC concrete C1 and C2 notched beams during the bending test was almost linear-elastic up to the peak load values, then the curve was sloping until the complete separation of the samples into two parts. FRC samples showed a tri-linear variation with significant cracking between first crack load and peak load. Typical experimental load (F) — deflection curves (δ), recorded during bending tests were shown in Fig. 3.

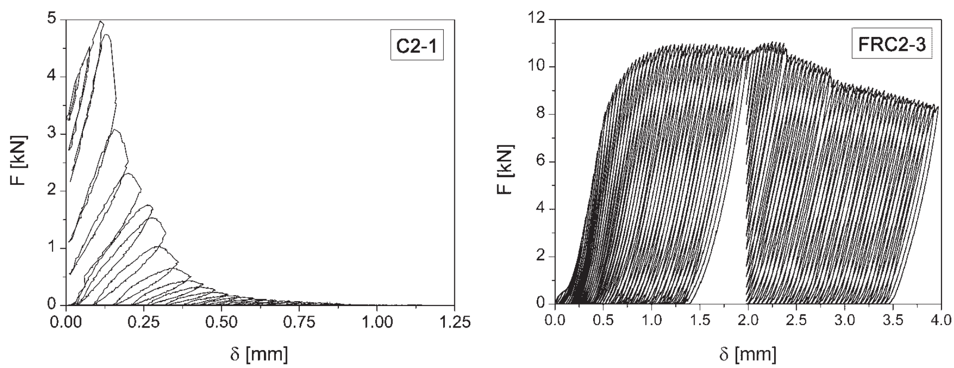

Fig. 3. Typical experimental load-deflection curves of C2 and FRC2 concrete

The examination of FRC beams was carried out in two stages. During first stage, crack mouth opening displacement (CMOD) and deflection were measured until cracking the beams along the whole height. After dismantling the strain gauge, only deflection was recorded.

A typical generic curve of FRC samples is characterized by a linear curve up to the first crack, and then non-linear behaviour up to the peak load. After reaching the peak load, the load carrying capacity declines, however the higher the loss of strength, the lower the steel fiber content. As micro-cracks grow and join into larger macro-cracks, the long hooked-end fibers become engaged in crack bridging.

2.3. Static modulus of elasticity

The cylinder mean values and standard deviation of modulus of elasticity were shown in Fig. 4. The value of the cylinder elasticity modulus was only slightly affected by adding steel fibers and it increased with their percent volume added. At a 0.5% content of steel fibers in FRC2, the modulus value was lower by 6% compared to that of the FRC1.

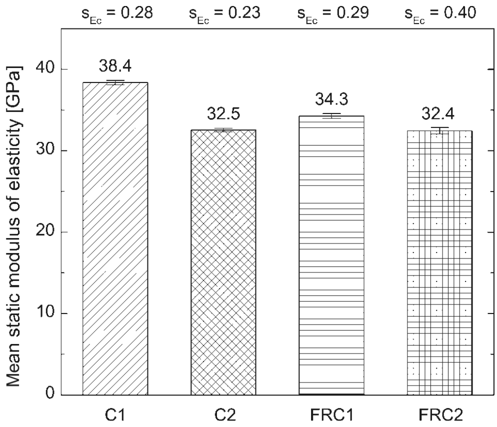

Fig. 4. Cylinder static modulus of elasticity

In the case of concrete made from granite aggregate, the modulus of elasticity of C2 was comparable with that of FRC2, a decrease by 0.3% only was recorded. For concrete with granodiorite aggregate such a drop was much higher and comparing C1 with FRC1 it amounted to 12%. Comparing modulus of concretes with and without fibers made from various aggregates, higher values for concretes with granodiorite aggregates by 18% and 6% respectively, were recorded.

2.4. Fracture properties

To evaluate the crack resistance of concrete samples C1 and C2, the K_{Ic} stress intensity factor was used, which described the stress field near the tip of a crack. This factor was computed using equation [9] as:

$$K_{Ic} = \frac{F_{\max}}{b\sqrt{h}} f(\alpha) \quad [\mathrm{MN/m^{1.5}}] \tag{1}$$

where: $F_{\max}$ — is the applied critical force, b — is the sample width, h — is the sample height (see Fig. 1), $f(\alpha)$ — is the function of geometry.

Equation (1) can be used for single-notched three points bend beam with span/height = l/h = 4. The geometry function $f(\alpha)$ is:

$$f(\alpha) = 6\sqrt{\alpha}\left\{\frac{1.99 - \alpha(1-\alpha)\left(2.15 - 3.93\alpha + 2.7\alpha^2\right)}{(1+2\alpha)(1-\alpha)^{3/2}}\right\} \tag{2}$$

where: α — is the relative crack length, $\alpha = a_0/h$, a_0 — is the notch depth.

The energy release rate GIc is defined as a measure of the energy that must be supplied to the sample in order to destroy it by bending, is computed as follows:

$$G_{Ic} = \frac{K_{Ic}{}^2(1-\nu)}{E_{cm}} \quad \text{[N/mm]} \tag{3}$$

where: ν — is the Poisson's ratio, E_{cm} — is the sample modulus of elasticity.

Based on experimental dependencies of element load-deflection, the unit work of destruction JIc:

$$J_{Ic} = \frac{A}{2b(h-a_0)} \quad \text{[N/m]} \tag{4}$$

where: A — is the energy accumulated in the sample to achieve F_{max}.

Another parameter of crack resistance is the crack tip opening CTOD, which determines the material resistance to cracking. The mean results of strength parameters and fracture energy of standard concrete C1 and C2 were included in Table 4.

Table 4

Mean values of tensile strength parameters and experimental fracture energy of C1 and C2

Mixture designation	F_{max} [kN]	K_{Ic} [MN/m$^{1.5}$]	G_{Ic} [N/mm]	J_{Ic} [N/m]	$CTOD_c$ [μm]
C1	7.5	1.61	0.065	352	0.144
C2	5.1	1.09	0.036	107	0.116

In order to describe the steel-polypropylene fiber reinforced HPC (FRC1, FRC2) post-cracking enhancement, different toughness indexes were proposed. RILEM proposed the concept of equivalent flexural tensile strength — f_{eq}, residual flexural tensile strength — f_R, which is more readily to assess [10, 11]. According to RILEM final recommendation $f_{eq,2}$ or $f_{R,1}$ is used in verification of serviceability limit states, and $f_{eq,3}$ or $f_{R,4}$ is applied in the ultimate limit state analysis [12].

With regard to the experimental curves, the load at the limit of proportionality F_L, the corresponding strength, $f_{fct,L}$, the equivalent flexural strength, $f_{eq,2}$ and $f_{eq,3}$, and the residual flexural strength, $f_{R,1}$ and $f_{R,4}$, relating to a mid-span deflection equal to 0.46 mm and 3 mm were evaluated. The load at the limit of proportionality F_L is equal to the highest value of load recorded up to a deflection of 0.05 mm. The strength corresponding to the limit of proportionality can be computed using the following equation:

$$f_{f_{ct},L} = \frac{3F_L l}{2b\left(h - a_0\right)^2} \quad \text{[MPa]} \tag{5}$$

where (h-a_0) is the distance between the tip of the notch and the top edge of the sample.

The parameters $f_{eq,2}$ and $f_{eq,3}$ were evaluated up to a deflection of $\delta_3 = \delta_L + 0.65$ mm and $\delta_3 = \delta_L + 2.65$ mm, where δ_L is the deflection corresponding to F_L. The portion of the energy required by fracture of concrete corresponding to OBA field in Fig. 5 (A^b_{OAB}), was not considered in assessing the equivalent flexural strength. Only the effect of fibers was considered (A_{ABCD} — $A^f_{BZ,2}$ and A_{ABEF} — $A^f_{BZ,3}$ in Fig. 5).

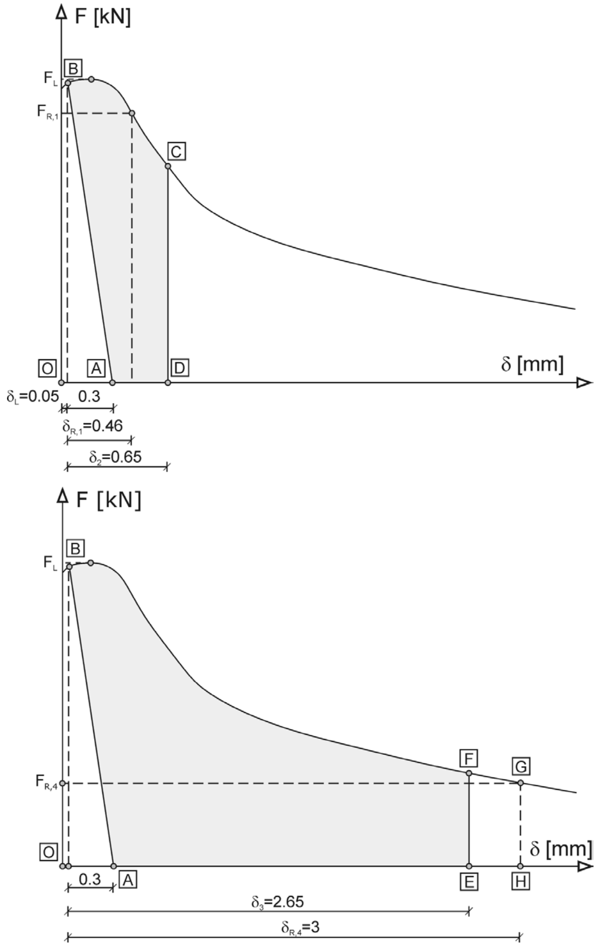

Fig. 5. Evaluation of flexural tensile strength FRC parameters [10-12]

The equivalent flexural strength was calculated from the following expressions:

$$f_{eq,2} = \frac{3l}{2b\left(h - a_0\right)^2}\frac{A_{BZ,2}^{f}}{0.5} \quad [\text{MPa}] \tag{6}$$

$$f_{eq,3} = \frac{3l}{2b\left(h - a_0\right)^2}\frac{A_{BZ,3}^{f}}{2.5} \quad [\text{MPa}] \tag{7}$$

The residual flexural tensile strength $f_{R,1}$ and $f_{R,4}$ at a mid-span deflection of 0.46 mm and 3 mm, respectively, were computed according to:

$$f_{R,1} = \frac{3F_{R,1}l}{2b\left(h - a_0\right)^2} \quad [\text{MPa}] \tag{8}$$

$$f_{R,4} = \frac{3F_{R,4}l}{2b\left(h - a_0\right)^2} \quad [\text{MPa}] \tag{9}$$

Energy dissipation in the fractured concrete is the most advantageous characteristic of FRC due to the addition of fibers to the material. The fracture energy (G_F) was computed as the area under the stress-displacement curves. Assuming a linear stress distribution in relation to the fracture depth, the tensile stress was calculated according to the following formula:

$$\sigma = \frac{3Fl}{2b\left(h - a_0\right)^2} \quad [\text{MPa}] \tag{10}$$

where F is the load recorded during the three point bending test on specimens with dimensions of 100 × 100 × 500 mm (see Table 5).

The fracture energy was computed until a predetermined load-deflection point based on the following formula:

$$G_F = \int_{\delta=0}^{\delta=\delta_{\lim}} \sigma d\delta \quad [\text{N/mm}] \tag{11}$$

The fracture dissipated up to a deflection of 3 mm seems to be interesting from the designing viewpoint, and such a deflection value was adopted while computing energy in this work. The results of strength parameters and fracture energy were included in Table 5.

Table 5

Mean values of tensile strength parameters and experimental fracture energy of FRC1 and FRC2

Mixture designation	F_L [kN]	F [kN]	$f_{fct,L}$ [MPa]	$f_{eq,2}$ [MPa]	$f_{eq,3}$ [MPa]	$f_{R,1}$ [MPa]	$f_{R,4}$ [MPa]	G_F [N/mm]
FRC1	4.0	8.7	5.3	8.0	9.9	8.8	10.1	34.257
FRC2	2.1	8.2	2.8	8.0	12.7	8.4	13.0	32.287

The mean results and standard deviation of fracture energy of concrete without fibers (C) and with ST-PP fibers (FRC) were shown in Fig. 6.

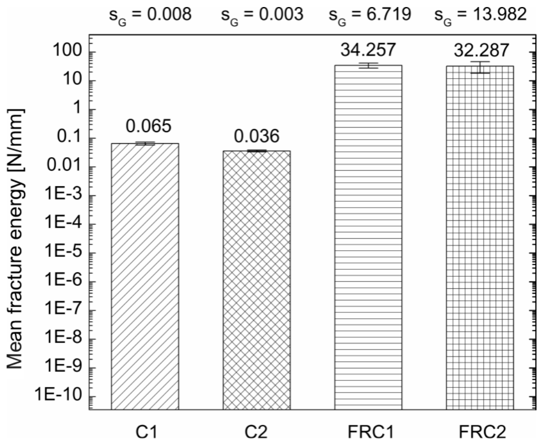

Fig. 6. Fracture energy

In the above mentioned figure, the results were shown in a logarithmic scale. Studies have shown that the most fracture resistant concrete was FRC2 which had 0.5% of ST fibers and 0.5% of PP fibers and it was made from granite aggregate. C1 concrete without dispersed reinforcement based on granodiorite aggregate was characterized by energy 80% higher than that of the C2 concrete with granite aggregate. In the case of concretes with fibers however, it was observed that the FRC1 had the energy by 6% higher than that of FRC2.

In the case of C samples, a brittle fracture occurred through separation of elements into two parts. The FRC samples with cracks underwent significant deflection, however not in all cases the brittle destruction of samples by breaking them into two parts occurred, since some bridges were formed by the fibers on the crack surface and limited the split.

Summary

The presence of steel/polypropylene fibers had an insignificant effect on the compressive strength of high performance concrete. However, the presence of fibers had a significant impact on the splitting tensile strength of HPC. The volume of fibers was also a decisive factor regarding the modulus value of high performance concretes. The above mentioned characteristics of C and FRC concretes were to a great extend affected by the type of coarse aggregate used. In the case of granodiorite aggregate of higher crushing strength, all mechanical parameters of concrete were better than those of the corresponding aggregate concretes made from granite aggregate. On the other hand, when comparing C and FRC concretes made from the same type of aggregate, it was found that adhesion forces between granite aggregate and fibers were much stronger.

All samples of FRC showed a typical three-line variation in load-deflection curves at flexure. The curves mostly consisted of a linear branch up to the first crack, followed by non-linear behaviour up to the peak and extensive descending/strain hardening region. The FRC samples which contained fibers of low elasticity modulus showed an abrupt decrease in the load capacity immediately after the peak load, and depended on the percent volume of polypropylene fibers. A critical evaluation of the load-deflection curves of C and FRC proves that the fibers contribute significantly to preserve the structural integrity and stability of high performance concrete.

The GF values for the high modulus fibers should be numerically higher than the corresponding values for the low modulus fibers. However, the fracture energy is not a measure of the efficiency and effectiveness of the fibers in inhibiting the cracks. Adding steel/polypropylene fibers to concrete and the type of coarse aggregate has a significant effect on splitting tensile strength, modulus of elasticity, flexural strength, fracture behaviour, fracture energy and ductility. The factors which influence these properties of FRC are modulus of elasticity and geometry, the content and properties of fibers as well as the crushing strength of coarse aggregate and aggregate adhesion to the cement paste and fibers.

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P. SMARZEWSKI, D. BARNAT-HUNEK

Wpływ włókien stalowych-polipropylenowych na parametry pękania betonu wysokowartościowego

Streszczenie. Celem artykułu jest ustalenie wpływu włókien na parametry pękania betonu wysokowartościowego (HPC). W badaniu zastosowano dwa rodzaje kruszywa grubego: granit i granodioryt frakcji 2/8 mm. Badania doświadczalne wykonano na próbkach sześciennych, walcach i belkach z nacięciem. W betonie typu FRC dodano odpowiednio 0,5 i 0,75% włókien stalowych oraz 0,5 and 0,25% włókien polipropylenowych. Ustalono średnie wytrzymałości i odchylenia standardowe przy ściskaniu, rozciąganiu przez rozłupywanie, średnie statyczne moduły sprężystości i średnią energię pękania. Badania doświadczalne I modelu pękania pokazały, że HPC bez włókien (C) charakteryzował się kruchością, a fibrobeton (FRC) był bardziej plastyczny. Włókna mostkowały rysy podczas obciążenia, opóźniały powstawanie rys włoskowatych i zapobiegały łamaniu się naciętych belek. Kształt opadającej krzywej obciążenie–ugięcie zależał od geometrii i mechanicznych właściwości oraz ilości zastosowanych włókien, a w przypadku BWW bez włókien od rodzaju kruszywa grubego. W przypadku kruszywa granodiorytowego obserwowano lepsze parametry mechaniczne betonu.

Słowa kluczowe: beton wysokowartościowy, włókna stalowe, włókna polipropylenowe, kruszywo grube, parametry pękania
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