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To cite this article: J Prokeš et al 2021 IOP Conf. Ser.: Mater. Sci. Eng. 1039 012008

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Effects of elevated temperature on the behaviour of concrete beams reinforced with fiber reinforced polymers

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Abstract. Fiber reinforced polymer (FRP) rebars have increasing popularity in the construction industry all around the world although steel rebars are widely used for reinforcing of the concrete so far. FRP bars, which have higher tensile strength compared to steel rebars with the same nominal diameter under normal conditions, are composed of resin matrix and fibers. In this paper, the load-bearing capacity of FRP reinforced concrete after elevated temperature exposition are present. The results are compared with concrete sample with steel reinforcement. Commercially produced glass FRP (GFRP) and carbon FRP (CFRP) rebars with sand coatings surface treatments were implemented in concrete beam and subjected to four-point bending load. The residual flexural strength of reinforced concrete after heating to 1000°C was obtained and evaluated and compared to results of non-heated elements. The results indicate that fire resistance of FRP reinforced beams can reach at least 60 minutes according the standard ČSN EN 13501-2.

1. Introduction

The present building industry is increasingly influenced by the requirements for low energy performance and long-life of buildings. Concrete structures are often operated in areas with high chemical load. Otherwise, a lot of waste products is used as cement component, leading to a decrease in the pH of the concrete. Therefore, increasing attention is paid to the fibre-reinforced polymers (FRP) as a corrosion resistant alternative to the steel rebar.

Great attention is paid to fire and temperature resistance of FRP reinforcement. A relatively large number of authors dealing with this issue indicate that this is a key feature of these materials. A behavior of FRP rebars and concrete bond at elevated temperature are very well presented in paper [1]. The authors carried out pullout tests, and obtained the bond-slip curve at the temperature ranging from 20–350°C. They propose model based on the glass transition temperature $T_g$ and the decomposition temperature of matrix. The bond-slip constitutive relationship model at elevate temperature can be used for the analysis of fire performance of FRP reinforced concrete members at early stage of the fire, but the analysis of other stage of the fire and post-fire are not the subject of this paper.

Similar study of the effect of elevated temperatures on the mechanical properties of FRP bars and the bond behavior between FRP bars and concrete was carried out [2]. Four types of reinforcement bars: basalt FRP (BFRP), CFRP, GFRP, and steel bars of 10 mm diameter were used. The results showed that the FRP bars suffered significant reductions in their mechanical properties upon exposure to high temperatures of up to 450°C. At a critical temperature of 325°C, the FRP bars lost as high as
(55% and 30%) of their tensile strength and elastic modulus, respectively. The percentage reduction in mechanical properties and bond strength was more pronounced in specimens with FRP bars than those with steel bars under elevated temperatures; the percentage reductions in bond strength between concrete and FRP bars reached as high as 81.5% after exposure to 325°C.

The paper [3] study residual tensile strength, modulus of elasticity, and bond strength to concrete of GFRP bars after exposure to elevated temperatures of up to 400°C and subsequent cooling to an ambient temperature. The results showed that the residual strength generally decreases with increasing temperature exposure. However, as much as 83% of the original tensile strength and 27% of the original bond strength was retained after the specimens were heated to 400°C and then cooled to ambient temperature.

The authors of the study [4], took a next step and perform subsequent axial tensile tests and pull-out tests of these materials after exposing them into elevated temperature effects in the range of 23–800°C. Severe effects on the tensile properties of bare steel bars were observed after 600°C, while this critical limit is 300°C for bare GFRP bars. Test results show that bond strength degradation is almost linear for both type of rebars, however, 600°C is also the critical temperature regarding the serious deterioration on concrete.

These papers [1–4] and many others are a very good starting point for our study, when several types of reinforcement are casted in concrete and after heating the element to 1000°C is subsequently tested.

2. Experimental analysis
An experiment to determine the effect of elevated temperatures on the residual bearing capacity of concrete beams reinforced by FRP or steel reinforcement was carried out. The experiment was performed on concrete elements of size 100 × 150 × 1800 mm reinforced by GFRP, CFRP or steel rebars.

2.1. Materials
All concrete beams were reinforced in flexure with 1 longitudinal bar at the bottom, four types of main reinforcement were used: GFRP rebars 10 and 14 mm with sand coated and braided surface (beams G10 and G14), CFRP rebars 9 mm sand coated and braided (beams C9) and steel rebars 12 mm of class 10505 (beam S12). For longitudinal reinforcing bars in a beam, the concrete cover was 25 mm. GFRP and CFRP rebars are standard product of company PREFA KOMPOZITY. The same epoxy resin is used for the production of these rebars. Glass transition temperature T_g of the resin is 120°C.

Stirrups made of steel rebars with a diameter of 6 mm were bent into the box-shape and were placed symmetrically at a distance of 150 mm from mid-span and then another 4 stirrups were placed with spacing 80 mm. Stirrups were located symmetrically about the center of the beam, a total of 10 stirrups per beam. Assembling reinforcement (steel φ6 mm) was placed at the top of the beam. Anchoring fixtures were placed into the anchoring area on the longitudinal FRP reinforcement [5].

The samples were made of concrete of strength class C 30/37. The mean compressive strength of the concrete and the modulus of elasticity were 40.95 MPa and 27.9 GPa, respectively.

Six beams were produced for each beam type - three for bending test after exposure to elevated temperatures and three as reference for bending tests of unheated elements. A total of 24 reinforced beams were prepared for the tests.

2.2. Fire test
The samples were exposed to high temperatures in a fire furnace to study the high temperature behaviour of materials. The beams were placed on top of the furnace to provide heating from one (bottom) side. The temperature increase in the furnace up to 1000°C was controlled in accordance with ISO 834 [6], the maximum temperature on the samples was maintained for 2 hours.
During the fire experiment, temperatures were measured using a type-K thermocouple fixed to the reinforcement in the heated mid-span and in the unheated anchorage zone of members.

2.3. Mechanical loading
The beams cooled to ambient temperature were subjected to four-point bending test. The samples were placed on a cylindrical support at a distance of 1000 mm. Beams were loaded by two forces at a distance of 300 mm. During loading, vertical deformations of the structure were measured by means of potentiometric displacement sensors (POT1-5). The deflection was measured in five sections along the beam - in the mid-span, under the loading forces and at a distance of 200 mm from the supports. The test geometry is shown in figure 1. The force F acting as two loads was measured and recorded in the test.

Figure 1. Four-point bending test of reinforced concrete beams.

3. Results and discussion
The reinforced concrete samples were exposed to high temperatures in the furnace. Temperatures in furnace and at reinforcement can be seen in figure 2. The highest measured temperature at the end of fire test at reinforcement was 570 and 140°C in mid-span and in the anchorage zone, respectively. The time-temperature curves show flat area (slightly above 100°C) between 15–45 minutes. It is caused by evaporation of residual moisture. The temperature 325°C often mentioned in the literature as a critical temperature [1, 2, 7] is achieved in mid-span in most cases between 60 and 90 minutes. Although some degree of irreversible polymer degradation has certainly occurred, the decomposition temperature was not reached. The temperature of reinforcement in anchorage zone (in the ends of concrete beam) reaches temperature Tg = 120°C in approximately 150–180 minutes.

The samples cooled to ambient temperature were subjected to four-point bending test. The mechanical tests records of the selected beam are shown in figure 3 in the form of a load displacement diagram. The deflection measured at mid-span is considered to be a displacement. For all samples, exposed to temperature 1000°C is characteristic that the slope of the load-displacement dependence is smaller, compared to unexposed samples. Also, the samples reinforced with steel show a change in slope, which indicates reduction in steel-concrete bond. It can be caused by irreversible temperature change in concrete. The change of slope in load - displacement diagram of CFRP and GFRP reinforced samples is greater than that of steel and also the maximum load bearing capacity is lower. It should be noted that load - displacement diagram of C9_T20 and G14_T20 respectively C9_T1000 and G14_T1000 are very similar. The usage of GFRP can be comparable to users, but more cost-effective option.
Figure 2. Temperatures measured in the furnace and at the reinforcement during fire test.

Figure 3. Load displacement diagrams in 4-point bending test of concrete beams reinforced by steel, GFRP or CFRP rebars. The symbol “_T20” indicates reference sample which has not been exposed to high temperature. The symbol “_T1000” indicates sample exposed to temperature 1000°C.
The resulting maximum bending capacity determined from the mean of three beams of each group are given in table 1. Also comparison of load-bearing capacity of reinforced beams are shown in table 1 for members exposed and not exposed to high temperatures.

**Table 1. Flexural bearing capacity of reinforced concrete beams.**

| Beam | T (°C) | F_{max,T} (kN) | F_{max,T} / F_{max,20} (%) |
|------|--------|----------------|---------------------------|
| S12  | 20     | 44.4 ± 0.9     | 100.0                     |
| S12  | 1000   | 40.0 ± 1.4     | 90.2                      |
| C9   | 20     | 42.5 ± 3.0     | 100.0                     |
| C9   | 1000   | 27.9 ± 2.0     | 65.6                      |
| G10  | 20     | 40.5 ± 2.8     | 100.0                     |
| G10  | 1000   | 12.6 ± 0.8     | 31.1                      |
| G14  | 20     | 41.0 ± 10.7    | 100.0                     |
| G14  | 1000   | 18.7 ± 0.7     | 45.7                      |

The residual load bearing capacity of steel reinforced concrete beam is about 90% of initial value. The residual load bearing capacity of CFRP reinforced beam is about 66% of initial value, otherwise the beam reinforced with diameter 10 mm GFRP has the residual strength reduce to 31% of initial value.

The resin used for CFRP and GFRP rebars is the same. It can be assumed that the difference in relative residual strength of GFRP a CFRP is caused by the nature of glass fiber. Glass is generally considered as supercool liquid unlike to carbon, which is considered as thermally stable material.

**4. Conclusion**
Initial and residual load bearing capacity of concrete beams reinforced with four different reinforcements was determined. As a reinforcing material were used steel rebar of diameter 12 mm, carbon fiber reinforced polymer rebar of diameter 9 mm, and glass reinforced polymer rebar of diameter 10 and 14 mm. Concrete cover was 25 mm. Beams were exposed to temperature 1000°C for 2 hours. After cooling the beams:

- The steel reinforced concrete beam reached 90% of initial load bearing capacity.
- The carbon reinforced concrete beam reached 66% of initial load bearing capacity.
- The glass rebar of diameter 10 mm reinforced concrete beam reached 31% of initial load bearing capacity.
- The glass rebar of diameter 14 mm reinforced concrete beam reached 46% of initial load bearing capacity.

The slopes of load-displacement diagram are reduced in all cases (included the steel). This indicates that reduction in rebar to concrete bond occurred. It can be caused by irreversible temperature change in concrete.

The temperature measured on the reinforcement in the mid-span (zone exposed to 1000°C) reach critical temperature 325°C in 60 minutes or later. The temperature measured in anchorage zone (“cold end of the beam”) reach 120°C (Tg of resin) in 150 minute or later. The residual load bearing capacity is 31–66%. This indicates that fire resistance of such beam can reach at least 60 minutes according the standard [8]. FRP reinforcement can be considered as safe fire-proof solution, although diameter and material of FRP should be taken into account.

The increasing Tg of resin to 150°C or more can rise up the fire resistance to theoretical limit 180 minutes.
Future research will be focused on the elevated temperatures effect on the residual behaviour of concrete beams reinforced with both GFRP rebars and stirrups.

Acknowledgments
This research was supported by the Technology Agency of the Czech Republic, grant No. TH04020431 (Extension of the application area of FRP reinforcement in concrete structures).

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