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Assessment of damages on a RC building after a big fire

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Abstract. This paper presents a case study about the damages on the structural elements of a cast in place reinforced concrete (RC) building after a big fire which was able to be controlled after six hours. The fire broke off at the 2nd basement floor of the building, which has five basements, one ground, and two normal floors. As a result of intensely stocked ignitable materials, it spread out to the all of the upstairs. In visual inspection, most of the typical fire damages were observed (such as spalling, net-like cracks, crumbled plasters, bared or visible reinforcement). Also, failures of the 2nd basement columns were encountered. It has been concluded that the severity failures of the columns at the 2nd basement caused utterly deformation of the building, which is responsible for the massive damages on the beam-column connections. All of the observed damages were categorized related to the types and presented separated regarding the floors. Besides to the visual inspection, the numerical analysis was run to verify the observed damaged on the building for columns, beams, and the connection regions. It is concluded from the study that several parameters such as duration of the fire, level of the temperature influence on the damages to the RC building. Also, it is highlighted by the study that if the damaged building is considered on the overall structural system, it is not able to satisfy the minimum service requirements neither gravity loads nor earthquake conditions.

Keywords: beam-column connection damage; elevated temperature; fire damage; RC building; structural element

1. Introduction

Fire is one of the major disasters that affect the structures rarely. However, it may induce unpredictable severe damages (Folic *et al.* 2002, Garlock *et al.* 2012, Ha *et al.* 2016). A fire spread to the whole structure can cause unexpected damages to the structural elements. Mainly, the building type is crucially important for the type and the level of damage to the building because of the fire. In reinforced concrete (RC) building (implemented with appropriate reinforcement details) the damages are allowable through the redistribution of loads from damaged to undamaged sections/elements (Yaqub *et al.* 2011). However, for steel (or composite) structures the fire may cause degradation of the structural materials, thermal buckling and yield of the material under small stresses (Singer *et al.* 2002). For instance, the Plasco Building was a landmark 17 storeys high rise steel and concrete building in Tehran (Iran), collapsed as a result of a fire and caused the death of dozens of people, recently (see Fig. 1) (Reuters 2017).

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Fig. 1 The photographs of the Plasco Building while the fire and after its collapse (Reuters 2017)

RC buildings are affected due to fire, especially elevated temperatures. The concrete's properties are more complicated than the most materials by the reason that it is a constitution of different components. In case of a big fire, due to the concrete spalling and direct exposure of the flames to the reinforcements may cause overall structural damages (Naik 2008, Ma *et al.* 2015)

In the literature, the behavior of ordinary concrete which is not fireproof or containing fine-grained component than cement grains with elevated temperature has been sorted in different ways but following the similar stages. The free water inside the concrete starts to evaporate with increasing temperature. Above 150℃, the water bound the hydrated calcium silicate has been released and reaches its peak about 270℃. Inertial stress development has occurred in this initial stage. Dehydration of the matrix and relative thermal expansions of the aggregates in the concrete lead to microcracks in the material after 300℃. This phenomenon leads to decrease in the strength and elasticity modulus of the material. If the concrete is exposed less than 300℃, it can absorb the moisture from the air and recover. However, after shaping the microcracks, the strength loss is not recoverable. Therefore, it is recommended to remove the material that exposed above 300℃. Between 400-600℃, the calcium hydroxide decomposes into calcium oxide and water, and then water evaporates, and shrinkage occurs in the matrix. The most severe decrease in the strength of the concrete occurs in this range. After the fire, the calcium oxide reacts with the cooling water and the moisture in the air that forms new calcium hydroxide. This reaction causes the expansion of the cracks in the concrete and further decrease (about 20%) in the strength of the material. The hydrated calcium silicate decomposes after 600℃, and the concrete can be crumbled to gravel by the finger after 800℃.Then, feldspar melts and remaining minerals of the cement turns into a glass phase above 1150℃ (Hertz 2005).

The strength of the concrete that is fully hydrated may increase within the first 100-200℃. However, this increase should not be considered in the design stage because of the dependency of the age of the concrete and condition of the material (Hertz 2004). Ultimately, only a small part of the original compressive strength (9-20%) (Chan *et al.* 1999). The deterioration of the compressive strength of the concrete with temperature can be clarified with physical and chemical changes in the concrete with elevated temperature. The physical and chemical changes can be sorted by changes in water content, hydration products, pore structure, microstructure and spalling. The tendency of these changes in the concrete are affected from many parameters: compression strength of the concrete, moisture content, density of the concrete, thermo-physical properties of concrete, external load, pre-stress, temperature gradient, temperature distribution on the structural elements, dimensions of the elements, reinforcement ratio, existing of fibers, aggregate and type of the cement additives, etc. More detailed information about these factors could be obtained from the literature (Lin *et al.* 1996, Chan *et al.* 1999, Hertz 2003, Tanyildizi and Coskun 2008, Ozawa *et al.* 2012, Kizilkanat *et al.* 2013, Ma *et al.* 2015).

Under ordinary circumstances, the reinforcement of a building is not subjected to temperatures higher than 50℃ (Chiang and Tsai 2003). However, during a fire, the structural elements may be exposed to higher than 1000℃. Even though, there is a concrete cover surrounding the reinforcement for a building that is constructed appropriately, as a result of spalling the reinforcements may be directly exposed to the fire. That may cause dramatically lessen the load-bearing capacity of the elements (Anderberg 1997). The placement of reinforcement close to the surface may cause damages or collapse of the structures, but the enhancing the concrete cover is an option however it supplies somewhat a limited improvement (Shi *et al.* 2004). The mechanical properties of the reinforcement start changing around the 500℃, especially the tensile strength of the reinforcement decreases crucially at the higher temperatures than 800℃. The proportional loss in the strength of the reinforcement to the initial strength varies for differently graded reinforcements: the damage on the lower graded reinforcement is less than higher the graded reinforcement (Topcu and Karakurt 2008). Post-fire strengths of hot rolled bars can be considered to regain their initial strength; however, for cold worked bars, the strength reduces to its original strength of the steel that it had before it was cold worked (Hertz 2004). More detailed information about the behavior of hot rolled and cold worked reinforcement under elevated temperatures for different strain levels can be obtained from Hertz (2004). Besides the loss in the strength of the reinforcement, the bond strength between the reinforcement and concrete depends on the temperature level and exposure time (Chiang and Tsai 2003). The anchorage capacity is the minimum of the splitting capacity and bond capacity. While the splitting failure depends on the geometry of the cross-section, the transverse reinforcement, and the quality of the concrete, the bond strength depends on the shape of the deformed reinforcement and the properties of the concrete. Under normal conditions, the splitting capacity is used as the anchorage capacity, but for the fire damaged concrete the bond capacity is the capacity of the anchorage (Hertz 2004). The assumption of perfect bond condition between concrete and reinforcement is indicated as un-conservative (Khalaf *et al.* 2016). More detailed information about for normal and high compressive strength concretes with different types of aggregates and embedment lengths the bond behavior under elevated temperatures can be obtained from Aslani and Samali (2013).

According to the literature review, besides the studies these investigated the effects of the fire in view of material science, there are other studies which considered the elemental or system response to a fire. There are several studies related to the assessment of structural damages of buildings after fires (Kmet *et al.* 2016). Also, some nonlinear computational models to predict the behavior of the structural systems subjected to fire (Choi *et al.* 2012). Various RC slabs and beams were performed under fire to get the fire effects on the elements (Yu *et al.* 2012). As declared in the literature, a big fire may cause damages on the structural system of the building. This study addresses and states about the elements and system damages of a RC building after a fire. In the content of this study, initially, a literature review is given above. Secondly, fire safety codes were briefly mentioned regarding temperature, and material sides with and effects on the structures investigated safety codes are considered due to the importance of the subject. Then, the damages observed on the RC building after the fire are categorized and presented for all structural elements. Lastly, the conclusions obtained from the study are presented.

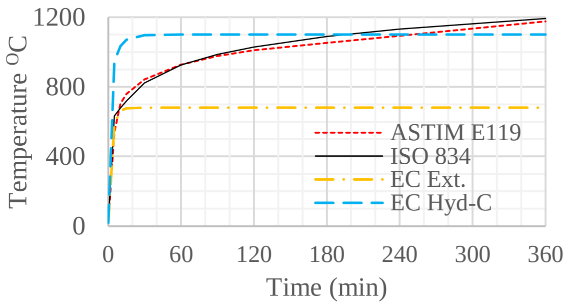


Fig. 2 Typically used standard temperature-time curves

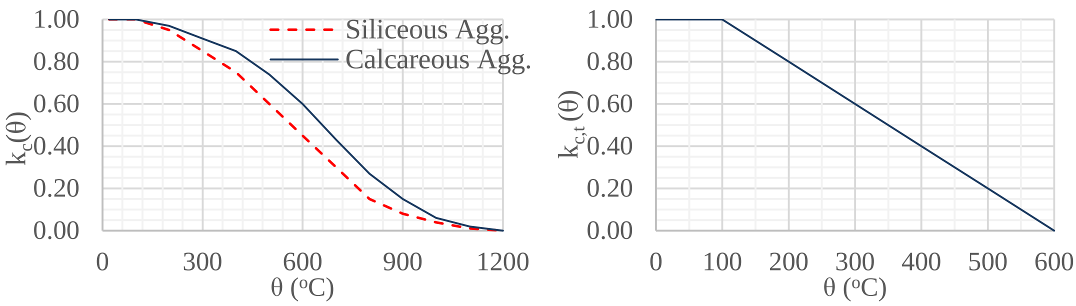


Fig. 3 The coefficients ($k_c(\theta)$) allowing for decrease of characteristic strength (a) and the coefficient ($k_{c,t}(\theta)$) allowing for decrease of tensile strength of concrete (b) (CEN 2004)

2. The existing codes about fire safety of structures

In codes, many but similar standard fire exposure curves that are used to calculate the increase in the temperature with time are stated. The E119 (ASTM 2016) and ISO 834 (ISO 1999) time-temperature curves are probably the most common furnace exposures used in fire resistance testing. These furnace exposures are utilized to evaluate the fire resistance of structural elements on the building. ASTM E119 is primarily used in North America whereas ISO 834 is more common (Lattimer 2016). The variation of the temperature (℃) according to ISO 834 is calculated by the following equation

$$T = 345 \log_{10}(8t + 1) + T_0 e \tag{1}$$

where, t is time (in minutes), and T_0 is ambient temperature (℃). Additionally, there are two more curves that are stated in EN 1991-1-2 (CEN 2002) which characterize the hydrocarbon and exterior fire effects. The comparison of these typically used standard fire temperature-time curves is demonstrated in Fig. 2. It is evident that the temperatures reach to high levels even though at the beginning times of the fire. This sharp increase indicates the importance of immediate extinguishment. In EN 1992-1-2 (CEN 2004), there is a method to calculate the fire resistance period depending on the physical characteristics of the fire section. The more detailed information about these could be obtained through the EN 1991-1-2 (CEN 2002) and more information about the methods can be accessed on Lennon *et al.* (2007).

Particularly in EN 1992-1-2 (CEN 2004), the decrease of characteristic strength of concrete is prescribed in details. The reduction in characteristic compressive strength of normal weight

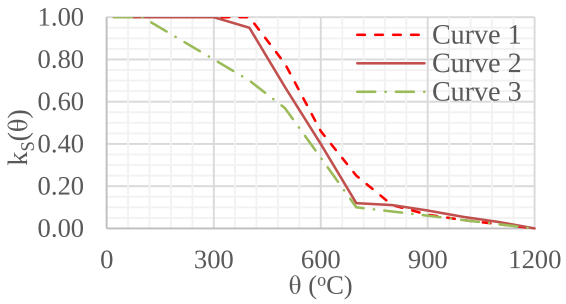


Fig. 4 The coefficient ($k_s(\theta)$) allowing for decrease of characteristics strength of tension and compression reinforcement (CEN 2004)

concrete with siliceous and calcareous aggregates (containing at least 80% calcareous aggregates in weight) as a function of temperature θ is given in Fig. 3(a). The tensile strength of the concrete starts decreasing linearly above 100℃ until 600℃, after 600℃ the tension strength of the concrete is assumed to be zero (Fig. 3(b)).

In the same manner, the decreasing of the coefficient $k_s(\theta)$, which stands for the decrease of the characteristics strength of the reinforcement with temperature, is presented in Fig. 4. In the figure, curve 1 and 2 are hot rolled, and cold worked reinforcements which effective yield strength ($k_{y,\theta}$) obtained with 2% strain was given. It may be applicable for tension reinforcement in beams and slabs. Considering the yield strength occurs at 0.2% strain level, the reduction of yield strength (or the changing of the coefficient that allows the decrease of strength) of hot-rolled reinforcement is given in the Curve 3 in Fig. 4. The trend of the curves changes according to the strain level, and how the reinforcements are processed.

3. General information about the building and the fire

3.1 The building information

The building is used as an office block that composed of 550 workshops of all sizes in various sectors. The building, designed in 1993, consists of five basements, one ground, and two normal floors. The height of each storey is 4.0 m, in turn, the total height of the building is 32.0 m. Due to the inclined topography of the site, and some basements are below the ground level on one side while at the other side these basements are above the ground level. In here, it should be declared that the basements are named differently from general definition stated in codes (e.g., IBC (ICC 2011)), however, to keep the same phrase with the project the word is not revised. The schematic side view of the building is given in Fig. 5(a) with ground levels. In the figure, "F" and "B" letters stand for indicating the "floor" and "basement," respectively. The building composes of reinforced concrete (RC) frame with the arrangement of columns with 6.40 m span length, and shear walls are located in the staircase area and building edges. All of the columns are in the circular cross-section. The diameters of the columns that are 70 cm are same in the entire building. The concrete cover of the columns and shears walls are 4 cm and 3 cm, respectively. The dimensions of the beams are 60 cm / 30 cm, and the concrete cover of the beams are 3 cm. The slab thickness is 17 cm, and the surface of the slabs are covered with the paving that thickness is 2 cm (except the top

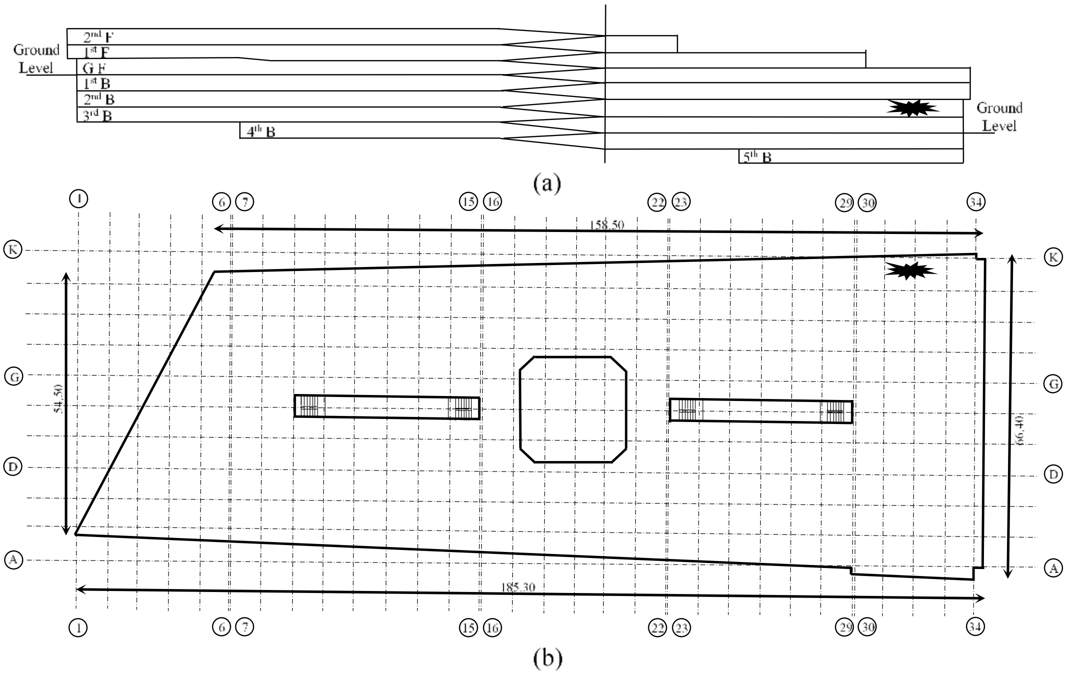


Fig. 5 Schematic side view (a) and plan view (b) of the building

floor). All surfaces of the elements were not covered with any plaster.

The plan dimensions of the building are 158.5 m×66.4 m. For the top four floors (2nd F, 1st F, GF and 1st B), the plan dimensions are extended about 1.5 m on each side. The plan view of the 2nd basement is given in Fig. 5(b). The building consists of 5 blocks which are between axes 1 and 6 (Block 1), 7-15 (Block 2), 16-22 (Block 3) and 23-29 (Block 4) and 30-34 (Block 5). In the plan view (see Fig. 5(b)), the location of the expansion joints between 6-7, 15-16, 22-23 and 29-30 axes are shown. Moreover, a lighting clearance with vehicle path space is placed at the center of the building, and on either side of the building, four stairway cases with two lighting clearances were located. Excluding elongated sides of the stairway clearances, all sides of them are open-air.

According to the project, used concrete and reinforcement strength classes are C18 and S420, respectively. In general view, it can be said that during the design of the building the fire effects were considered neither in view of the element sizes nor the selected material types. Also, there were not taken any essential precaution to prevent from spread of the fire or prevent structural elements from fire such as fire barriers or encasement on the elements. The separation materials of the adjacent offices were separated with thin brick or glass/metal walls.

3.2 The information about the fire

On October 22nd, 2017 ante meridiem, the fire broke out from the chimney on the external wall of the factory that is on the 29-32 and F-K axes (see Fig. 5). In consequence of the factory storage at the axes was full of thermos granule raw materials. Therefore the fire was able to spread quickly. Also, there was the most severely damaged region in the building. By way of the ignitable materials on the hallways, the fire spread to the Block 5 and half of Block 4 in this storey. Soon

(a) (b)

Fig. 6 The photographs of the building during the fire (NTV 2017, DNA 2017)

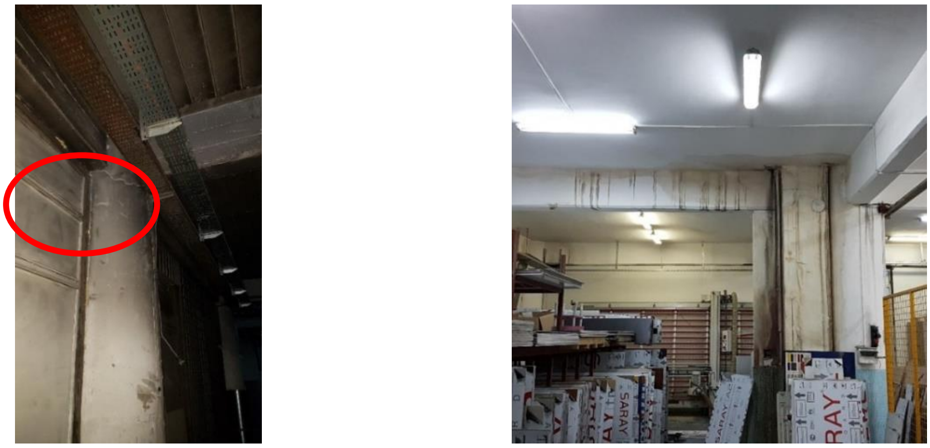
(a) (b)

(c) (d)

Fig. 7 External view of the building after the fire

after, through the vertical openings between storeys, the fire spread to the upper floors. Meanwhile, damaged windows on the elongated sides with already open-air sides of the clearances made the transmission of the fire quickly. The fire was able to be taken under control after 6 hours of operation. As a result of the fire, 30 offices were burned out completely, 97 office damaged partially and 52 vehicles wholly buried or damaged partially (IMM 2016). The ventilation chimney marked within the circle can be seen at the left of Fig. 6(a) and the fume caused by the fire can be seen in Fig. 6(b).



(a) (b)

Fig. 8 Beam-column connection cracks (a) and the local leakage of the cooling water on 3rd basement (b)

4. Fire damages on the RC building

All structural elements (columns, connections, beams, slabs of the RC building) were investigated visually after the fire to assess the damage level of the building. The damages were categorized according to the element types, floors and the level of damage. Although the most elements of Block 5 was damaged, only some part of Block 4 was damaged. The fired regions could be observed from the external view of the building (Figs. 7(a)-(c)). The spalling of the exterior elements were observed (Fig. 7(d)).

In view of the damages on the structural elements, the 2nd basement where the fire was broke out was the most damaged floor. Other floors were damaged depending on the spreading of fire due to the ignitability of the wares which existed. As expected, the storeys below this floor were not damaged. It should be stated that a few of the beam-column connection regions very slightly cracked (Fig. 8(a)) and especially at the connections regions the leakage trace of the cooling water could be observed at some regions (Fig. 8(b)) at the 3rd basement.

The loss of the elements caused by the fire was fundamentally similar. On the other hand, the exposure times and the characteristics of the elements such as the type of the element, loading conditions and surface area, were the main parameters that assign the level and type of the damage.

4.1 The assessment of the damages on columns

The durability of the columns is vital for a structure. Thus the investigation of the damages on the columns is crucial to the assessment of a building. It is observed that the damages on the columns mainly depend on the temperature, exposure time, and the loading condition. The columns on the 2nd basement where the fire was spread out were the most damaged columns in the building. This area was exposed to the fire longer than any other the storey in the building. Moreover, according to the reports and observations, it can be said that the duration of the fire at this storey is approximately equal to the duration of the fire (6 hours).

The absence of plaster or any covering on the columns the damage level of the columns were

(e) (f) (g) (h) (i)

Fig. 9 Failures of the several columns at the 2nd basement

Fig. 10 Cracks of several columns and spalling on the 1st basement and 1st floor

higher than expectations. For 2nd basement, some of the loss was damaged partially with inclined cracks (Figs. 9(b), (c), (g) and (i)). However, on the multiple columns lost their bearing capacities completely. It is noteworthy to state that the columns damaged heavily in the 2nd basement are

Fig. 11 The displacement of Block 5 relative to Block 4 from the roof of the building

Fig. 12 The displacement of Block 5 relative to Block 4 from inside of the building (shot on 1st floor)

Fig. 13 Shear wall beam cracks on 2nd basement

between the 30-34 and H-K axes. As a result of complete crush on the columns, the longitudinal reinforcements were buckled, and bond strength was lost between reinforcements and concrete, stirrups were crushed and deformed (Figs. 9(a), (d), (e), (f), and (h)).

In other storeys, only partially deep cracks and spalling were observed. Moreover, the longitudinal and stirrups were bared but not damaged or lost contact with the core concrete (Fig. 10). In consequence of crush of the multiple columns in 2nd basement, the whole structure was deformed, and a relative movement was occurred between Block 5 to Block 4. This phenomenon could be seen on the expansion joint between Block 4 and 5 (Figs. 11 and 12). Informatively, there was not any relative displacement on the expansion joint between Block 3 and Block 4.

Fig. 14 Connection cracks on the 1st basement and ground floor

4.2 The assessment of the damages on connections

Connection areas such as column-beam, column-floor, beam-floor or shear wall-beam are important for structural safety. A relative movement in this area causes the load concentration. In observed building, the relative movement is reasoned increasing of failure on columns in the second basement. Besides, the effect of the thermal expansion during the fire may generate extra loads on the connections. The damages were generally with cracks those widths are not more than half of a centimeter, and depth varied depending on the element types, storey and location of the elements.

The connection of the shear wall with the beam an obvious horizontal crack was determined (Figs. 13 and 16). For column beam connections single or multiple horizontal cracks were observed (Figs. 14-16). Simultaneously the elevated temperature and the relative movement of the building some of the column beam connections were cracked and local spalling were observed (see Fig. 16).

Fig. 15 Connection cracks on the 1st and 2nd floors

Fig. 16 Spalling of beams and visible reinforcements on the ground floor

4.3 The assessment of the damages on beams

The bending capacity of a beam primarily varies by the strength and the elasticity modulus of the materials, size & shape of the beam, and reinforcement characteristics. It was observed that cross-section area of the some of the beams was decreased as a result of spalling of concrete cover. Longitudinal and stirrup were bared and direct flame effected to these reinforcements on some beams (Fig. 16). Moreover, on several beams are vertically deformed and cracked with the loss of the cross-section and the material strength reduction (Fig. 17).

Fig. 17 Deformed beams at Ground and 1st floors

Fig. 18 Spalling of slabs and visible reinforcements at the 2nd basement

4.4 The assessment of the damages on slabs

In some part of the building, especially on the 2nd basement, slabs are damaged with net-like cracks, cracks at the bottom surface, loss in bond strength between reinforcement and concrete, bared or visible reinforcements (Figs. 18-20). The physical characteristics of the fire and bared reinforcement, some of the slabs were deformed slightly. For damaged floors, it was hard to determine slab deformations and cracks on the slabs because of the burnt stock ruins. Luckily, on the upper surface of the top floor, multiple deep cracks were observed with varying slopes of the cracked slabs (Fig. 20).

4.5 Other observed damages

Sometimes non-structural damages may give important clues to estimate the level of the fire. In this study, to predict the level of the elevated temperature during the fire, the damages of the automobiles at 2nd ground floor was able to be examined. Because of the elevated temperatures, the aluminum wheels of cars melted (Fig. 21), considering these vehicles were at roof parking area

Fig. 19 Net-like cracks, spalling and bared reinforcements at 1st basement slabs

Fig. 20 Spalling and bared reinforcement at 1st floor and deep cracks on 2nd floor slabs

(open air area) and the melting heat of the alloy the temperature is about 650°C (Ashby 2006). This can be interpreted that the temperature inside the building was higher than 650°C. Regarding the strength loss of the concrete and the reinforcement with the elevated temperatures in EN 1992-1-2 (CEN 2004), the strength of the structural materials of the building was decreased critically (see Figs. 3-4).

The natural convective flow occurs when there is a temperature difference between inside and outside of the building. Air enters the building through the openings on the exterior walls in the lower floors. Then the air rises to upstairs through the vertical shafts, and get out of the building though on the exterior walls or any openings in the building in the upper floors. The phenomena of the indoor temperature are higher than the outside, the flow upward of warm air through the vertical shafts is commonly named as “stack effect” (Yung 2008). Before the fire, the longitudinal

Fig. 21 Burnt vehicles that some of the aluminum wheels trims melted in the parking area on the roof

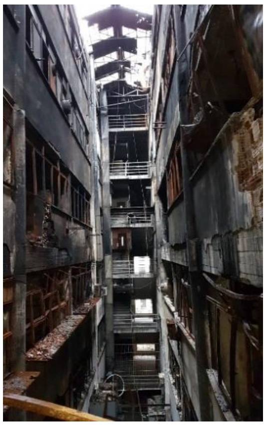


Fig. 22 The view of the stairway case with lighting clearance

Fig. 23 A deformed steel roof rail at 1st floor

sides of the clearance (see Fig. 5) was closed with windows, but with the fire, all of the windows were splintered, and the clearance behaved as a vertical shaft (Fig. 22). Thus, the fire could easily transmit to the upstairs, was not able to keep down and took longer than it should be.

Besides, the severity of the elevated temperatures of the fire may be observed from the deformed a steel roof rail that is on the 1st floor (Fig. 23). If the melting point of the steel is considered, the temperature during the fire can be estimated.

5. Numerical study

A numerical study was performed to simulate the time-independent behavior of a circular column considering beams and the connection under elevated temperatures which was observed with the damages on the site.

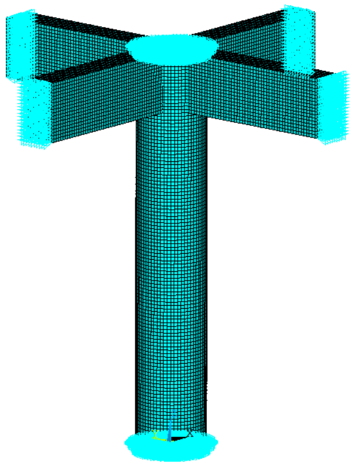


Fig. 24 The meshed model with boundary conditions

5.1 Properties of the model

The validity of an analysis performed with finite element method highly depends on the assumptions of the model. Therefore, the geometrical and the material properties of the numerical model were determined from the properties of the investigated building. The diameter of the column was considered as 0.70 m; height was 4 m as in the building. The mechanical properties of the concrete selected based on the C18 concrete strength class, as the modulus of elasticity is 27,500 MPa, density is 2500 kg/m^3. The thermal coefficient of expansion for the concrete considered as 1x10^{-5}/℃ for concrete (Johnson and Parsons 1944). The primary purpose of this numerical study is to reveal the locations and the level of the strain/stress developments in the axially loaded column under elevated temperatures. Hence the material nonlinearity was ignored.

The components of the model are one column and four brace beams on the top level of the column. The boundary conditions for the beams was restrained against to the translation along the longitudinal directions. Each end of the column was fixed to represent the inter-storey column (Fig. 24). At the connection, the column and the beams are fully bonded to each other with the contact elements. All of the components were modeled with the SOLID185 element. The SOLID185 is a 3D element with eight-nodes which have translational freedoms in three directions (x, y, and z). For the connections between the components were established with the CONTA174 and TARGE170 (contact) elements in ANSYS. CONTA174 is an element used to represent contact and sliding between 3D "target" and a deformable surface defined by this element. The element has the same geometric characteristics with the solid or shell element face which it is connected. TARGE170 is used to represent various 3D target surfaces for the associated contact elements. This target surface is discretized by a set of target segment elements and paired with its associated contact surface. More detailed information about the used element can be obtained from ANSYS Mechanical APDL Theory Reference (ANSYS 2017). The model consists of 99296 finite elements which size are 3.5 cm mesh size.

According to the fire report obtained from the local fire officials, the level of the temperature reached more than 750℃. Hence, the level of the thermal loading applied on the surfaces was assigned as 750℃. It should be stated that the thermal loading was applied on the surfaces of the

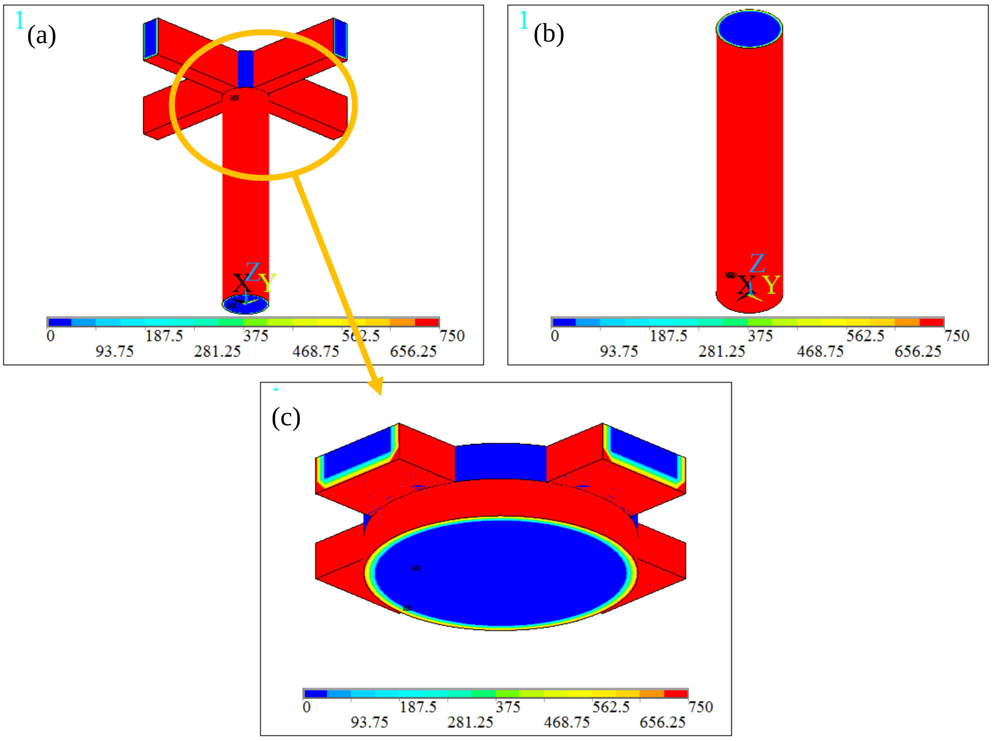


Fig. 25 The body temperatures (in °C) of the model (a), column (b) and the joint section (c)

components, but considering the fire exposed peripheral area ratio of the column section at the beam level, thermal loading was not applied on this section.

5.2 Validation of the numerical model with observations

In the performed thermal analysis with finite element method provided damages similarly with the observed damages on the column, beam, and connections in the fire-damaged building. The variation of the body temperatures in the components and the connection are given in Fig. 25. The results show that the effects of the fire are diminished about 4 cm depth from the surface, however, because of the spalling of concrete cover and cracks that develop on the surface of the elements may cause the reaching of the thermal effects more in-depth than this value.

The strain is one of the parameters that can be used to determine the location of the damages. Therefore, in this study, the strains caused by the thermal loading were considered to evaluate the validity of the numerical model by comparing the results with the investigated building. It is accepted that the failure strain of the concrete is about 0.0035 for normal strength concrete (Reynolds *et al.* 2008). In this regard, the thermal strains exceed the limit value for failure in most of the model (Fig. 26(a)). Especially on the surfaces, the level of strains is about 0.0035 range which will cause the spalling of the concrete cover and plaster. Besides, in some specific locations, such as the connection, the level of strains are more than 0.0045 which declare the mechanical failure will take place in this region (Fig. 26(b)). All of the obtained results from this numerical model is coherent with the observed damages on the columns, beams and connection region of

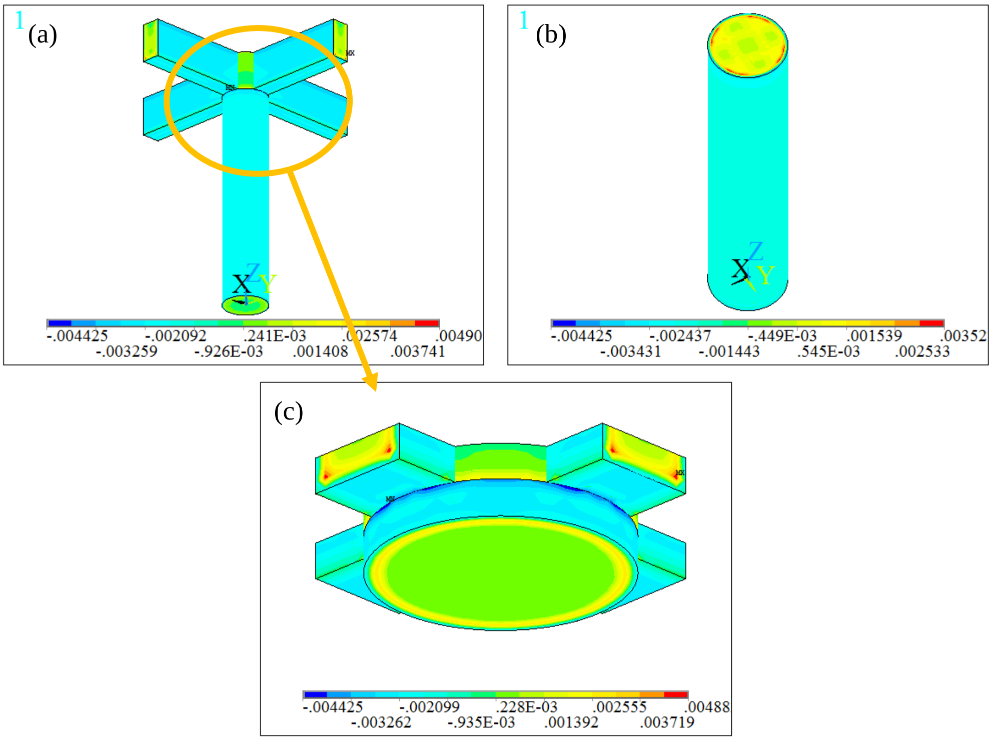


Fig. 26 The principal strains of the model (a), column (b) and the joint section (c)

Fig. 27 The principal strains of all the model, column and the joint section

them. In Fig. 27, the most commonly observed damages which are the spalling of the concrete cover, cracks on the connection zone and spalling on the edges of the beams. The locations of the cracks on the columns and the connection region in Fig. 27 are similar to the locations of the strains obtained from the numerical analysis that is over the failure limit of the concrete. Also, near the edge of the beams, the strains are more than 0.0045 which is already much more than the failure limit of the concrete (Fig. 26(c)). In consequence of this strain concentration, spalling may be one of the most expected damages in the beams (see Fig. 27).

6. Conclusions

In this study, the visual assessment of the fire damaged reinforced concrete building elements was performed. The fire was broke out from the inter-storey (2nd basement) and spread to the upper floors. It was able to be taken under control after six hours. Because of the long duration of the fire, severe structural damages were observed on the elements. The following conclusions were obtained from the study are summarized.

- The columns are one of the most critical elements of a structure. Therefore they should be coated with fire protection material. In this study, it was observed that the failure of the multiple columns on the 2nd basement caused an overall structural deformation.
- Because of the relative movement and thermal expansion of the materials with temperatures, more loads were generated on the column beams connections and shear wall beam connections. As a result of this, some cracks were observed in these regions.
- Cross-sectional loss of the beams with spalling, loss in the bonding strength and decrease in the material strengths caused a decrease of the bearing capacity of the beams. Therefore, bared reinforcement was observed most of the beams. Unfortunately, in some beams, there was visible bending cracks and vertical deformations on the beams.
- It was obtained from the visual assessment the thickness of the concrete cover is an essential parameter on the damages of the elements. The concrete cover of the slabs are less than other elements, so due to the spalling most of the reinforcement of the slabs were bared. Further, in some slabs, the concrete behind the reinforcement was fallen off. An important conclusion is that if the damaged building is considered in view of the overall structural system, it is not able to satisfy the minimum service requirements neither gravity loads nor earthquake conditions.
- The performed numerical analysis provided damages similarly to the columns, beams and the connection regions. This coherency validates the reliability of the numerical analyses for thermal loading conditions with appropriate assumptions.
- It is clear that there are various parameters such as duration of fire and temperature level have a significant influence on the damages to the RC building due to fire. However, the vitally important thing that specifies the damage level of the building is the followed precautions which are stated in the codes (e.g., IBC (ICC 2011)). By applying these precautions, from the beginning the fire may not be, the spreading of the fire can be limited, and the structural elements can be recovered from irrecoverable damages.

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