# Assessment of technical condition and repair of steel structure elements on the example of fire damage in a warehouse building

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**Abstract.** The paper analyses a case study on the structural assessment of warehouse building partially damaged by fire caused by external source (fire of lorries close to the building). The authors focus on the site investigations and laboratory test results prior to assessing actual condition of the structural elements. Both strengthening concept and repair procedure of a steel column are addressed here. A short literature survey in the paper regards fire damages and its impact on the entire structural systems and its members.

## 1 Introduction

Fire is an accidental and unexpected situation due to the entire domain of structures. The U.S. National Fire Protection Association reported that an average of 38,000 fires at industrial or manufacturing properties was noted between 2011 and 2015 every year in the U.S.A., the annual fire losses are estimated at 16 civilian fatalities and 273 civilian injures [1]. Every incident is a social and economic disaster. In order to minimize the threat to life and property, specific standards should be implemented in the building design process (see e.g. [2, 3]). The obligatory fire protective signal systems in buildings may not cover the exterior (building vicinity). The fire hazard is possible to come from the building exterior. The possible external sources of fire are mechanical vehicles parking close to the buildings. The mechanical vehicle short-circuits or others factors are possible sources of fire, eventually bring an extreme danger to life and property, see Fig. 1 and Fig. 2. It should be noted that, obtaining the rules of proper vehicle parking reduces the risk of fire hazard.

The investigations on fire damage and the impact of fire conditions to steel structures has been taken by an entirety of engineers and scientist researchers. Lawson [4] presented a development review in terms of elemental performance in standard fires, the full-scale behaviour of floors and structural systems in natural fires. Maślak [5] discussed structural analysis of members in fire conditions. Krentowski and Tribiłło [6] investigated failure and reconstruction of a fire-destroyed sport hall cover. Maślak [7] presented a testing method of structural steel in post-fire conditions and the possibility of its continual use in load-bearing elements. Jopek [8] presented a case study of fire event analysis in a production hall in Łyse, Poland. Biegus [9] examined the steel roof structural resistance in the hall damaged by fire

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and the repair concept. Glema et al. [10] analysed the yielding impact of vertical bracing on the cross-sectional force redistribution and stability of a primary steel hall structure in fire conditions. Gwóźdź et al. [11] presented the reconstruction strategy due to fire-damaged load-bearing structures of steel industrial halls. Ha et al. [12] presented a case study on the rehabilitation of a fire-damaged main control room of a thermal power plant and described the results of site investigations and laboratory tests. Kowalski [13] addressed repair preparation of an industrial steel structure after fire. Łumkowski et al. [14] investigated the resistance of unprotected steel beams under fire conditions, they compared the tests results with simple and advanced computational models. Kowalski [15] focused on repair and strengthening of a steel multistorey industrial tower after fire by means of prestressed components. Piroglu et al. [16] presented an experimental study on unprotected structural steel members exposed to fire in an industrial factory. Woźniaczka [17] performed a computational fire simulation of steel warehouse hall storing recyclable materials. The authors are aware of the fact that the review is partial only, paying attention to the chosen studies concerning a wide and comprehensive concept of fire impact on buildings. Nevertheless, this field is regarded in many engineering and scientific investigations. The expert opinions of fire-damaged structures should follow the situ inspection and testing specimens taken directly from real structural elements. General, arbitrary evaluation of mechanical parameters of fire-affected elements is not sufficient.

The research is bound to determine durability and strength of structural elements of warehouse buildings affected by fire conditions. The investigations partially contribute to the expert opinion on load-bearing capacity of structural elements, providing possibilities of safe load carrying by means of elements after fire events. The laboratory tests on steel specimens taken from fire-affected column were carried out in order to assess mechanical properties.


**Fig. 1.** View of fire damaged lorries and warehouse façade.

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**Fig. 2.** View of fire damage to the roof elements.

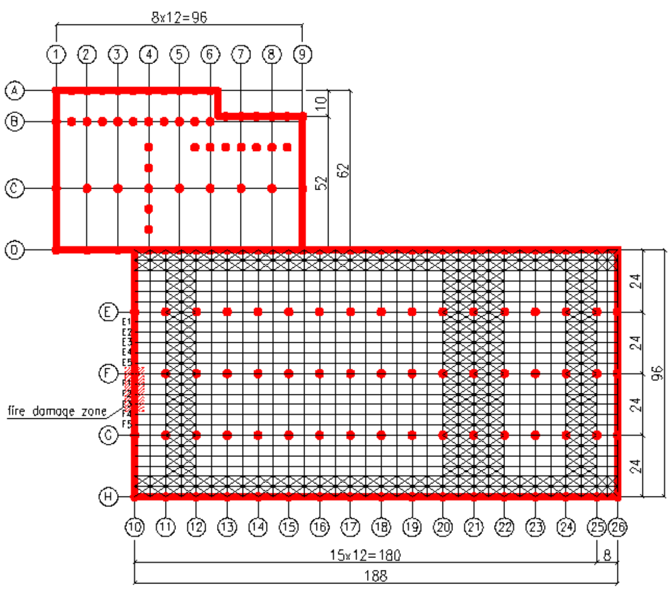


**Fig. 3.** Floor projection of warehouse building.

## 2 Building description

The warehouse building (see Fig. 3) including an office function was an approximate amount of 24000 m$^2$ covered area. The insulated wall assembly with sheet cladding and insulated membrane roofing on top of self-supporting sheets is applied as an exterior wall and a roof system. The main structural members of the warehouse building are:

- reinforced concrete column of a square cross-section 60x60 cm, spaced in a rectangular grid pattern 12 m by 24 m. The design concrete class is C30/37 (see [18] its characteristic cylindrical compressive strength equals 30 MPa, characteristic 28-day compressive cube strength is 37 MPa).

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- steel truss of a 24 m span, located parallel to the structural axes numbered from 11 to 25, every 12 m.
- steel girder (purlins) of a 12 m span spaced between structural axes numbered from D to H, every 4 m.
- the I-beam steel columns spaced every 4 m between reinforced concrete column in axes 10 and 25. The designed steel class is S355J2G3 (see [19], the minimum yield strength is $R_{eH}$=355 MPa, the tensile strength range is $R_m$ = 470 to 630 MPa)

## 3 Visual inspection and geodetic measurements

Partial visual inspection of a warehouse fire-impaired building was conducted (see Fig. 1, Fig. 2, Fig. 3). Safety of a steel structure can be primarily assessed observing the post-fire behaviour of individual structural steel members. Tide [20] proposed to classify the structural members into three categories, as follows:

- Category 1: Structural members unaffected by fire. The members are visually linear, slight deformations barely detected by visual observation.
- Category 2: Significant deformations of structural members to be possibly heat-straightened.
- Category 3: Structural members severely deformed with large deformations.

Visual inspection shows that the insulated wall assembly with a sheet cladding and insulated membrane roofing on top of self-supporting sheets is partially damaged (see Fig.2) thus it should be removed and replaced. The main structural elements of I-section steel column and transoms (see Fig. 4) can be generally classified to the 1st category. However, small deformations of steel members were detected by visual observation, surfaces of selected structural members detect tightly adhering mill scales and surface erosion, denoting a high temperature exposure. In general, steel loses its mechanical properties in high temperatures (see e.g. [21]). Uniaxial tensile tests were further performed to assess specified post-fire parameters of I-section steel members.


**Fig. 4.** Visible fire impact on steel construction elements.

Geodetic measurement was aimed at displacement values and deformation of two selected I-section steel columns (denoted Sc1 and Sc2, see Fig. 3) affected visibly by fire. The measured displacements were analysed and compared to the allowable erection tolerances specified by EN 1090-2 [22]. Four types of erection tolerances were taken into

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account, the allowable deviations $\Delta_{max}$ are specified. Firstly, overall inclination of single-storey columns height *h* is specified as:

$$\Delta^1_{max} = \frac{\pm h}{300} = \frac{\pm 12000}{300} = \pm 40mm \tag{1}$$

Next, straightness of a single storey column, with regard to a straight line between the position points at top and bottom is:

$$\Delta^2_{max} = \frac{\pm h}{750} = \frac{\pm 12000}{750} = \pm 16mm \tag{2}$$

Functional manufacturing tolerances of web curvature, in the form of web height *b* deviation:

$$\begin{aligned} \Delta^3_{max} &= \frac{\pm b}{100} \text{but} \left|\Delta^3_{max}\right| \geq 5mm \\ \Delta^3_{max} &= \frac{\pm 400}{100} = \pm 4 \text{ but} \left|\Delta^3_{max}\right| \geq 5mm \end{aligned} \tag{3}$$

Next, functional manufacturing tolerances of web distortion and undulation in the form of deviation on gauge length $L$ = web height $b$ are:

$$\begin{aligned} \Delta^4_{max} &= \frac{\pm b}{100} \text{but} \left|\Delta^4_{max}\right| \geq 5mm \\ \Delta^4_{max} &= \frac{\pm 400}{100} = \pm 4 \text{ but} \left|\Delta^4_{max}\right| \geq 5mm \end{aligned} \tag{4}$$

Based on geodetic measurements and determined deviations (see Table 1**)** it was generally found that the I-beam fire-impaired steel columns detect the allowable deviations contained in specified tolerances. It should be noted, that only for Sc1 column located in cross axis F1/10, the web undulation deviation $\Delta^4$ insignificantly exceed standard manufacturing tolerances. It has been assumed that the indicated deviations are not significant, thus allow to apply standard procedures to determine the column's bearing capacity.

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## 4 Laboratory tests

The uniaxial tensile tests were aimed at estimating the post-fire mechanical properties of structural steel. The rectangular samples 50 mm by 250 mm were cut-out from the webs of I-beam steel columns Sc2 and Sc3. The Sc2 steel column sample was taken in the location of the largest visible fire damage, approximately 9 m above the floor level. The next sample was taken from the Sc3 column, the location without an observable fire damage, approximately 1 m above the floor level.

The uniaxial experimental tests apply the Zwick 250 mechanical testing machine. The experiments were performed up to steel specimen failure at room temperature. The constant loading rate was assumed of a 0.0005 1/s range, according to ISO 6892-1:2016-09 standard [23]. Table 2 includes the results of upper yield strength ($R_{eH}$) and tensile strength ($R_m$). Additionally the Brinell hardness test (HBW) was performed due to steel specimens according to ISO 6506-1 :2014-12 standard [24]. The Brinell hardness symbol HBW is preceded by the hardness value and supplemented by the test condition index. The Brinell hardness values of 111 and 150 were determined (see Table 2) applying a ball of 2.5 mm diameter and a test force 1.839 kN applied in a 10 s period.

The laboratory test results indicate lower strength and hardness parameters due to the Sc2 steel column samples. The yield strength value $R_{eH}$=235 MPa is accepted in numerical computations.

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**Table 1**. Comparison of geodetic measurement results.

| | Steel column Sc1 | | | | | Steel column Sc2 | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| H | $\Delta_A^1$ | $\Delta_A^3$ | $\Delta_B^1$ | $\Delta_B^3$ | $\Delta^4$ | $\Delta_A^1$ | $\Delta_A^3$ | $\Delta_B^1$ | $\Delta_B^3$ | $\Delta^4$ |
| [m] | [mm] | [mm] | [mm] | [mm] | [mm] | [mm] | [mm] | [mm] | [mm] | [mm] |
| 0.05 | 0 | - | 0 | - | 0 | 0 | - | 0 | - | 0 |
| 0.25 | 1 | 0.5 | 1 | 0.5 | 0 | 0 | 0 | 1 | 1 | -1 |
| 0.50 | 3 | 1 | 1 | 0 | 2 | 0 | 0 | 0 | 1 | 0 |
| 0.75 | 3 | 2 | 1 | 1 | 2 | 0 | 0.5 | 1 | 0.5 | -1 |
| 1.00 | -1 | 2.5 | -1 | 2 | 0 | 1 | 1.5 | 1 | 0.5 | 0 |
| 1.25 | 0 | 0.5 | 1 | 1 | -1 | 5 | 1.5 | 2 | 0.5 | 3 |
| 1.50 | 2 | 1 | 1 | 1.5 | 1 | 6 | 0.5 | 4 | 0 | 2 |
| 1.75 | 2 | 0.5 | 4 | 1.5 | -2 | 8 | 0 | 6 | 0.5 | 2 |
| 2.00 | 1 | 1 | 4 | 0.5 | -3 | 10 | 0.5 | 9 | 0.5 | 1 |
| 2.25 | 2 | 0.5 | 5 | 0 | -3 | 11 | 0.5 | 11 | 0.5 | 0 |
| 2.50 | 2 | 1 | 6 | 1.5 | -4 | 13 | 2 | 14 | 1.5 | -1 |
| 2.75 | 4 | 1.5 | 4 | 1.5 | 0 | 11 | 2.5 | 14 | 1 | -3 |
| 3.00 | 3 | 1 | 5 | 0.5 | -2 | 14 | 1 | 16 | 0 | -2 |
| 3.25 | 4 | 0 | 5 | 0 | -1 | 15 | 0 | 18 | 0 | -3 |
| 3.50 | 5 | 0.5 | 5 | 1 | 0 | 16 | 1 | 20 | 1 | -4 |
| 3.75 | 7 | 1.5 | 7 | 0.5 | 0 | 15 | 2 | 20 | 0 | -5 |
| 4.00 | 6 | 1 | 8 | 1 | -2 | 18 | 2.5 | 20 | 0.5 | -2 |
| 4.25 | 3 | 2 | 7 | 0.5 | -4 | 16 | 2.5 | 19 | 0.5 | -3 |
| 4.50 | 4 | 0.5 | 7 | 0.5 | -3 | 19 | 2.5 | 19 | 0.5 | 0 |
| 4.75 | 6 | 1.5 | 8 | 0.5 | -2 | 17 | 0.5 | 18 | 0 | -1 |
| 5.00 | 5 | 0 | 8 | 1 | -3 | 16 | 2 | 17 | 0 | -1 |
| 5.25 | 4 | 0.5 | 6 | 0.5 | -2 | 19 | 2.5 | 16 | 0.5 | 3 |
| 5.50 | 4 | 1 | 5 | 0 | -1 | 17 | 2 | 14 | 2 | 3 |
| 5.75 | 2 | 2 | 4 | 0.5 | -2 | 19 | 0.5 | 16 | 1 | 3 |
| 6.00 | 4 | 0.5 | 4 | 1 | 0 | 20 | 0 | 20 | 2 | 0 |
| 6.25 | 5 | 0.5 | 6 | 1.5 | -1 | 21 | 0.5 | 20 | 0.5 | 1 |
| 6.50 | 5 | 1 | 5 | 1.5 | 0 | 21 | 0 | 21 | 1.5 | 0 |
| 6.75 | 7 | 1 | 7 | 0 | 0 | 21 | 0.5 | 19 | 2.5 | 2 |
| 7.00 | 7 | 0 | 9 | 1 | -2 | 22 | 1 | 22 | 1.5 | 0 |
| 7.25 | 7 | 0.5 | 9 | 0 | -2 | 21 | 1.5 | 22 | 0.5 | -1 |
| 7.50 | 8 | 1 | 9 | 0 | -1 | 23 | 0.5 | 21 | 1.5 | 2 |
| 7.75 | 7 | 1 | 9 | 0.5 | -2 | 24 | 0 | 23 | 0 | 1 |
| 8.00 | 8 | 1 | 10 | 1 | -2 | 25 | 0.5 | 25 | 1 | 0 |
| 8.25 | 11 | 3.5 | 13 | 2.5 | -2 | 25 | 1.5 | 25 | 1 | 0 |
| 8.50 | 7 | 2 | 11 | 1.5 | -4 | 28 | 1.5 | 27 | 0.5 | 1 |
| 8.75 | 7 | 2 | 12 | 0.5 | -5 | 28 | 0 | 28 | 0.5 | 0 |
| 9.00 | 11 | 4 | 14 | 2 | -3 | 28 | 1 | 28 | 0.5 | 0 |
| 9.25 | 7 | 2 | 12 | 1 | -5 | 30 | 0 | 29 | 1 | 1 |
| 9.50 | 7 | 2 | 12 | 1 | -5 | 32 | 0 | 32 | 1.5 | 0 |
| 9.75 | 11 | 3.5 | 14 | 1.5 | -3 | 34 | 1 | 32 | 0 | 2 |
| 10.00 | 8 | 1.5 | 13 | 1.5 | -5 | 34 | 0.5 | 32 | 0 | 2 |
| 10.25 | 8 | 2 | 15 | 0.5 | -7 | 33 | 0.5 | 32 | 0 | 1 |
| 10.50 | 12 | 1.5 | 16 | 0.5 | -4 | 33 | 0 | 32 | 0 | 1 |
| 10.75 | 13 | 0 | 18 | 0.5 | -5 | 33 | 0.5 | 32 | 1 | 1 |
| 11.00 | 14 | 0.5 | 19 | 0 | -5 | 32 | 1 | 30 | 0.5 | 2 |
| 11.25 | 14 | 0 | 20 | 0.5 | -6 | 29 | 2.5 | 29 | 0.5 | 0 |
| 11.50 | 14 | - | 20 | - | -6 | 31 | - | 29 | - | 2 |

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**Table 2**. Laboratory tensile test results.

| Specimens | $R_{eH}$ [MPa] | $R_m$ [MPa] | HBW 2.5/187 |
|---|---|---|---|
| Sc2 | 249 | 396 | 111 |
| Sc3 | 395 | 524 | 150 |

## 5 Discussion and Conclusions

The results of tensile tests indicated a meaningful decrease of both yield and tensile strength due to a steel sample affected by fire conditions. Kosiorek [25] pointed out the necessity of minimum 10% mechanical parameter reduction of steel members affected by fire condition. Laboratory tests in the present investigations determined reduction of about 30% of yield strength. Nevertheless, Maślak [7] emphasized that a higher low carbon steel temperature during fire conditions is associated with recrystallization, thus should not be achieved. Further microstructural changes of steel are affected by conditions and intensity of the material cooling process, acting upon the strength characteristics after cooling down in the post-fire condition. Another solutions to minimalize the negative influence of short duration of fire condition is the application of FRS (Fire Resistant Steels) to manufacture the high fire risk members. Grabarz and Adamczyk [26] showed that the FRS of a yield strength exceeding 500 MPa at ambient temperature shows considerably higher resistance to a typically 20-minute high-temperature heat impact till 600°C, than structural steel species of wide application today.

In order to check the load-bearing capacity of steel structural elements numerical calculations were performed by means of the ARSA software package (Autodesk Robot Structural Analysis). The calculations were carried out due to a separate system consisting of a single column and a purlin lattice separated from the structure. Numerical calculations were carried out in two variants. The first dimensioning variant was adjusted to the existing state. Taking into account the results of the laboratory tensile test it was assumed that the yield strength of all steel members affected by fire was reduced to S235 class steel. In the case of the I-beam steel column dimensioning it was assumed that the existing transoms at the height of 3 and 6 meters protect the steel column from buckling in the plane of the gable wall, but do not fully prevent it from lateral torsional buckling. The joint action of numerical analysis and dimensioning detected an insufficient load-carrying capacity of the I-beam steel column.

In order to increase the capacity reduced buckling length of the column in the gable wall plane (from 6 to 3 m) was taken into account introducing additional transom and additional support points to fully protect the column against lateral torsional buckling. Due to this computational variant sufficient load-bearing capacity was confirmed. Finally, the assembly of steel strengthening (additional transom) was added and insulated wall assembly with a sheet cladding and insulated membrane roofing on top of self-supporting sheets was removed and replaced before the warehouse building partially damaged by fire caused by external source (fire of lorries close to the building) was being fully operational.

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