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Fracture parameters of concrete after exposure to high temperatures: pilot tests

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Abstract. Experimental concrete panels were after an appropriate aging time loaded by high temperatures (550, 600, 800 and 1000 °C) in a furnace intended for fire tests of building materials. These panels were heated according to the standard temperature-time curve according to EN 1991-1-2. One of panels was a reference without temperature load. Test specimens were obtained as cores drilled out from panels after performing fire tests. The cylindrical specimens were provided with a central chevron type notch and subsequently tested in three-point bending fracture test. The load versus displacement (deflection in the middle of span) diagrams were recorded during testing and basic fracture parameters were subsequently evaluated.

1 Introduction

Concrete belongs to the plentifully used building materials for a wide range of applications. Structures and construction components utilizing this composite were commonly designed assuming normal service temperatures. However, high temperatures acting on concrete – for example in connection with certain technological procedures, fire etc. – cause wide range of physical and chemical processes, which result in changes in the microstructure of composites and thus affects the mechanical properties of concrete [1].

The currently valid standard for the assessment of existing structures ISO 13822:2010 [2] states that assessing of the real condition of building structures have to reflect the actual properties of the materials, material parameters characterized in the initial proposal of construction, in standards or in regulations. It is necessary considered the degradation and the potential influence of the load during the structure life to determine real material parameters. The standard highlights the need to consider extraordinary loads, among which also belongs the fire. However, currently there are no guidelines for effective determination of the material characteristics needed for the evaluation of the static state of structures damaged by fire.

This paper is focused to determination basic fracture parameters via evaluation of pilot fracture tests of cylindrical specimens which were obtained as cores drilled out from the concrete panels after their exposure to fire.

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2 Specimens

The normal concrete with ordinary material characteristics in terms of fire resistance was used for the production of experimental panels. It was a C30/37 strength class concrete (selected as widely used for building structures), exposure class XC3 and quartz aggregate (16 mm maximal size) was used. In total, the seven concrete panels were made of nominal dimensions 2300 × 1300 × 150 mm intended for subsequent temperature loading.

Experimental panels (P1, P2, P3, and P7) were after an appropriate aging time loaded by high temperatures (maximal values were 550, 600, 800, and 1000 °C) in a furnace intended for fire tests of building materials, see Fig. 1; the furnace equipment belongs to the AdMaS (Advanced Materials, Structures and Technologies) science centre, part of the Faculty of Civil Engineering at Brno University of Technology [3]. Panels were heated according to the standard temperature-time curve according to EN 1991-1-2 [4].

Fig. 1. Concrete panel P7 covered furnace of AdMaS science centre [3] and loaded by high temperature – maximal value was approximately 1000 °C in this case.

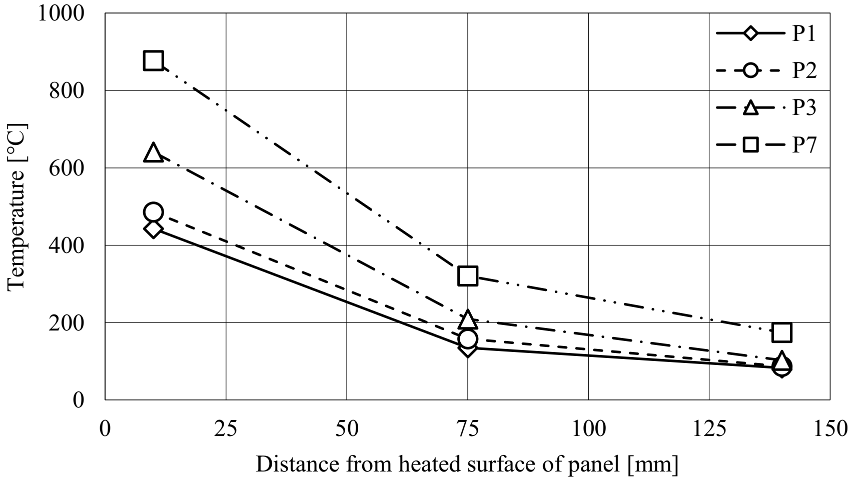

Fig. 2. Measured temperature versus distance from heated surface of panels P1, P2, P3, and P7.

Test specimens were obtained as cores drilled out from concrete panels after performing fire tests. Nominal diameter of these cylinders was 75 mm. Three specimens from each panel were used for subsequent fracture tests. Temperature sensors were placed in three locations in longitudinal axis of panels (as can be seen in Fig. 1) and in each of these places in three (or four) locations along the thickness of the panel. Temperature introduced in Fig. 2 represents mean value determined from sensor in middle of panel and from the sensor placed nearest the place from which the specimens were drilled.

One of panels was a reference without temperature load (P4). Five specimens from this panel were used for fracture tests.

3 Fracture tests

The cylindrical specimens (drilled cores) were provided with an initial stress concentrator in the form of chevron notch and subsequently tested in three-point bending fracture test, see Fig. 3 – scheme and selected realization of the test.

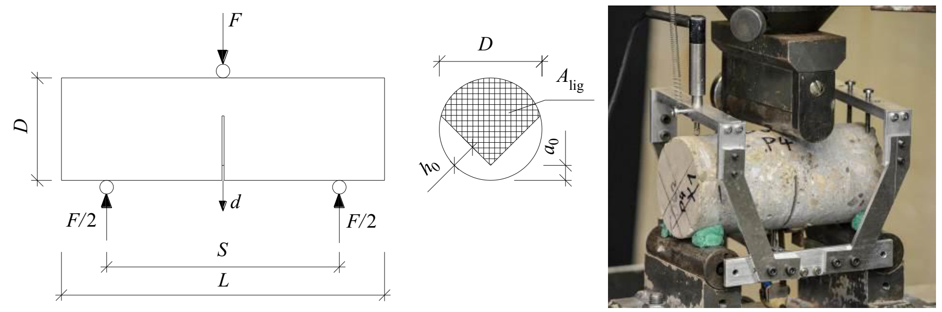

Fig. 3. Scheme of three-point bending fracture test of specimen with chevron notch (left) and illustration of realization of the test.

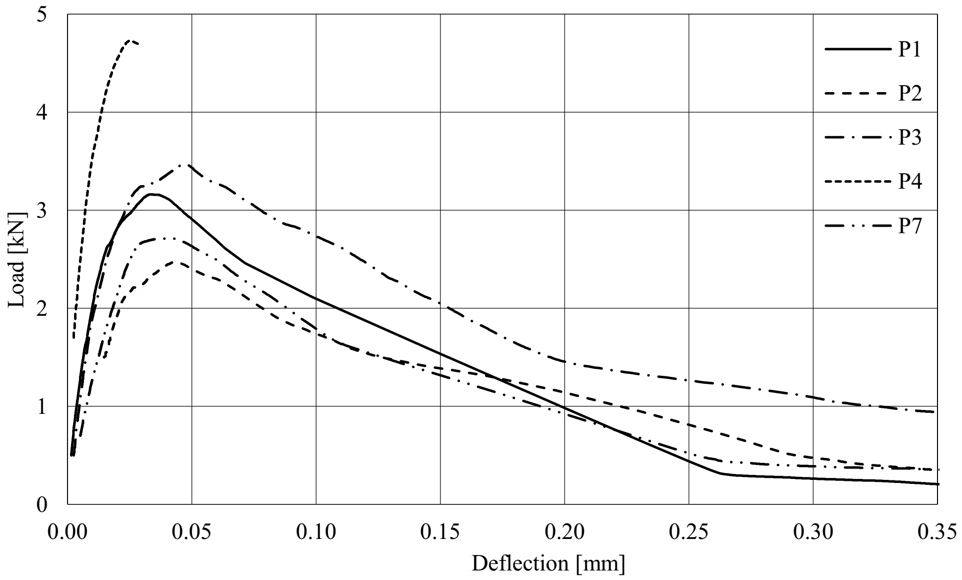

Fig. 4. Selected load–deflection diagrams.

Specimens were loaded under displacement control, therefore the load versus displacement (deflection in the middle of span) diagrams (*F*–*d*) were recorded during testing from which basic fracture parameters [5, 6] were subsequently evaluated. Selected *F*–*d* diagrams are depicted in Fig. 4. Nominal thickness of panels – and thus length of the specimens – was 150 mm, for that reason span length was only 127 mm. The fracture tests were carried out using a Heckert FPZ 100/1 testing machine with measuring range of 0−10 kN. All specimens after fracture tests are captured on photo in Fig. 5.

Fig. 5. All tested specimens (and selected detail) after fracture tests.

4 Methods

The fracture parameters of concrete were calculated from corrected *F*–*d* diagrams. Maximal load F_{max} was used for fracture toughness assessment with help of geometrical function A_{min} [7] and span S:

$$K_{Ic} = A_{min} \cdot \frac{F_{max}}{D^{1.5}}, \text{ where } A_{min} = \frac{S}{D} \cdot \left[1.835 + 7.15 \cdot \frac{a_0}{D} + 9.85 \cdot \left(\frac{a_0}{D} \right)^2 \right], \quad (1)$$

where D is specimen diameter and a_0 is notch tip depth, see Fig. 3.

The work of fracture value W_F, and then the specific fracture energy G_F value, was assessed from the whole *F*–*d* diagram [8]:

$$W_F = \int F(d)\,\mathrm{d}d \quad , \quad (2)$$

$$G_F = \frac{W_F}{A_{lig}} \quad , \quad (3)$$

where A_{lig} is area of ligament, see Fig. 3.

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5 Results

Basic fracture characteristics evaluated using Eq. (1) and (3) – fracture toughness and specific fracture energy – are summarized in Figs. 6 and 7 in form of mean values and standard deviations.

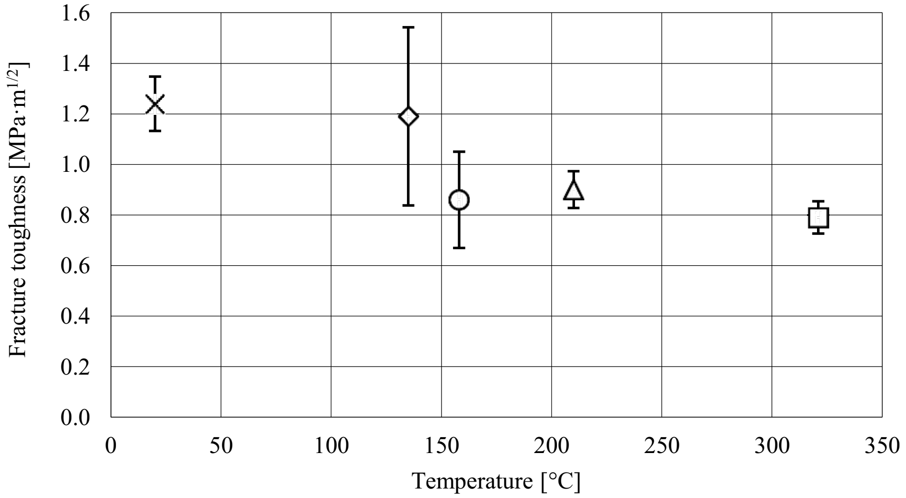

Fig. 6. Fracture toughness versus temperature: mean values and standard deviations.

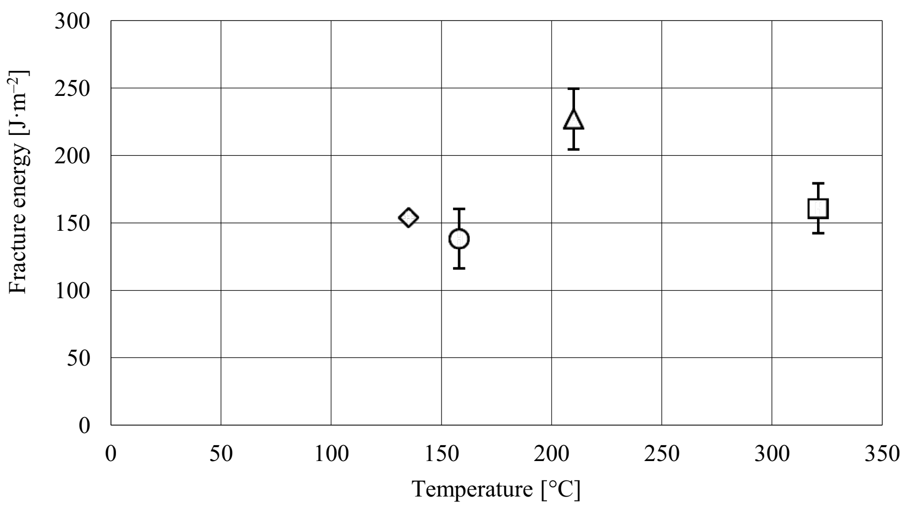

Fig. 7. Fracture energy versus temperature: mean values and standard deviations.

With increased temperature, a gradual decrease in fracture toughness values can be seen. Compared to the reference concrete specimen, the maximum decrease in fracture toughness was about 40%. The results showed variability typical for concrete: the coefficient of variation being below 10%, with the exemption of the P1 and P2 panel specimens (the maximum temperature was 550 °C/600 °C on the heated surface and about 135 °C/158 °C in the middle of the panel/specimen), where the result variability reached 20–30%.

Fracture energy results proved to be complicated to evaluate, seeing as they fluctuated significantly with temperature increase. The *F–d* diagrams in post peak part were often disrupted by sections where stability of loading was lost. Most importantly, this was the case for the reference specimens, where no values could be evaluated whatsoever (see Fig. 4).

6 Conclusions

As results in this paper showed, there are limitations to highly heated specimen fracture parameter assessment. The results were probably most influenced by the following factors: (i) the specimen diameter was too small in relation to the maximal aggregate size of used concrete, (ii) the specimen length was not sufficient for standard three-point bending fracture test evaluation, and (iii) the specimen relatively small size did not allow comfortable measurement device connection. Based on these pilot findings, fracture parameter assessment is planned to be conducted on beam specimens cut out of the panels.

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References

1. I. Hager, Bull Pol Acad Sci-Te **61**, 1 (2013)
2. ISO 13822:2010, *Bases for design of structures – Assessment of existing structures*, (ISO, Switzerland, 2010)
3. http://www.admas.eu/en/, March 10, 2017
4. EN 1991-1-2:2002/AC:2009, *Eurocode 1: Actions on structures – Part 1-2: General actions – Actions on structures exposed to fire*, (CEN, Brussels, 2002)
5. B. L. Karihaloo, *Fracture Mechanics and Structural Concrete*, (Longman Scientific & Technical, New York, 1995)
6. V. Veselý, P. Frantík, Adv Eng Softw **72,** 66–76 (2014)
7. F. Ouchterlony, Int J Rock Mech Min **25,** 71–96 (1988)
8. RILEM TC-50 FMC Recommendation, Mater Struct **18,** 285–290 (1985)