Ain Shams Engineering Journal 14 (2023) 102519

Contents lists available at ScienceDirect

Ain Shams Engineering Journal

journal homepage: www.sciencedirect.com

Bridges after the fire – Experiences, tests and repair methods

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A B S T R A C T

To date, only 1 746 bridges have been reported to collapse due to various effects. Of these accidents, 1 001 are due to hydraulic causes (friction, leaching), 520 collisions or overloads, 52 fires and only 19 times an earthquake caused the damage. Currently, there is no standard specifically developed for the fire behaviour of bridges. Therefore the designer can decide what methodology he wants to use during the design process. Two bridge fires have occurred in Hungary over the past three years. In this paper, the authors present the experiences of the fire investigation and the suggestions for the restoration.

1. Introduction

Bridge construction is as old as the history of humanity. Initially, people used the natural formations, and then by observing they started creating them. After the rediscovery of concrete in the second half of the 18th century, it appeared in the bridge construction at the beginning of the 19th century. It was first used for massive foundations and piers, but since the middle of the century, it has been used to build whole arch bridges. Although attempts were made to strengthen the concrete with iron, the reinforced concrete was only invented in the 1870 s. In the 1880 s, mixed iron was replaced by steel in the reinforced concrete, so the continuous development of the reinforced concrete could begin [1].

Bridges play an important role in economic development, and their construction is considered to be a major political, social and economic event. The lifespan of the bridges can be hundreds of years, with appropriate renovations. However, still today, only 1 746 bridges have been collapsed due to various effects. Of these accidents, 1 001 were due to hydraulic causes (friction, leaching), 520 collisions or overloads, 52 fires and only 19 times an earthquake caused the damage.

Due to the social, economic and political importance of the bridges, it is important to minimize the risk of collapse and to reduce the probability of the causes, which are leading to the collapse at the design phase. Considering fire as a possible cause of the damage, Naser and Kodur [2–4] developed a fire risk analysis method. The basis of the risk analysis is the structure of the bridge and the importance of the traffic on the bridge. The method allows bridges to be classified into risk classes. Based on it, it is possible to distinguish the low, medium, high and critical risks. For low and medium risk, no additional fire protection is required. However, in the case of structures of high or critical risk, additional fire protection shall be provided.

In order to carry out the risk analysis for fire, it is necessary to analyse the consequences of possible fires and also to examine the processes occurring in the structure of the reinforced concrete.

2. Nature of fire

There is currently no model specifically developed for calculating the fire loads, especially in case of bridges. The designer can decide what method to use during the design process, but it requires the knowledge of the nature of fire.

Fires endangering bridges were typically rapid and very intense. Fires in bridge could also break out due to wildfire, burning of stored materials etc. but experiences show that in most cases, an open-field liquid fire occurred. In most cases, the combustible liquid was fuel, usually gasoline or gas oil. The consequences of the fire, the maximum temperature and the amount of heat radiation depend on:

- the material characteristics of the flammable liquid,
- the amount of the flammable liquid,
- meteorological conditions (primarily temperature and wind speed),
- and the formation of an unlimited or limited puddle.

In many cases the well-known hydrocarbon fire (used for tunnels) is used to characterize the fires endangering the bridges [5]. However, these fire curves have been developed for tunnel fires, and experiences have shown that bridges have different fire characteristics [6]. The

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https://doi.org/10.1016/j.asej.2023.102519
Received 18 October 2022; Received in revised form 13 September 2023; Accepted 27 September 2023
Available online 24 October 2023

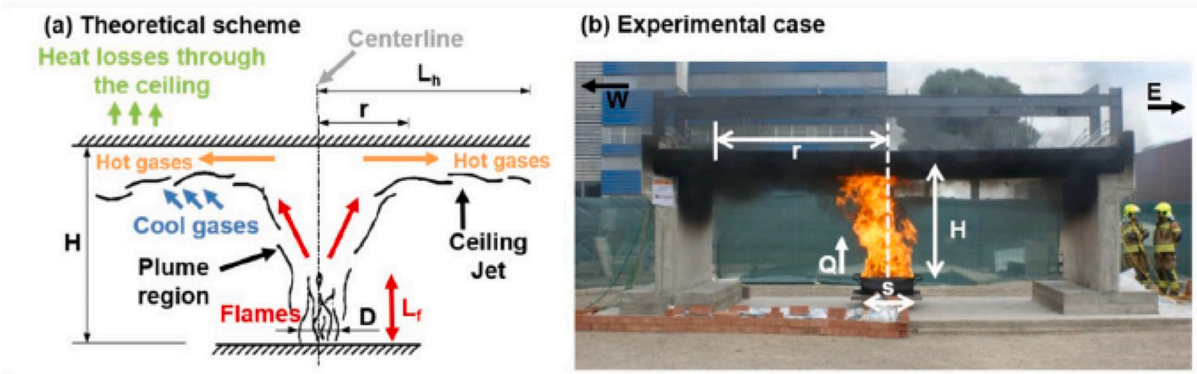

Fig. 1. Adaptation of the Heskestad method based on the real test .
Source:[7]

thermal effect on the bridges was investigated by [7,8]. They created the final fire model based on four different fire experiments. Based on their experience, the fire had a local effect, so they used the so-called flame jet models. Heskestadt [9] introduces an apparent point source instead of a point source in his model of the flame, which gives the real extent in the plane of the development of the fire. He also introduces the convective flame power. Instead of a constant temperature and speed profile at a given altitude, he assumed a Gaussian normal distribution in that plane. This is closer to the temperature and velocity profile of the real flame. He has omitted the simplification criterion for the density used by Boussinesq [6]. Alos-Moya [7] showed that the Heskestad method (Fig. 1) is applicable in case of vehicles burning under the bridges.

3. Behaviour of materials in case of fire

3.1. Behaviour of the concrete in case of high temperature

As the temperature increases, the strength characteristics of the concrete deteriorate. The concrete does not regain its original characteristics during the cooling. This is because irreversible processes take place in the structure of the concrete under the influence of heat. As a result of it, the structure of the concrete is destroying slowly. According to [10], the failure of concrete due to fire load is basically based on two reasons:

– chemical transformation of the components of the concrete,
– the layered detachment of the surface of the concrete (spalling).

These include internal micro-cracks, which are evolving during the heating. The chemical processes due to the high temperature in cement stone and concrete can be investigated by thermoanalytical methods (TG/DTG/DTA). The TG (thermogravimetric) and DTG (derivative thermogravimetric) curves allow quantitative analysis of mass-related transformations. DTA (Differential Thermal Analysis) curves are used to monitor the evolution of exothermic or endothermic processes in samples as a result of the temperature increase.

According to Thielen [11], the changes in the strength characteristics of the concrete at high temperatures depend on the following parameters:

– type of the cement,
– type of the aggregate,
– water-cement ratio,
– aggregate-cement ratio,
– initial moisture content of the concrete,
– the type of heat load.

At around **100** °**C**, the weight loss is caused by the water leaving from the macro-pores. Ettringite ($3CaOAl_2O_3 \bullet 3CaSO_4{\cdot}32H_2O$) decomposes between **50** °**C** and **110** °**C** [12]. Further dehydration processes take place around **200** °**C**. It leads to a further slight increase in weight loss. The weight loss of specimens with different initial moisture content is different until they lose the pore water and the chemically bound water content. Depending on the initial moisture content, the difference in weight loss is significant, especially in the case of lightweight concrete. No additional moisture content related weight loss can be detected from **250 to 300** °**C**. Between **450** °**C and 550** °**C**, the decomposition of non-carbonated portlandite occurs ($Ca\ (OH)_2 \rightarrow CaO + H_2O \uparrow$). This process causes an endothermic peak and additional weight loss [13]. In the case of conventional concrete, the transition of quartz from α to β results a low-intensity endothermic peak at 573 °C. The conversion of quartz results in a volume increase of 5.7 % [14]. It results in significant damage in the concrete. Above this temperature, the concrete has no significant load-bearing capacity. At **700** °**C**, calcium silicate hydrate ($3CaO\ 2H_2\ 2SiO_3\ H_2O$) compounds decompose with water loss, which also results in a volume increase and further loss of strength [15]. In case of concretes with quartz gravel aggregate, the porosity of the cement stone and the porosity of the contact zone between the cement stone and the aggregate increases up to 150 °C. Cracks can occur in the contact zone up to 150 °C, which can be explained by the different thermal expansion of the aggregate and cement stone. The structure of the cement stone is stable up to 450 °C, but micro-cracks can occur under this temperature. However, between the temperature of 450**,** 550 °C, the porosity increases due to the decomposition of the portlandite. After that up to 650° C, the structure of the cement stone does not change. Above 650 °C, the decomposition of CSH compounds begins, and the number of capillaries increases. Above 750 °C, the diameter of the pores increases greatly. The size of the various micro-cracks depends on the maximum grain size of the [15]. Due to the chemical and physical changes in the concrete, the strength characteristics of the concrete also change. In addition, layered spalling of concrete surfaces may occur. There are two main mechanisms of it [16–18]:

1. the pressure of the water vapour strains the surface of the concrete
2. the strain zone cannot absorb the new forces resulting from the thermal expansion, so it will drop off and fall off.

Spalling of the surfaces of the high strength concrete is usually caused by stresses due to the rise of the temperature; in case of normal concrete, the water vapour from the concrete usually strains the surface layers.

According to Hertz [17] in addition to the high-strength concrete, even high-density concrete is dangerous for the explosive spalling of the concrete surface. (for example, concretes that contain silica fume).

Hertz found in his experiments that in many cases, the spalling of the surface of the concrete occurs during the cooling. The critical temperature for spalling of the concrete surface is 374 °C. He also determined that under a water content of 3–4 %, the chances of the spalling of the concrete surface is very low.

The following factors have an influence on the spalling of the concrete surface:

- external factors: the nature of fire, the size of the external loads on the structure;
- geometrical characteristics: geometrical details of the structure, size of concrete cover, number and location of steel reinforcement;
- Composition of the concrete: size and type of the aggregate, type of the cement and the supplementary material, number of pores, polypropylene fibre content, steel fibre reinforcement, water content, permeability and strength of the concrete.

Depending on the compressive force on the concrete, the chances of the spalling of the surface vary. In case of low utilization of the cross-section, the geometric sizes of the cross-section can be reduced.

3.2. Behaviour of steel at high temperatures

During the fire load, the physical characteristics of the steel change, as a result of the temperature increase. The steel undergoes a number of phase transitions at higher temperature ranges. The speed of the change is different in case of hot rolled and cold-formed steels. This can be explained by the different manufacturing technology of steels [19]. This is because of the sudden drop of the inner stress applied during cold forming due to high temperature. With the increasing temperature, the thermal conductivity, the specific heat, the strength and the stiffness characteristics of steel are constantly changing.

From a structural engineer point of view, steel strength, stiffness and the elastic modulus are among the most important variables. However, from the thermal engineering and fire protection point of view, the change of the thermal conductivity and the specific heat is also very important. They have an influence on the rate of the heating of the steel and the amount of the energy required for heating, that is, the degree to which the strength and stiffness characteristics change. The thermal conductivity coefficient decreases by increasing the temperature. It means that less and less heat can pass through the unit cross-sectional area in a specific time [20].

The specific heat is the amount of energy per unit mass required to raise the temperature by one degree Kelvin. It hardly changes up to 735 °C, but it takes much energy to raise the temperature. The reason for that is that at 723 °C (PSK line), the conversion of the eutectoid consisting of ferrite and cementite begins, and austenite appears in the alloy. Energy is used for the chemical transformation and not for raising the temperature. Then, at the end of the transformation process, at about 800 °C the specific heat decreases again [19].

The change in yield strength structural steels (A_c) start to decrease at 400 °C, whereas it starts to decrease at 300 °C in case of cold-formed steel (A_b) and this change is considered linear up to 700–800 °C. However, the decrease in the elastic modulus begins at 100 °C, and the steel starts to soften. In the case of hot rolled steel, the critical temperature is 500 °C, and in the case of cold-formed steel, it is 400 °C. The critical temperature is where the material changes from linearly elastic to plastic ductile behaviour, i.e., undergoes large deformations at relatively low load [20].

4. Our fire experiences

4.1. Type of the damage and the nature of fire

The nature of the fire can be characterized as a function of the amount of heat or thermal radiation.

By calculating the material damage, two different levels of damage can be given [21]:

Table 1
General values of the critical radiation intensity of the considered substances.

Material	Critical radiation intensity [kW/m^2]	
	1st level of damage	2nd level of damage
wood	15	2
synthetic material	15	2
glass	4	–
steel	100	25
concrete		

- First level of damage: Ignition of surfaces exposed to heat, it is followed by fractures or other types of structural damage.
- Second level of damage: Damages such as discolouration of certain surface materials, detachment of paint or substantial deformation of structural elements.

The values in Table 1 are to be interpreted as general guidance, which applies to radiation that is not too short (for example, more than 30 min). Shorter fires require a finer approach, which also takes into account the geometry of the component and its position related to the radiation. We have modelled the thermaldevelopment by a liquid fire. We give the heat radiataion in fuction of the distance from fire on Fig. 2. This may be particularly relevant in case of assessing the second-level of damage to the steel structures. The critical radiation intensity values for the various materials are shown in Table 1. Considering that the paint layer and the concrete surface (layered spalling) were damaged in case of the two examined bridges, the speed of the heat radiation was also calculated here. In each of the two cases described below, a fuel tank filled with 1200 L of fuel (n-nonane is used as reference material for modelling) was assumed. We show the result of the modelling in Fig. 2. Modelling can lead to the second level of damage, which is the spalling of paint. However, the first level of damage is not to be expected.

4.2. Fire of the bridge over the M1 highway, Hungary

On 27th of January 2017, at 17:59, there was an accident on the left lane of the Miklóstanya highway bridge over the 71 + 794 km section on the M1 motorway. A truck carrying rubber-making material collided with a pier next to the emergency lane. Accident officials say the vehicle crashed to the pier at a speed of about 80 km/h. The tractor rolled onto the pier and ignited. Due to the burnt tractor, the pier column, the pier headers, the precast bridge beams of the adjacent openings, the cross member of the prop and the affected sections of the bridge edges were subjected to the permanent heat load. The material test was carried out 3 month after the fire case (on 31. 3. 2017).

4.2.1. Data of the bridge

The bridge was built in 1975. The superstructure is an EHGE bracket with a reinforced concrete slab. The biggest opening was 15.0 m. Based on the original plans of the bridge, the pier and the beam were made of concrete with a cement load of 350 kg/m^3 B280 (C16/20). The concrete quality of the bridge beams (Hoyer beams) is B400 (C25/30). The reinforcing steel grade of the pier and the beam is 6 mm in diameter B 38.24 (S240C), and the other steel is B 50.36. In 2003 the bridge was renovated. Corrosion protection also affected the main supporting elements, with a plastic-based salt protection coating on the bridgehead and headers. In Hungary the water contant in briges is not more tahan 5 m%.

4.2.2. Visual examination after the fire

After the visual examination of the bridge piers, we determined that the heat load struck the slope side of the pier more strongly than the route side. The concrete surface is whitened in a clearly visible area because the highest flame effect was there (Fig. 3). Due to the high temperature on the slope side of the pier, the corners of the concrete

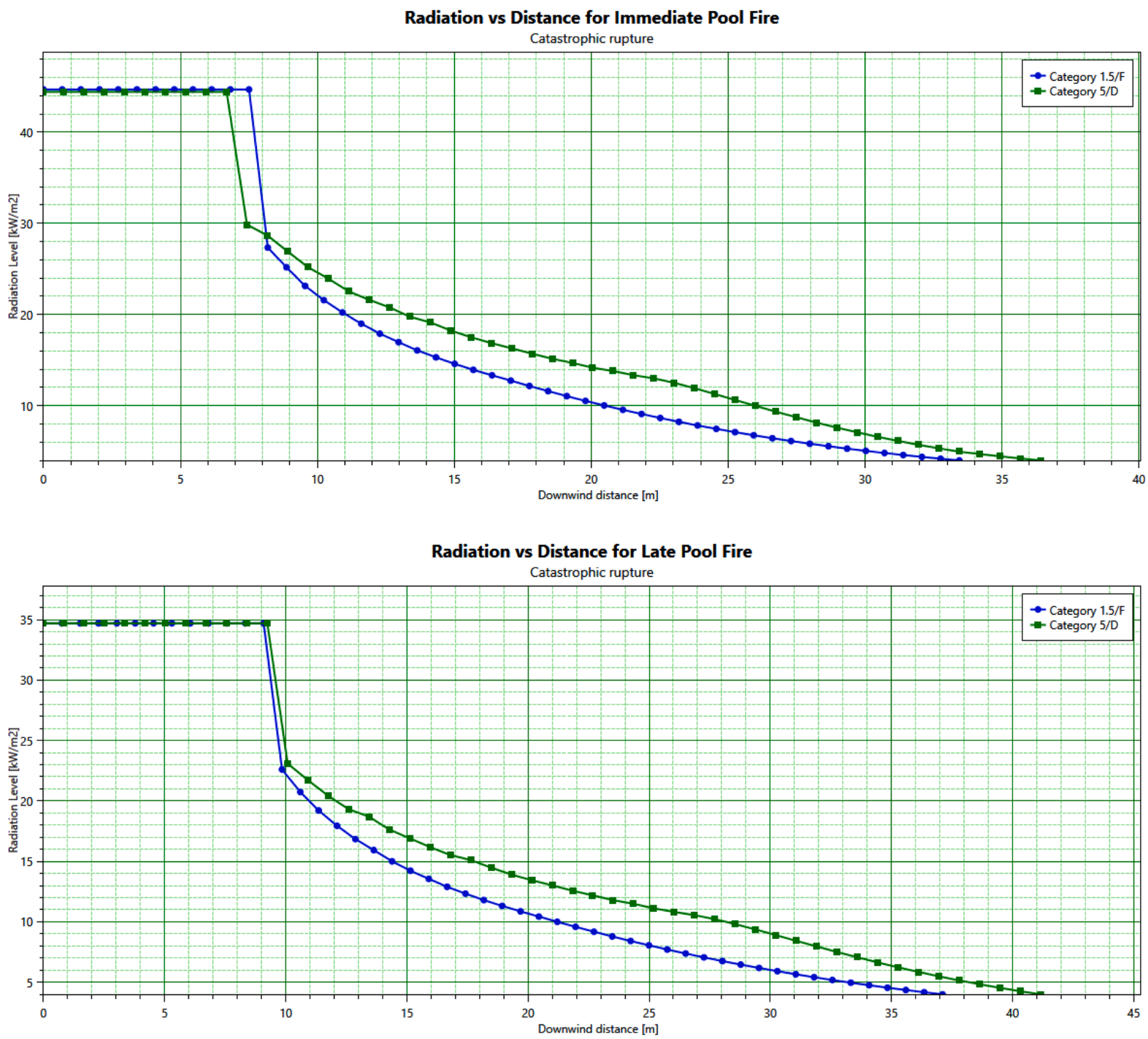

Fig. 2. Heat radiation as a function of the distance.

Slope side of the pier *The spalled and discoloured concrete*

Fig. 3. Damage of the bridge piers after the fire (31/3/2017).

Fig. 4. Header and main beams after the fire (31/3/2017).

were detached (Fig. 4). The surface of the concrete became pink at the outer few millimetres. It suggests that the temperature was above 500° C *(*Fig. 4*)*. The salt protection coating on the slope side of the bridge pier was burnt in many places. However, the cementitious layer under the salt protection coating is only slightly damaged by the heat load. On the route side of the bridge pier, the plastic coating did not burn down completely. It cracked and spalled in some places. It suggests that the maximum temperature of the heat load was about 200 °C.

On the fire side of the beam, the surface of the concrete has been spalled in some places, especially at the corners of the element (Fig. 4). The surface of the main structural elements became sooty. In the slope-side opening (1–2 opening) the concrete cover of the beams spalled in one or two places. On the side of the route, the remaining formwork ignited. Here, the surface of the beams was whitened, and the heat load on the beams was probably the highest here.

4.2.3. Examination methods after the fire

Due to the influence of fire and high temperature, $Ca(OH)_2$ decomposes in the concrete, thus the pH value of the concrete decreases, so the surface test was solved by measuring the pH-value. The pH was measured with a phenolphthalein solution. The colour change of phenolphthalein happens at pH 9, above this value, corrosion protection of the steel reinforcement is solved. During the measurement, the freshly peeled concrete surface was examined. In case of the beam and the column, the pH decreased only in the outer few millimetres, but this layer did not reach the steel reinforcement anywhere. It shows that the temperature around the steel bars did not reach the 500 °C and the alkalinity of the concrete is suitable on the surface of the steel bars. The estimated concrete strength of each component was determined by the rebound hammer test. During the rebound hammer test, the direction of the blow was horizontal in case of the header and the column and vertical (upwards) for the main beams. The rebound hammer strength estimate showed that the post-fire strength of the concrete of the header, the column and the main beams classifies as B500 (C35/45). It is significantly higher than the planned B280 concrete quality or the B400 for the EHGE beams. Due to the carbonation of the concrete, it is recommended to change the strength class to B400 (C25/30) instead of B500 (C35/45). The post-fire concrete strength (taking into account the strength reduction effect of carbonation) significantly exceeded (or reached for the EHGE beams) the planned concrete strength.

After the polishing of the metal surfaces, the strength of the steel was estimated with a Poldi hammer. According to the results, the reinforcing steel quality corresponded to the B 50.36 reinforcing steel, even after the fire load. Based on the measurements, it can be stated that the reduction of the assumed steel strength was not necessary neither for cold-formed nor hot-rolled steel.

4.2.4. Finite element modelling of the cross-sections

The warming of the cross-sections was checked with a finite element model. This modelling was performed with ANSYS Workbench R16.2 software. The goal of the development from the models was to modell the het transfer in the cross section. The heat conductivity parameters of the concrete were calculated according to the Hungarian standard [22]. The time course of the fire load of the examined concrete was considered with the outdoor fire model. The duration of the fire effect was set for 2 h. In case of modelling the column's fire load, a 4-sided thermal load was supposed for safety reasons.

Fig. 5 shows a cross-section of the column with the isotherm lines formed during the fire in the second hour. Temperature range above 500 ℃ is indicated by the red colour.

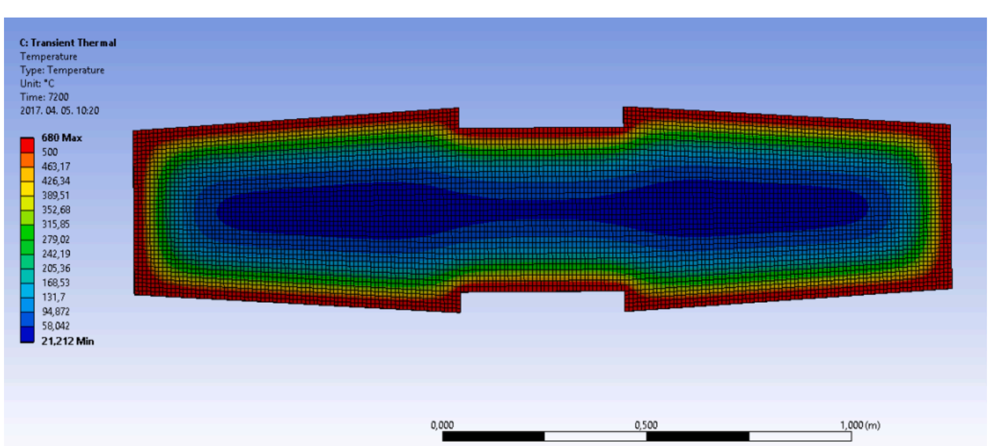

Fig. 5. Modelling the isothermal lines formed in the column in the second hour of fire.

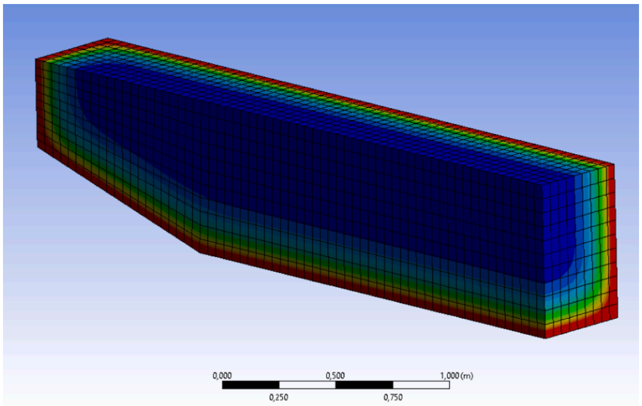

Fig. 6. Thermal model of the header console.

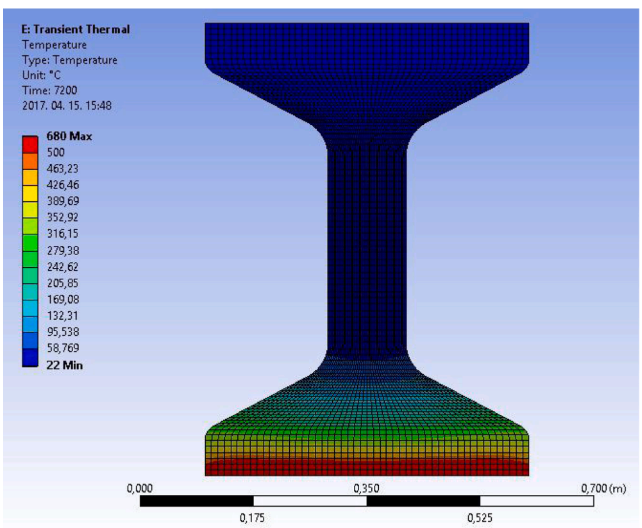

Fig. 7. Modelling of the isothermal lines formed in the EHG beam in the second hour of fire.

Based on the finite element modelling, the depth of the burned part (over 500 ℃) in the cross-section of the column is 5 cm maximum at the corners and 2 cm at flat surfaces after two hours of fire load.

In the case of the header consoles due to the variable cross-section, only the quarter of the total cross-section was modelled (Fig. 6).

In case of modelling the fire load of the header, a three-sided thermal load was assumed for safety reasons. The isothermal lines formed during the fire are shown in the cross-section of the consoles after two hours. Temperature range above 500 ℃ is indicated by the red colour.

Based on the finite element modelling, the depth of the burned part (over 500 ℃) in the cross-section of the column is5 cm maximum at the corners and 2 cm at flat surfaces after two hours of fire load.

In case of modelling the fire load of the EHG beam, a one-sided thermal load was assumed for safety reasons. Fig. 7 illustrates the isothermal lines formed during the fire at the cross-section of the EHG beam after two hours. Temperature range above 500 ℃ is indicated by the red colour.

Based on finite element modelling, the depth of the burned part (over 500 ℃) in the cross-section of EHG beam is 2,5 cm maximum, at the flat surface in the lower zone after two hours of fire load.

4.2.5. Repair and renovation strategy

The investigation confirmed that the accident did not change the stability and the carrying capacity of the structural elements of the bridge. However, the structural elements affected by the mechanical and fire effects of the accident caused such faults and damage, which -without proper repair - would result in a significant acceleration of the deterioration of the structures and a decrease in the life expectancy.

During the repair, the following restoration works were needed:

At the bridge pier and at the header, the concrete was damaged, cracked or spalled in several places. These parts needed to be repaired.

The surface of the concrete was knocked, and the loose parts were removed. The sooty concrete surface was cleaned with chemicals as this neutralized the harmful effects of soot.

After that, the exposed reinforcing steel was coated with an anti-corrosive material.

On the removed concrete sections, a bonding bridge was formed between the old reinforced concrete structure and the repair material.

In case of minor dameges, the repair material could be applied with hand tools.

Depending on the size of the concrete part that has was removed, it was also possible to use shotcrete for replacement purposes. On the parts which needed to be reinforced with shotcrete, rawlplug and fixed steel mesh had to be applied to ensure proper mechanical functionality with the original reinforced concrete structure.

If reinforcing was needed, it was designed in accordance with the mechanical plans. After that, the shotcrete reinforcement was prepared. The recommended concrete was C30/37-XC4-XF2-XD1 (min cement 300 kg/m^3, max w/c 0.5). Consistency and the maximum aggregate grain size depended on the technology.

After the repair of the concrete, it was also necessary to replace the protective coating corresponding to the original layer structure of the bridge pier or header, in order to ensure adequate durability.

After the restoration work, the bridge became suitable for further use without limits to its carrying capacity.

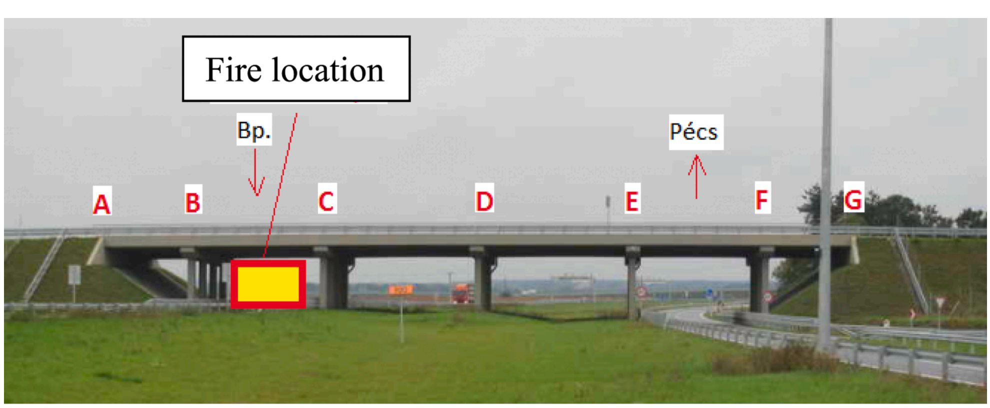

Fig. 8. Side view and location of the fire (from Budapest).

Fig. 9. Marking of the structure elements damaged by the fire- Pier B view from Mohács (angol felítarok).

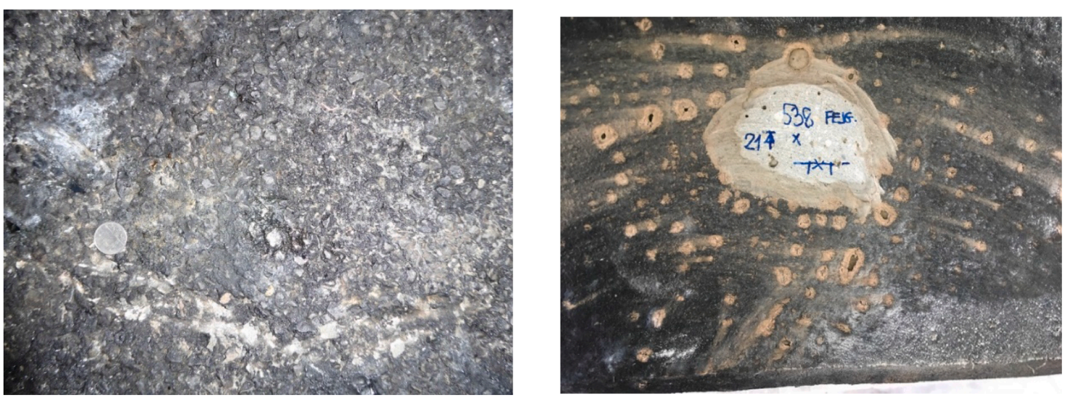

Fig. 10. Scaled B-4 salt protection coating and the polished concrete surface underneath (1/6/2017).

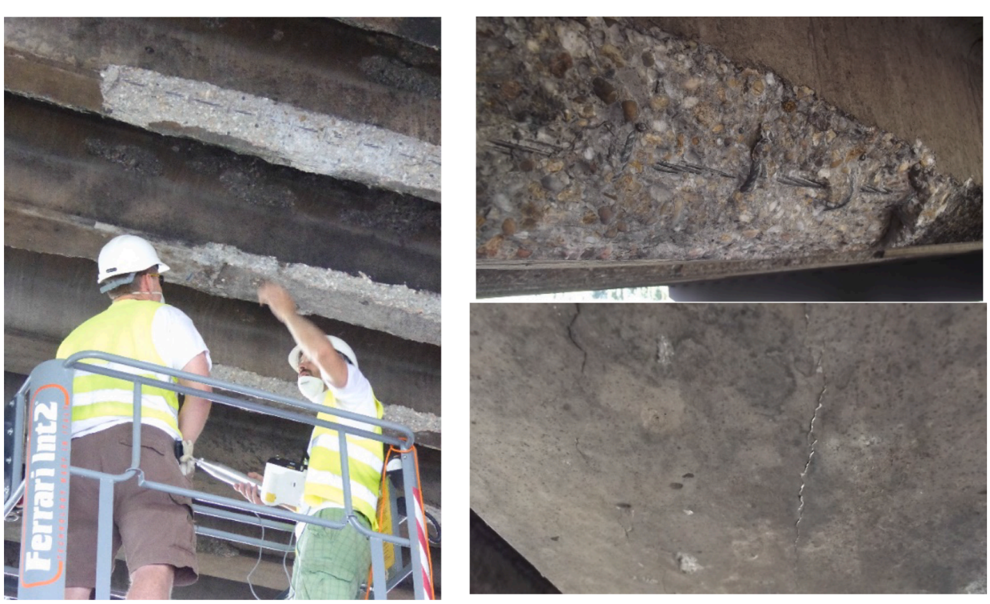

Fig. 11. Damage of the bridge beams and the sampling (1/6/2017.

Fig. 12. Surface of the spalled concrete and the broken aggregate grains. (12.07.2017).

4.3. Fire of the bridge over the highway M6, Hungary

On 29th of May 2017 at 8:29 am a cab of a truck caught fire for unknown reasons on the M6 motorway. The cab of the truck burned down completely, but the gas tank did not get damaged. The flames caused minor damages in the front of the trailer, but the cargo stayed unharmed. The fire burned for almost half an hour, and beside load-bearing structural elements, the guardrail and the protective edge of the pier were damaged. The material test was carried out in 1 week after the fire case (on 01. 6. 2017).

4.3.1. Data of the bridge

The bridge over the highway M6-M60 was built in 2010. The side view of the bridge is shown in Fig. 8. The span of the section affected by the fire was 12.41 m. This involved 25 FPT-45/11.80 beams.

The columns and the header were made of C35/45 concrete and B500 B steel reinforcement. The prestressed beams were made of C50/60 concrete and Fp93 / 1860 cold-formed steel. The structure of the bridge: six-hole, continuous, multi-support beam bridge. The superstructure consists of FPT-45 precast bridge beams with an on-site reinforced concrete slab. The structure gauge of the motorway branches under the bridge is 4.70 m + 0.51 m (reserve). There is no public utility on the bridge. The bridge was classified to the non-flammable class ("E"). In Hungary the water contant in briges is not more tahan 5 m%.

4.3.2. Visual examination after the fire

Structural elements away from the fire typically have a sooty surface. However, due to the higher temperature near the fire, there was less soot on the surface. The heat reached directly one of the piers, one of the headers and nine prefabricated reinforced concrete bridge beams (Fig. 9).

The salt-protection layer was scaled by the fire on the pier and the header (coated with B-4 plastic cement-based). On-site examinations showed that the concrete layers under the salt protective coating are barely damaged by heat. The sooty coating was cracked and detached at some points but was not completely burned down (Fig. 10).

4.3.3. Examination methods after the fire

Based on the rebound hammer test (estimated concrete strength on the surface was C16/20 on the beams and piers) and the condition of the coatings, the maximum heat load temperature of the B-2 pier and B-header was approximately 500 °C. Based on it, the zones of the cross-section affected by strength loss in these structural elements were negligible (about 4–5 cm).

During the mechanical inspection of the pier and the header, because of safety reasons, the size of the cross-sections exposed by fire is reduced by 5–5 cm for the calculations. In the reduced cross-section, the original values of concrete strength were taken into account.

The concrete cover spalled at the lower edges of the web of four of the fire affected bridge beams 3–5 m away from the header four of the fire affected bridge beams (Fig. 11).

The spalling occurred at different lengths on the beams, typically in right-angled triangle cross-sections of approximately 12–15 cm legs. At the border, longitudinal cracks could be observed. These mark the borderline of the concrete parts, which did not spall but can be easily detached with a hammer (Fig. 12).

The spalling of the lower edges of the bridge beams can be explained by the high concrete strength (C50/60) and the sudden high vapour pressure due to the associated concrete structure. Based on experience and the case study of the fire, the edges of the concrete were spalled around 10 min from the start of the fire.

In case of the most heavily damaged bridge beam, the concrete cover on the lower edges spalled significantly, and longitudinal cracks were formed. Non-destructive strength estimation, thermal modelling and static monitoring of these beams were required. In the case of the middle beams, the amount of spalling was higher. At the spalling, the grains of the aggregate were ruptured, so it suggests that the concrete strength was higher in case of these beams. For the other three beams, the cross-section was only slightly damaged. The concrete cover spalled only at the edge from the side of the fire. A longitudinal crack was observed in the middle of the lower flange of the beams. During the phenolphthalein measurement, the phenolphthalein solution was sprayed onto the freshly peeled and professionally dusted concrete surfaces. At the two sampling points, the sprayed solution immediately turned to pink colour on the surface of the cracked cement stone. The thickness of carbonated concrete layers of the FPT beams is negligible (<1mm). The steel bars were not reached at any of the test areas.

Due to the flue gases from the burning plastic and their polluting effect, the chloride content and carbonation depth of the bridge beams were also needed to be determined. At the test area, powder samples were taken from the concrete zones (1) 0–2, (2) 2–4, (3) 4–6 cm from the concrete surface and were stored in hermetically closed plastic bags. The pH of the powder sample was measured in a distilled water suspension by using a TESTO 206 type pH meter. The chloride ion content was determined by the Mohr argentometry method. The chloride ion content in the concrete is below the permitted limit, so the extent of it is not dangerous for either the steel reinforcement or the concrete.

Non-destructive tests were done to estimate the strength of the concrete. Based on the rebound values measured with the rebound hammer, the moderately damaged bridge beams showed a decrease in strength of 30–47 % and an average decrease in strength of about 40 %. It can be concluded that the temperature at the measurement points was about 500–700 °C.

After polishing and cooling of the metal surfaces, the strength of the steel elements (hot rolled and cold formed as well) was measured with a Poldi hammer. Based on the measurements, the reinforcements (hot-rolled) and prestressed bars (cold-formed), which were directly exposed to heat load, had a loss of strength of approximately 20–30 %. In the case of the modulus of elasticity, a decrease of about 20 % was taken into account. For this bridge, an external fire curve was used, which clearly

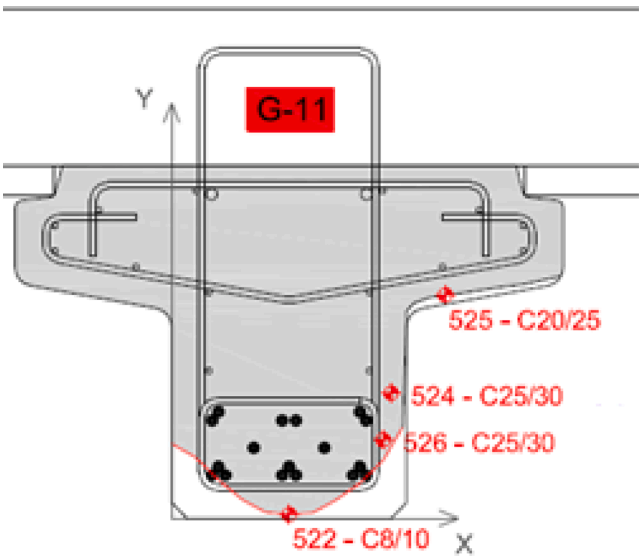

Fig. 13. Cross-section of the most damaged beam (in red). (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

shows that the maximum temperature of the heat load is around 700 °C.

4.3.4. Finite element modelling of the cross-sections

A finite element model was created with the ANSYS Workbench R16.2 software. The goal of the development from the models was to modell the het transfer in the cross section. The thermal conductivity parameters of the concrete were calculated according to the Hungarian standard [22]. The duration of the fire was recorded as half an hour based on the disaster management report, and the video record and an external fire curve was used. We used the most damaged beam (G-11) during the modelling. The damage level of the beam is shown in Fig. 13.

The following boundary conditions were used in the model:

1) 2D heat model *(*Fig. 14*)*.
2) Standard external fire load on three sides of the beam.
3) Spalling after 10 min, after that for a further 20 min, a reduced cross-section was used according to the heat load. The spalled cross-section was taken according to the on-site measurements.
4) Total time of the heat load: **30 min.**

The course of the isothermal lines is illustrated in Figs. 15 and 16.

4.3.5. Repair and renovation strategy

During the renovation of the bridge structure the following repair works must be done:

I: The salt-protective cover on the surface of the **B-2 bridge pier and B-2 header** became flaky as an effect of the fire. The sooty salt-protective coating was cracked and spalled in some places, but the plastic coating was not completely burned. The strength of the concrete layer under the salt protective coating has decreased as an influence of the heat. However, the surface is suitable for a new salt-protective coating. Based on the control calculations, the carrying capacity of a pier cross-section with concrete cover is safe even after the fire. *Suggestion for the repair*: The standard salt-protective coating of the B-2 pier (B-4 coating) shall be removed on the sides affected by the fire and restored to its original condition.

II: The lower surfaces of **the bridge superstructure**, as well as the sides of the edges, were smeared with soot during the fire.
Suggestion for the repair: The soot from the surface of the concrete and other contaminants from the bridge structure must be removed (for example with high-pressure cleaning). Due to the negligible chloride content, no special chemicals or detergents are required for the cleaning.

III: On the **G-7 to G-12 bridge beams between B-C supports,** there are visible damages. In addition, there was also concrete spalling due to the vapour pressure and the associated longitudinal cracks. No additional cracks (for example due to traffic reasons) are visible after the fire. The level of carbonation and chloride penetration in the beams is negligible. In the zones affected by spalling and in other parts affected by heat, the strength of the concrete was reduced by an average of 40 %, and the strength of

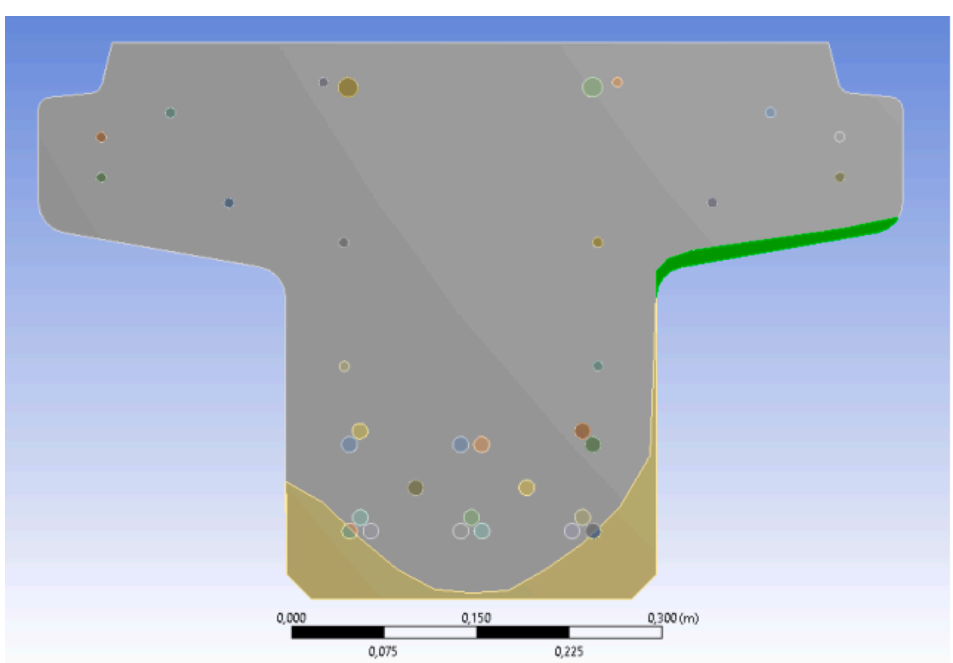

Fig. 14. G-11 beam, 2D model, spalled cross-sections according to on-site records.

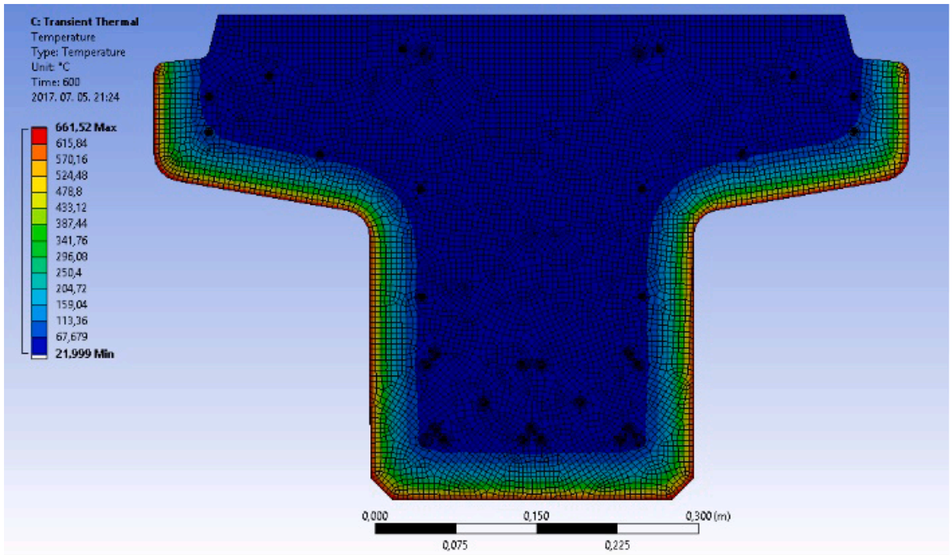

Fig. 15. Position of isotherm lines and prestressed steel bars at the 10th minute at the time of the considered spalling.

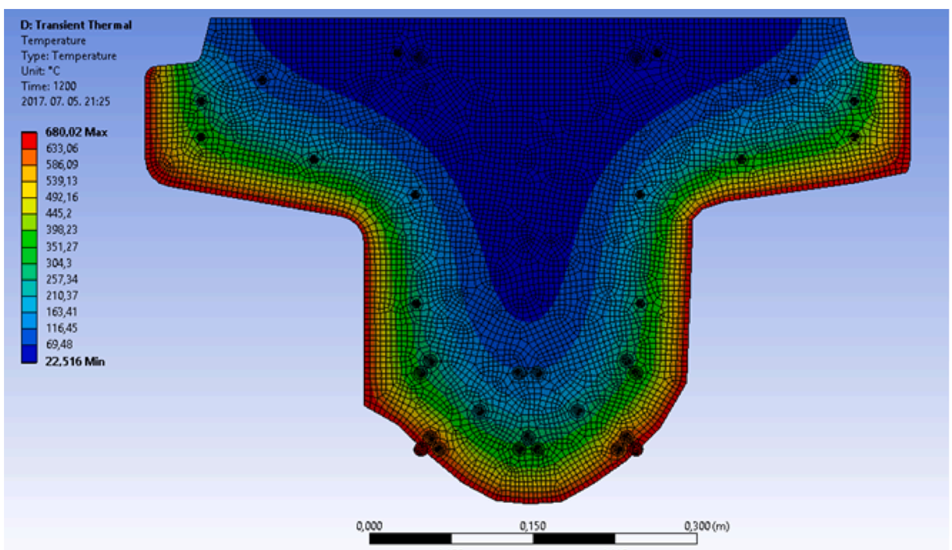

Fig. 16. Position of isotherm lines and prestressed steel bars at the 30th minute with a reduced cross-section.

the outer cold-formed steel bars was reduced by 20–30 %, but their replacement is not justified. The carrying capacity of the damaged bridge beams was reduced by nearly 10 % by the fire. However, it can be stated that the beams with reduced carrying capacity are still suitable for roads with sign “A”. Damaged bridge beams must be restored to their original condition as soon as possible.

5. Summary

Still today 1 746 bridges have been reported to collapse due to various effects. Of these accidents, 52 was due to fire. There is currently no specifically developed standard for the fire behaviour of the bridges. Therefore the architect can decide what methodology he wants to use during the planning. In the reinforced concrete, several chemical processes take place as an effect of fire. The damage of the structure of the reinforced concrete is determined, by the magnitude of the maximum temperature, the amount of the heat radiation and the time of exposure. Two bridge fires have occurred in Hungary over the past three years. The fires were caused in both cases by a truck. The fire tests and the fire modelling proved that the structural elements of the bridge were not irreversibly damaged, they could be repaired. The authors summarized the methods for the examination of the bridge structure, the results of the measurement and the suggestions for the restoration.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Acknowledgements

The research for this study was supported by the Higher Education Institutional Excellence Program announced by the Ministry of Human Resources, within the BME FIKP-WATER thematic program.

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