PREMATURE DESTRUCTION OF TWO-SPAN RC BEAMS EXPOSED TO HIGH TEMPERATURE CAUSED BY A REDISTRIBUTION OF SHEAR FORCES

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Abstract. When multi-span RC elements are exposed to fire one usually observes a yielding of span cross-sections while a safety reserve of support cross-sections is still significant. Due to this phenomenon a redistribution of bending moments occurs and the values of sagging moment in span cross-sections decrease while the values of hogging moment in support cross-sections increase. This paper shows the results of tests conducted on two-span RC beams in a situation when only one span has been exposed to high temperature from the bottom. The beams were 12×16 cm in their cross-section. The length of the span was 165 cm. The load has been applied by two forces put on each span. The beams were made of C25/30 concrete with siliceous aggregate. As a result of significant stiffness decrease of the heated span, redistribution of shear forces and bending moment occurs. Due to this redistribution the tested beams were prematurely damaged due to exhaust of the shear load bearing capacity in the middle part of the beam span where there was no transverse reinforcement.

Keywords: two-span beams, reinforced concrete, fire, high temperature, redistribution, shear forces, bending moments, shear resistance, stiffness.

Introduction

Exposed to fire, reinforced concrete (RC) elements exhibit a significant reduction in load capacity and stiffness. This is caused by a decrease in the mechanical properties of concrete and reinforcing steel due to high temperature (EN 1992-1-2 2004; fib Bulletin 38 2007; fib Bulletin 46 2008; Colina *et al.* 2004; Abramowicz, Kowalski 2005; Bednarek, Kamocka 2006; Hertz 2004; Kodur 2014). During fire the slabs or beams are usually heated only from the bottom face. In this case only the tensile zone of the element (reinforcement) in the span region and the compressed zone in the support are heated. Stiffness of a cross-section with heated tensile zone decreases much faster than in a cross-section with heated compressed zone (EN 1992-1-2 2004; Kowalski 2008; Kowalski *et al.* 2013, 2015).

In multi-span elements this phenomenon leads to redistribution of internal forces resulting in a reduction of sagging moments and an increase of hogging moments (EN 1992-1-2 2004; fib Bulletin 46 2008; Kordina 2010; Kowalski, Urbański 2013; Scott, Whittle 2005; Bednarek 2003; El-Fitiany, Youssef 2014; Dwaikat, Kodur 2008).

Simultaneously, a redistribution of shear forces occurs, increasing the risk of shear load bearing capacity loss (Arslan 2012; Xiang *et al.* 2013).

Additionally, during the first heating a concrete thermal transient creep usually occurs simultaneously with a decrease in the cross-section stiffness and free concrete thermal elongation (Khoury 2006; Mindeguia *et al.* 2013; Sadaoui, Khennane 2012). As a result of the transient creep concrete might shorten or elongate, depending on the concrete compressive stress (Gernay, Franssen 2010).

The main goal of this paper is to present an experimental study of the redistribution of shear forces and bending moments in two-span loaded RC beams, in case when, only one span is exposed to high temperature from the bottom face. In the tested beams redistribution of shear forces was observed, leading to premature beam destruction due to shear. The experimental results were then compared to simplified calculations based on Eurocode (EN 1992-1-2 2004). Temperature distribution in the cross-section of beams was determined based on measured values supplemented by Finite Element Method (FEM) calculations.

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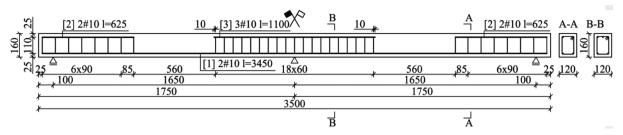E-mail: *m.glowacki@il.pw.edu.pl*

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Fig. 1. Tested beams

1. Experiment description

1.1. Elements

Two two-span RC beams with rectangular cross-section b×h = 120×160 mm, span of 1650 mm and total length of 3500 mm (Fig. 1) were tested.

Concrete with siliceous gravel aggregate was used (for detailed composition of the concrete mixture see Table 1). The average concrete compressive strength measured 28 days after casting on a 150×150×150 mm cube was 34.5 MPa. This classified concrete as C25/30. The high temperature experiments were conducted 5 months after beams casting when concrete average compressive strength reached 43.8 MPa and concrete moisture content gained 4.2%.

According to EN 1992-1-2 (2004), in case when moisture content in concrete exceeds 3%, concrete thermal spalling may occur. However, in this experiment this phenomenon was not observed, despite the higher moisture content.

Table 1. Concrete mixture (for 1 m³ of concrete)

No.	Component	kg/m³
1	Water	97
2	Cement CEM II B-V 32.5R	314
3	Sand 0–2 mm	768
4	Siliceous gravel 2–8 mm	548
5	Siliceous gravel 8–16 mm	664
6	Fly ash	62
7	Chemical additives	1
	Total	2454

The main beam reinforcement was made of B500SP steel bars produced in Poland with guaranteed yield strength of 500 MPa. The average yield strength was experimentally determined to be 590 MPa. The span reinforcement was made of two 10 mm bars (reinforcement ratio $\rho_l = 0.97\%$) while in the intermediate support three bars of 10 mm diameter ($\rho_l = 1.45\%$) were used. Stirrups made of St0S-b 6-mm-diameter steel were placed at spacing of 90 mm at end supports and 60 mm at the intermediate support. Concrete cover was made in a way that the main reinforcement axis distance was 25 mm from the element's face.

1.2. Testing procedure

The beams were loaded with two forces placed on each span at 1/3 of their length (Fig. 2). Before high temperature tests each beam was loaded and unloaded eight times to 60% of its ultimate load capacity. This value was calculated on the basis of real material properties. Concrete compressive strength was taken as 0.8 × 43.8 = 35.0 MPa. 80% of cube strength (at the time of the test) might be applied as a cylindrical (real) strength. Steel yield strength was taken as 590 MPa (average value of the test results).

Pre-loading elements before high temperature tests was done to simulate real structural conditions. The 60% load level might be considered appropriate as quasi-permanent load level in many practical cases. Based on previous research (Kowalski 2008), loading and unloading the specimen 8 times is enough because between 6th and 8th load cycle both a crack pattern and deflection of a tested beam usually stabilises. Due to cracks, stiffness of the tested beam is close to the real conditions (cracked cross-sections stiffness). Cracks formed as a result of pre-loading, do not have any influence on reinforcement temperature during heating as long as cracks widths are small. However, big cracks (up to 2–3 mm width) begin to form only when reinforcement temperature is high and the beam is close to load capacity loss.
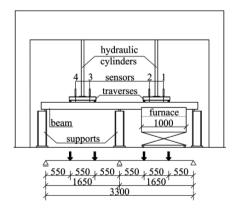

Fig. 2. Diagram of the experimental setup

According to calculations performed on the basis of real material properties, the beam loaded with the load equal to calculated load capacity (bending) has relatively big shear capacity reserve:

– 40% at the end support region (with stirrups; Fig. 1);

– 20% at the intermediate support region (with stirrups);

– 70% in the middle part of the span (no stirrups).

After 8 load-unload cycles, the beam was loaded with 60% of its calculated load capacity (a quasi-permanent load level in many practical cases). A furnace heated previously to the temperature of 350 °C was slid under the bottom face of the beam (see Figs 2 and 3). The furnace temperature increased to 850 °C in the subsequent 40 minutes (Fig. 4).
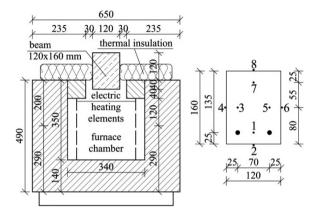

Fig. 3. Location of the beam in the furnace and thermocouples placement in the beam cross-section

During the tests the following parameters were measured:

– time of heating;

– beam deflection, at four points (see Fig. 2);

– support reactions;

– temperature in mid-span cross-section of the beam, Figure 3 shows thermocouples placement.

2. Discussion of experiment run

The tests were conducted until the beams were destroyed which happened after 56 and 65 min of heating. Figure 4 shows variation of temperature in the furnace chamber and at particular cross-section points of tested beams. It should be noted that the thermal conditions inside the furnace chamber were comparable to those described by the standard fire curve (EN 1991-1-2 2002).

The temperature at the bottom face (T2) did not significantly differ between the two beams; while the lateral surfaces of the beam 1 (T4 and T6) were heated slightly faster than in case of the beam 2. At the time of destruction the temperature in the tensile zone of the beam 1 (T1) exceeded 500 °C but in the beam 2 it was a little bit lower. Beam 1 was destroyed about 10 minutes earlier than beam 2. However, the beams were destroyed due to shear load bearing capacity loss, not as the result of the yielding of the main reinforcement.

Figure 5 shows the deflection of the tested beams, measured at two points of the heated span (sensors 1 and 2, Fig. 2) and at two points of the not heated span (sensors 3 and 4). The heated span deflected significantly while the unheated span deflected slightly upwards. The deflection growth was approximately linear.

Figure 6 shows the relative changes of support reactions. It should be noted that in the applied static scheme the reactions at both end supports are independent of the beam stiffness and are always equal to each other.

Figure 7 shows distribution of bending moments and shear forces in the tested beams: before the beginning of heating (blue line), after 20 minutes of heating (red line) and just before the beam destruction (green line).

Due to heating there was a gradual decrease of the sagging moments and an increase of the hogging moments. A similar phenomenon was observed for shear forces. The shear forces at the ending supports decreased while the shear forces in the middle section of the span and at the intermediate support increased.

Figure 8 shows relative changes of extreme sagging and hogging moments during heating. As a result of redistribution of bending moments, caused by the decrease of stiffness of heated beam span, the hogging moment increased by 65–75%, while the sagging moment decreased by 25–30%.
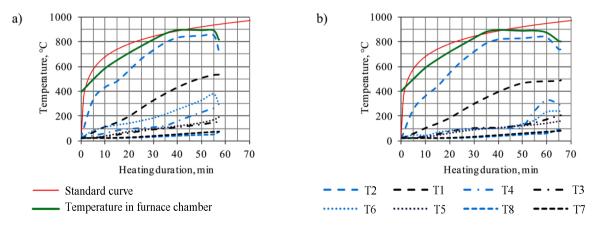

Fig. 4. Variation of temperature in the furnace and at different thermocouples locations (T1–T8, see Fig. 3) with time: a) beam 1, b) beam 2
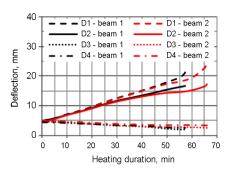
Fig. 5. Deflection of the beams: D1, D2 – at heated span, D3, D4 – at not heated span (sensor**s** location – see Fig. 2)
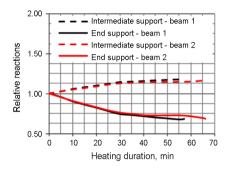

Fig. 6. Relative changes of support reactions: intermediate and end support

Figure 9 shows relative changes of shear forces in the middle section of the span and at the intermediate and end supports. As a result of shear forces redistribution, a significant shear force increase was observed in the middle section of the span (by about 75%). Shear forces at the intermediate support slightly increased (by

about 12–15%) while at the end supports decreased (by about 25–27%).

3. Temperature distribution in the beam cross-section

During the tests the temperature was measured at particular points (see Fig. 3) in a cross-section situated at the middle of the heated span of the beam. The obtained temperature values were used for verification of calculations of unsteady heat flow in the beam cross-section performed with Finite Element Method (FEM). Calculations results provided with an isotherms layout in the cross-section of the beam and the temperature of reinforcing bars.

The thermocouples were put into the beam cross-section after casting. In order to enable a precise placement of the thermocouples as well as their multiple use, they were placed through 5 mm holes drilled in concrete. Then the holes were filled with fine grained dry sand and chopped with rock wool.

When the temperature is measured in the above-mentioned way, it is impossible to directly measure the temperature of bars. If a thermocouple was placed within the beam before casting it would be impossible to guarantee that it would not move. This causes that it is difficult to measure bar temperature directly. When there is even a small gap (fully filled by concrete) between the thermocouple and the bar surface (the thermocouple does not touch the surface of the bar) the reading of reinforcement temperature might be falsified. It might too high if the thermocouple is placed closer to the heated element surface than the bar or it would be too low if the thermocouple was situated in the middle part of the cross-
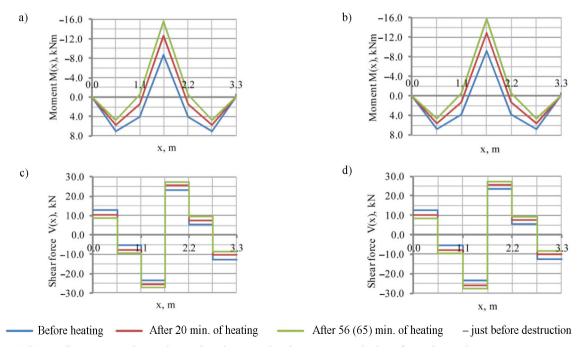

Fig. 7. a), b) Bending moments in two beams in subsequent heating stages, c), d) shear forces in two beams in subsequent heating stages; beam destruction occurred after 56 min (beam 1) and 65 min (beam 2) of heating
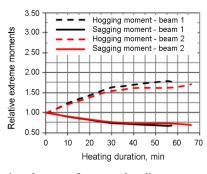
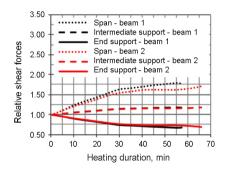Fig. 8. Relative changes of extreme bending moments: hogging and sagging

Fig. 9. Relative changes of shear forces: in the middle section of the span and at supports: intermediate and end

section. Therefore, in these tests the temperature of bars was not measured directly.

In order to perform the unsteady heat flow calculations inside the RC cross-section, concrete thermal conductivity and concrete specific heat must be known. To determine the heat penetrating into the beam cross-section through its surface one should consider both the convection and the radiation heat flux. Basic information on how to model these fluxes can be found in EN 1991-1-2 (2002). However, there is no reason to assume that conditions in electric furnace are similar to fire conditions described in EN 1991-1-2 (2002).

Therefore in the performed calculations the heat flux penetrating through the beam surface was not used as a boundary condition. Instead, the temperature values measured directly at the beam surface (T2, see Figs 3 and 4) were used as an input data for the calculation of unsteady heat flow inside of the beam cross-section.

The calculations were performed with ANSYS software. Concrete thermal conductivity and concrete specific heat were taken from EN 1992-1-2 (2004) for concrete with 3% moisture content. Additionally, real concrete density (2454 kg/m^3) was used.

Two dimensional model of the beam cross-section, made of eight node quadrangle elements with side width of approximately 0.5 cm was considered. The longitudinal reinforcing bars were modelled according to corresponding steel thermal properties (EN 1993-1-2 2005).

It was assumed that only the bottom face of the beam cross-section and the bottom 40 mm of both lateral faces were heated (Fig. 10e).

Figures 10a–10d show isotherms in the cross-section of the beam calculated after 15, 30, 45 and 60 minutes of heating.

Figure 11 shows comparison between the calculated and the measured values of the temperature at the points of the cross-section where the temperature was measured. For the position T3 and T4 the measured values were taken as the average of two thermocouples.

The calculated temperature values are very close to the measured. Thus, it can be concluded that concrete thermal properties taken from EN 1992-1-2 (2004) are good enough for modelling of the temperature distribution in the tested beams.

Additionally, in Figure 11 the temperature values calculated for points where reinforcing bars are situated were plotted as red triangles.

4. Analysis of test results

Before the beginning of heating the bending moment at the intermediate support in both beams was equal to 60% of the calculated cross-section load bearing capacity. However, in the final heating stage the bending moment reached 99% (0.60 × 1.65) of the load bearing capacity in beam 1 and 105% (0.60 × 1.75) in beam 2 (Fig. 8). Thus, in the final stage of beams testing the cross-section at the middle support was very close to the ultimate limit state.

In the heated span cross-section a decrease of load bearing capacity, caused by an increase in reinforcement temperature, occurred simultaneously with a decrease of the bending moment resulting from the redistribution of internal forces. In the final heating stage the bending moment at the span cross-section was estimated at ap-
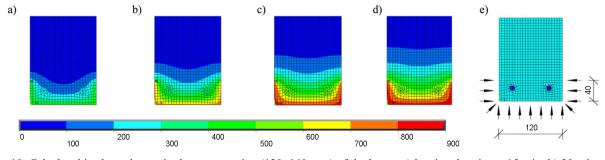

Fig. 10. Calculated isotherm layout in the cross section (120×160 mm) of the beam: a) heating duration – 15 min, b) 30 min, c) 45 min, d) 60 min, e) a part of the beam face exposed to high temperature
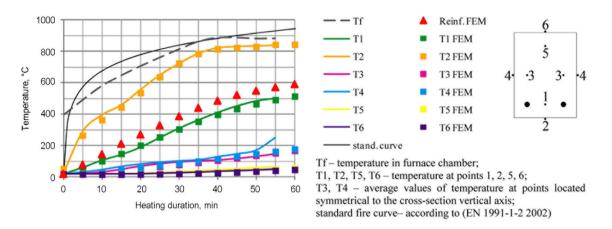
Fig. 11. Temperature in the beam cross section – comparison of experimental results and FEM calculations; lines represent experimental data, markers show FEM calculated results

proximately 42% (0.60 × 0.70) of the load bearing capacity at room temperature in the case of beam 1 and 45% (0.60 × 0.75) in case of beam 2.

This lead to a conclusion that the tensile zone of the span cross-section heated to about 600 °C (see Fig. 11) was also very close to load bearing capacity loss.

However, based on the crack pattern on the beams (Fig. 12), it can be concluded that not the bending moment, but the loss of shear load bearing capacity in the cross-section situated in the middle part of the heated span was a direct cause of the beam destruction.

Table 2 shows a comparison between the shear forces occurring in particular zones of the tested beams and calculated load bearing capacity: before heating (at room temperature) and just before the beam destruction.

Shear load bearing capacity at room temperature in intermediate and ending support zones $V_{Rd,s}$ were calculated with the formula (EN 1992-1-1 2008):

$$V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} \cot\theta, \qquad (1)$$

where: A_{sw} – the cross-sectional area of the shear reinforcement; s – stirrups spacing; z – the lever arm of internal forces; f_{ywd} – the design yield strength of the shear reinforcement; θ – the angle between the concrete compression strut and the beam axis perpendicular to the shear force.

Table 2. Comparison of calculated shear load bearing capacity and shear forces occurred in tested beams

	Intermediate support zone kN	Span zone, kN	Ending support zone, kN
Load bearing capacity at room temperature	50.3	18.5	33.6
The force before heating	23.5	5.5	12.5
The force just before a destruction	27.5	9.5	8.5

Fig. 12. The destroyed beam no. 1

Shear load bearing capacity in the middle span of the beam $V_{Rd,c}$ was calculated with the formula:

$$V_{Rd,c} = \left[C_{Rd,c} k \left(100 \rho_l f_{ck} \right)^{1/3} \right] b_w d, \qquad (2)$$

where: $C_{Rd,c}$ and k – are factors; ρ_l – the reinforcement ratio for longitudinal reinforcement; f_{ck} – the characteristic concrete compressive strength; b_w – the cross-section width; d – the effective depth of a cross-section.

Before the beginning of heating, there was a significant reserve of shear load bearing capacity in the tested beams. As a result of redistribution of internal forces caused by the heating, the shear force in the middle part of the heated span of the beam increased by about 75%. The increased shear force reached about 51% of the initial cross-section shear load bearing capacity (at room temperature). However, this load capacity decreased significantly due to heating. As a result of concrete degradation in the bottom part of the beam its active cross-section ("shear working") was significantly reduced. Furthermore, the reduction of shear load bearing capacity could also be caused by elongation of the main reinforcing bars. Their temperature reached about 600 °C (Fig. 11).

Table 3. Calculation data and calculated shear load bearing capacity of the cross-section situated in the middle part of the beam

	500 °C Isotherm method, formula (3)	*500 °C Isotherm* method, formula (4)	*Zone* method, formula (4)
Calculation data	$C_{Rd,c} = 0.18$, $k = 2.0$, $\rho_l = 1.31\%$, $f_{ck} = 35$ MPa, $b_w = 12$ cm, $d_{fi} = 10$ cm	$C_{Rd,c} = 0.18$, $k = 2.0$, $\rho_l = 1.74\%$, $f_{ctk} = 2.25$ MPa, $b_w = 12$ cm, $d_{fi} = 7.5$ cm	$C_{Rd,c} = 0.18$, $k = 2.0$, $\rho_l = 1.74\%$, $f_{ctk,fi} = 1.44$ MPa, $b_w = 12$ cm, $d_{fi} = 7.5$ cm
Shear load bearing capacity at high temperature, kN	15.5	12.8	10.2
Shear load bearing capacity at room temperature, kN	18.5		
Shear force just before the beam destruction, kN	9.5		

5. Estimation of shear load bearing capacity at high temperature

As it was described above, the destruction of the beams was caused by the loss of the shear load bearing capacity in the middle section of the heated span of the beam, where there were no stirrups. The redistribution of shear forces due to heating resulted in an increase of shear force in the middle section of the heated span of the beam from 5.5 to 9.5 kN (by about 73%). The calculated initial (room temperature) shear load bearing capacity in this part of the beam was equal to 18.5 kN (Table 2), but it decreased significantly due to high temperature exposure.

The estimation of the shear load bearing capacity in the middle section of the heated span of the beam (without stirrups) at high temperature just before the destruction of the beam is presented below.

Calculations were performed using three different methods in accordance with EN 1992-1-2 (2004):
  – *500 °C Isotherm* method, taking into account the concrete compressive strength;
  – *500 °C Isotherm* method modified in a way that the value of concrete tensile strength was considered instead of the compressive strength;
  – Modified *Zone* method based on reduced concrete tensile strength.

Calculation results and the most important data used for calculations are summarised in Table 3.

Both, the *500 °C Isotherm* and the *Zone* method are applicable only for standard fire exposure (EN 1992-1-2 2002). However, they could have been used because the temperature increase in the furnace chamber (Fig. 11) was close to the standard curve.

In calculations performed using the *500 °C Isotherm* method, it was assumed that the concrete layer with temperature exceeding 500 °C does not influence the cross-section load bearing capacity while the remaining part of the beam maintains its initial compressive strength (as at room temperature). The shear load bearing capacity was calculated with the formula (3) (EN 1992-1-1 2008). Isotherm 500 °C layout was determined based on FEM calculations (Fig. 10):

$$V_{Rd,c,fi} = \left[C_{Rd,c} k \left(100\rho_l f_{ck} \right)^{1/3} \right] b_w d_{fi}. \qquad (3)$$

In calculations performed using the modified *500 °C Isotherm* method cross-section shear load bearing capacity depended on concrete tensile strength. The isotherm separating active and destroyed parts of the cross-section was determined to be equal to the temperature at which the concrete tensile strength decreases as the concrete compressive strength at 500 °C. It is known that in the temperature of 500 °C the compressive strength of concrete with siliceous aggregate decreases by 40% (EN 1992-1-2 2004). The same reduction in concrete tensile strength occurs at the temperature of 300 °C (EN 1992-1-2 2004). Therefore, in order to estimate the shear load bearing capacity it was assumed that the "destroyed" concrete with temperature exceeding 300 °C, does not influence cross-section load bearing capacity.

In calculations performed using the *Zone* method the outer layer of the cross-section, which was entirely destroyed due to heating, was excluded from the analysis. To determine its thickness, the cross-section was divided into 16 parallel zones and the average temperature at the middle of each zone was calculated. A concrete tensile strength reduction coefficient ($k_{c,t}(\theta)$) was determined for each zone, based on the calculated temperature values. Finally, the mean value of concrete tensile strength reduction coefficient ($k_{c,t}(\theta)$) for the whole cross-section was calculated and the thickness of the destroyed zone was determined.

In calculations performed using the modified *500 °C Isotherm* method and the Zone method the shear load bearing capacity at high temperature was calculated with the formula (4). This formula was obtained by substituting into the Eqn (3) the relationship (5) which connects concrete compressive strength (f_{ck}) with concrete tensile strength (f_{ctk}) (EN 1992-1-1 2008):

$$V_{Rd,c,fi} = \left[C_{Rd,c} k \left(100\rho_l \sqrt{\left(\frac{f_{ctk}}{0.7 \cdot 0.3} \right)^3} \right)^{\frac{1}{3}} \right] b_w d_{fi}; \qquad (4)$$

$$f_{ctk} = 0.7 \cdot 0.3 f_{ck}^{\frac{2}{3}}. \qquad (5)$$

The best prediction was obtained using the Zone method (Table 3). The value of shear load bearing capacity of the cross-section at high temperature estimated with

this method was greater than the one obtained experimentally by only 7% (10.2 and 9.5 kN). The values calculated with the two other methods significantly exceeded the experimental result. This confirms that the *Zone* method is more suitable for predicting fire load bearing capacity of small cross-sections. Additionally, it can be noted that when determining shear load bearing capacity of a cross-sections without transverse reinforcement, one should consider that due to high temperature concrete tensile strength decreases much faster than concrete compressive strength.

Conclusions

The results confirmed the occurrence of a significant redistribution of internal forces in two-span loaded beams exposed to high temperature. In the examined elements the hogging moment increased by about 65–75% and the sagging moment decreased by about 25–30%.

However, the destruction of the tested beams happened not due to the bending but as a result of shear load bearing capacity loss in the middle part of the heated span of the beam. This destruction mechanism may appear a little surprising because before heating the shear load bearing capacity reserve was significant (approximately triple). However, as a result of shear forces redistribution, the shear force in the middle part of the heated span of the beam increased by about 75%, and the cross-section load bearing capacity was reduced approximately by half.

The redistribution of bending moments and shear forces in the tested beams was caused by the two most important phenomena: reduction in stiffness of the heated span and thermal elongation of concrete in the heated zone.

The performed calculations have shown that the best estimate of the shear load bearing capacity can be obtained with the *Zone* method.

From a practical structural fire design point of view, it must be noted that in some conditions the destruction of multi-span RC elements might occur due to the redistribution of internal forces resulting in significant increase of shear force in zone where it was very small before a fire exposure.

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