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CASE STUDY OF A FIRE DAMAGE ASSESSMENT OF A TWO-STORY STRUCTURE

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ABSTRACT

Nowadays and along with urbanization, the frequency of accidents resulting from fire attacks are high. This study deals with assessing the extent of fire damage to a two-story building occupied by a textile factory. The slabs are composed of pretensioned 1.2m width, 9.0m length slender panels, supported by prestressed concrete beams. The remaining structural elements are of reinforced concrete. The study throws light upon fire investigation tools utilized to evaluate the post fire integrity of the structure. It also describes fire investigation techniques that track the visual evidence in the fire-damaged structure. The study also includes estimating the peak temperature and duration of heat. The assessment involves both field and laboratory work to determine the severity of the damage. The work involves non-destructive testing throughout the parts of the building that were still in place and appear to be stable, followed by destructive testing at some selected locations. This paper presents an overview of how to conduct a structural evaluation of a fire damaged structure.

Keywords: fire damage assessment, structural assessment, fire endurance.

INTRODUCTION

Prescriptive codes do not provide the engineer much insight into the performance of structures subject to fire conditions, nor do they aid in the assessment and repair of fire damaged structures (Shneider, 1990).

One example illustrating the need for information on the post-fire assessment and repair of fire damaged structures was the dispute over the structural integrity of Meridian Plaza in Philadelphia. The 38 storey office building suffered severe fire damage between the 22nd and 30th floors in 1991, and remained empty for over five years while its safety was assessed, only to eventually be demolished (Gilvary and Dexter 1997).

Ensuring the structural integrity of buildings under fire exposure, leads ever more to understanding of deterioration phenomena. At the structural scale, elevated temperatures induce restrained thermal dilation due to kinematic restraints. At the material scale, thermal load induces strong micro-structural changes that alter the concrete behavior (W.Nechnech et Meftah, 2001). According to Hertz (2004), initially when the temperature of concrete rises, the free water inside the concrete evaporates. This evaporation causes a pressure build-up within the concrete. When the temperature reaches about 150° C the water that is chemically bound to the hydrated calcium silicates, is released. At a temperature of 300°C the aggregates expand and the cement paste starts to shrink. When the temperature reaches a value between 400°C and 600 ^{o}C, the calcium hydroxide $Ca(OH)_2$ breaks down into calcium oxide CaO and water H_2O, creating addition water vapour and more pressure. This pressure may be too high and may result in cracking and spalling of concrete.

Knaack *et al* (2011) and other researchers proposed temperature dependent material behavior for concrete. Also Elgazouli (2009) and others proposed temperature dependent material behavior for steel reinforcement, paving the way towards understanding the performance of reinforced concrete elements subjected to thermal loads.

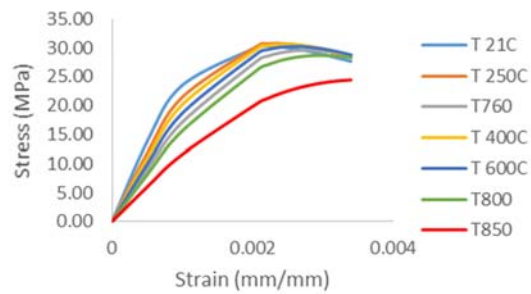

Figure-1. Temperature dependent concrete behavior (Knaack).

Chaing *et al*. (2003) reported that the heat associated with the fire vaporizes trapped concrete pore water. The lack of continuous voids for pressure relief creates internal tensile stresses that are relieved by cracks and spalls extending to the surface. The elevated heat would provoke cracking, concrete spalling, and eventually undermine the structural integrity of the flexural elements. Bostron (2008) indicated that fire spalling is a complex phenomenon. Light spalling takes place at sharp corners, and it is of minor importance for the load bearing capacity and the integration of the structure. The pop-corn spalling is the continuous scaling off when 5-10 mm thick pieces shoot from the surface. The continuous spalling may jeopardize the structure. The explosive spalling is very severe, whereby a large part of the structure explodes momentarily, resulting in a sudden loss of the load bearing capacity. Woznaik *et al* (2008) reported that when Prestress concrete slabs are exposed to fire, they show features different from traditional reinforced concrete structures. One may observe specific phenomena leading to the decrease of the fire resistance of the elements, such

as anchorage failure or shear. According to Woznaik *et al* (2008), there is a paucity of data on the effect of elevated temperatures on cold-drawn prestressing steel, both in terms of post-fire residual mechanical properties and high-temperature stress relaxation, which can lead to significant prestress loss, both during and after a fire. Significant losses in effective pre-stress and moment capacity occurred even with the appropriate amount of concrete cover, after one hour of exposure to a standard fire ASTM (2001 d).

Gales (2009) stated that two significant effects of fire can cause a reduction of prestress force in a prestressing tendon, namely a gradual, recoverable reduction of prestressing force resulting from restrained thermal expansion, and a more severe, irrecoverable reduction resulting from creep or relaxation under stress at high temperatures. A complex interaction exists between prestress levels and a temperature history for a tendon that undergoes a localized heating and a cooling cycle.

Zdenek *et al* (2013) indicated that an increase in temperature strongly accelerates the flow of metals and thus also the prestressing steel relaxation.

According to ACI (2018), the nominal flexural strength of pre-tensioned concrete panel slabs is:

$$M_n = A_{PS} f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_y (d - \frac{a}{2}) + A'_s f_y (\frac{a}{2} - d') \qquad (1)$$

$$a = \frac{A_{ps} f_{ps} + A_s f_y - A'_s f_y}{0.85 f'_c b}$$

$$f_{ps} = f_{pu}\left(1 - \frac{\gamma_p}{\beta_1}\left[\rho_p \frac{f_{pu}}{f'_c} + \frac{d}{d_p}(\rho - \rho')\right]\right)$$

Where:

f_{pu} : Ultimate tensile strength of prestress reinforcement

f_{ps} : Stress in bonded prestress reinforcement at nominal flexural strength.

γ_p : a coefficient that reflects the influence of prestressing reinforcement on the value of f_{ps}

A_{PS} : Total area of strands

d_p : Strands effective depth.

A_s : Total area of tension mild steel reinforcement.

A'_s : Total area of compression mild steel reinforcement.

f_y : mild steel yield strength.

d : Tension steel effective depth.

d' : Compression steel effective depth.

ρ : Mild steel reinforcement ratio

ρ_p : prestress steel reinforcement ratio

Equation 1 implies that the flexural strength of pretensioned slabs is affected by the followings:

- The ultimate tensile strength of the strands.
- The prestress loss.
- The mild steel reinforcement yield strength.
- The concrete compressive strength.

According to Hertz (2004), the ultimate tensile strengths of various prestressing steels have been shown to decrease significantly at between 400ºC and 500ºC. Abrams and Cruz (1961) performed series of tests to investigate the elevated temperature relaxation of cold-drawn prestressing steel. They concluded that a gradual loss of prestress takes place up to approximately 315ºC, followed by a more drastic decrease above this temperature. This drastic change in rate of prestress loss is attributed to creep related relaxation being activated above 315ºC.

The value of modulus of elasticity of mild steel deteriorates significantly over a range of temperatures.

The deterioration rate of mild steel at elevated temperatures is often expressed as a ratio of E_T/E_o. (Wong, 2011), where

$$\frac{E_T}{E_o} = 1.0 + \frac{T}{2000 \ln\left[\frac{T}{1100}\right]} \qquad for \;\; 0^oC < T \le 600^oC$$

$$\frac{E_T}{E_o} = \frac{690\,(1 - \frac{T}{1000})}{T - 53.5} \qquad for \;\; 600^oC < T \le 1000^oC \qquad (2)$$

Where E_T *is steel modulus of elasticity at temperature T*
E_o *is steel modulus of elasticity at ambient temperature.*

Phan and carino (2000) indicated that concrete exposed to temperatures up to 300ºC would experience little loss in strength. According to Khoury (2000), concrete exposed to temperatures greater than 300ºC often exhibits a distinctive deep pink color caused by a reaction in ferrous salts in sand and aggregate.

CASE STUDY

The subject site is Nahleh textile factory. It is located in Dulail region about 100 km. east of Amman city, the capital of Jordan. The building is composed of two floors. The slab units are composed of Precast Pretension concrete panels of 1.2m width, 9.2m length, and 0.2m thickness. The beams supporting the slab units are of Pre-stressed concrete. The columns and foundations are of ordinary reinforced concrete. The building total area is 9500 m^2. Two expansion joints split the building into three zones as shown in Figure-2.

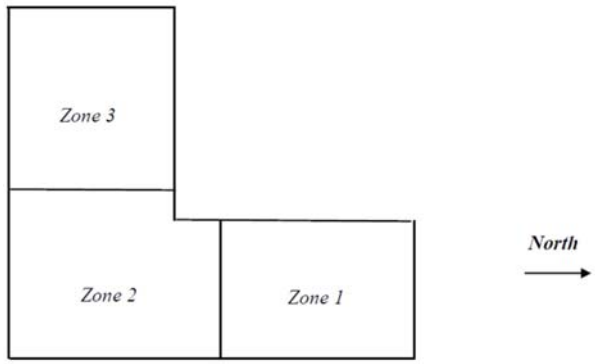

Figure-2. Schematic plan of the building.

The fire erupted in the ground floor within the northern sector of the building. Through the open windows, the fire spread towards the first floor that contained textile materials. The fire severity within the 1st floor, boosted by the wind breeze, exceeded that of the ground floor. About forty-five minutes later, the fire began to flash over, resulting in the collapse of the 1st floor slabs in the northern sector, falling on the ground floor slabs. Eventually resulting in the collapse of 50% of the ground slab units underneath as shown in Figure-3. The fire spread within the whole building, it lasted for about 7 hours, before it was eventually extinguished.

Preliminary investigation involved visual inspection of the whole structure to identify the damaged areas, through spotting the collapsed structural elements, concrete spalling, and through tracking the cracks, scale off and physical and color changes.

In order to assess the fire severity in different parts of the building, the study involved inspection of the affected surfaces and the debris, examining the color, the state, and the condition of the extracted samples. On the other hand, the peak temperature and the duration of heat at the damaged areas were estimated by means of temperature indicator tables.

The existence of two expansion joints that divide the whole structure into three separate buildings within three separate zones, as well as site inspection for the post fire building condition, lead to classifying the whole structure into three zones as shown in Figure-2.

Zone 1: Collapse of slab panels of the ground floor and the first floor slabs. The beams and columns remained in position, but they suffered from considerable cracking, in addition to spalling of concrete at the corbels and at the column edges as illustrated in Figure-3.

Zone 2: Collapse of the slab panels of the first floor within the remaining portion of the northern sector. They fell above the ground floor slab panels. Nonetheless, the ground floor slab panels were able to carry the fallen slab panels as illustrated in Figure-5.

Zone 3: This part was exposed to fire but maintained stability and nothing collapsed, as illustrated in Figure-6.

Figure-3. Collapse of slab panels of 1st and ground floors in zone 1.

Figure-4. Cracks in column and spalling of corbel concrete.

Figure-5. Collapse of slab panels of 1st floor on ground floor slabs in zone 2.

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Figure-6. Post fire inside view of the 1st floor within.

METHODOLOGY

A detailed investigation was carried out to identify the collapsed structural elements, to spot the spalling concrete, and to track the extension and depth of cracks and scale off in the structural elements that remained in place. Nondestructive testing was carried out for the whole structure, followed by destructive testing at selected points, to assess post fire concrete residual strength.

Extensive research has been carried out on the performance of OPC concrete on fire (Lane *et al*, 2008). When heated to temperatures above 300ºC, this type of concrete usually changes color to pink. When heated further, the color profile changes again at around 500ºC-600ºC to a grey-buff colors. These color changes are a result of iron slats on the aggregate particles. Construction materials and debris.

A good assessment of temperature values within the fire-affected areas may be concluded from the physical effects of temperature on concrete, construction materials and debris. Tables 1, 2 give an overview of temperature indicators in a typical building. In this investigation, the considered temperature indicators within the three zones of the building were aluminum mullions, glass panels, copper cables, and concrete. Nondestructive evaluation of the concrete is usually performed, using Schmidt hammer and ultrasonic pulse velocity testers. Megahid *et al*. (1987) reported that Schmidt-hammer measures concrete hardness, it provides excellent means for determining relative strengths of concrete in different parts of the same structure.

The Pulse velocity tester is a pulse velocity meter with a digital readout, transducers, and water soluble jelly used as the acoustic coupling agent. Lower measured velocities would indicate distress, defects, or cracks in the concrete.

Table-1. Physical effects of temperature on concrete, after Yuzer et al (2004).

Temperature	Color change	Changes in physical appearance and benchmark temperatures	Concrete condition
0-290ºC	**None**	**Unaffected**	**Unaffected**
290-590 ºC	Pink to red	Surface crazing: 290ºC Deep cracking: 550ºC Pop outs over chert or quarts aggregate:575ºC	Sound but strength significantly reduced.
590-950 ºC	Whitish grey	Spalling, exposing not more than 25% of reinforced bar surface: 800 ºC Powdered light colored dehydrated paste: 575ºC	Weak and friable
950 ºC	Buff	Extensive spalling	Weak and friable

Table-2. Temperature Indicators, after Slough (1990).

	Typical Examples	Conditions	Approximate Temperature, Cº
Polystrene	**Thin wall food containers**	**Collapse**	**120**
Cellulose	Wood, paper, cotton	Darkens	200-300
Lead	Plumbing	Melts, sharp edges rounded	300-500
Aluminum	Fixtures, Castings	Softens	400
Glass	Glazing, Bottles	Softens	200-300
		Floors	200-300
		Flowing easily	850
Brass	Locks, Taps	Melts	800-1000
Copper	Wiring	Melts	1100

RESULTS AND DISUSSIONS

Assessment of fire severity

In zone 1 (Figure-3), where all slab panels of the two floors collapsed, the debris had whitish grey color. The supporting beams and columns did not collapse but had whitish grey color. In zone 2, the debris of the fallen slab panels had also whitish grey colors. The bottom slab surface in the ground floor slab had whitish grey color. In zones 2, 3 shards of glass remained in some window frames. Some of these shards had melted and were forming drips. In zone 3, the concrete surface in the 1st floor had a black color all over, as illustrated in Figure-6. This is attributed to the burning of the textiles inside the building when fire erupted. The concrete slab surface color in the ground floor was pinkish.

Comparing the colors of concrete surfaces with the color indicators and the fact that concrete surfaces suffered from spalling and cracking, it was estimated that the temperature of the concrete surfaces of slabs and columns in zone 1 and zone 2 was within the range of 550 to 650°C.

Figure-7. Post fire inside view of the 1st floor within zone 3

Adopting the aforementioned approach for the estimated temperatures for zone 1, and zone 2, it was concluded that the temperature in zone 3 was within 300 to 400 ^{o}C.

Figure-8. Cracks in a reinforced concrete column.

Assessment of structural damage

The signs on the severity of damage of the inspected building in Zones 1 and 2 where concrete surfaces, for slabs columns and corbels, were exposed to estimated temperatures within 550 ^{o}C to 650°C, are as follows:

a) The collapse of all slab units for both two floors in zone 1, (Figure-3) and of many 1st floor slab units in zone 2, (Figure-5). The collapse of such a large amount of slab units would be attributed to the followings:

- Concrete cracking and spalling when the temperatures of concrete surfaces in slabs and columns exceeded 550°C, thus loss of concrete protection bottom layer which resulted in directly exposing prestressing tendons to high temperatures and in drastic change in rate of prestress loss due to creep related relaxation being activated above 315°C.
- Substantial decrease in the ultimate tensile strength of prestressing strands at between 400°C and 500°C
- Deterioration of mild steel modulus of elasticity (E_T) to about $0.5E_o$ as estimated according to equation 2.
- Significant decrease in the concrete compressive strength.

b) The large amount of spalling of concrete in columns and column corbels, resulting in exposing the reinforcement, (Figure-4).
c) The severity of cracks. The crack extension and the width and depth of cracks.
d) The change in color of concrete surfaces to whitish grey.
e) The temperature was estimated to be within 550 to 650 ^{o}C. The concrete was weak and friable.

Therefore, it was concluded that the remaining parts of the building, in zones 1 and 2, were structurally unsafe. They ought to be demolished.

On the other hand, in zone 3 the temperature was estimated to have reached 300 to 400 ^{o}C. Apart from the change in color of the concrete surfaces as mentioned earlier, and other minor signs and clues, the structure in this zone seemed to be stable and safe. Thus, it was decided to carry out further field and laboratory testing to evaluate the residual strength of structural concrete in zone 3, prior to any conclusive assessment.

Nondestructive evaluation of concrete strength for the structure in zone 3

In this investigation, both Schmidt hammer and Pulse ultrasonic digital testers (Pundit) were utilized. The test points were the 2m by 2m grid intersections for the slabs, and at 2m distances for beams and columns. The Schmidt hammer testing was carried out in accordance with ASTM C805. The Pulse Ultrasonic Nondestructive Digitial Tester was carried out in accordance with ASTM C597-83. The results are shown in Figures 8 and 9 for the

1st and ground floors slabs respectively. Figures 10 and 11 show the results for the 1st and ground floors respectively.

Figure-9. Core taken from a concrete column in zone.

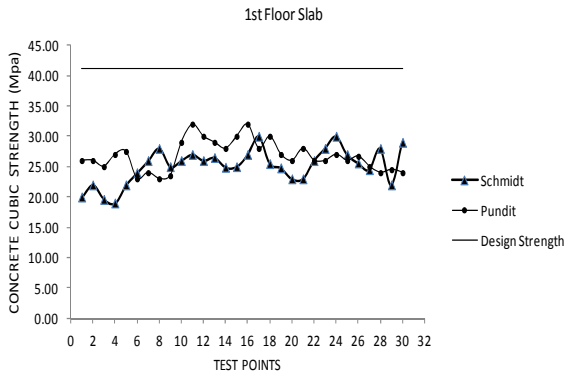

Figure-10. Comparison between Nondestructive cubic strength and Design strength at 1st Floor slab.

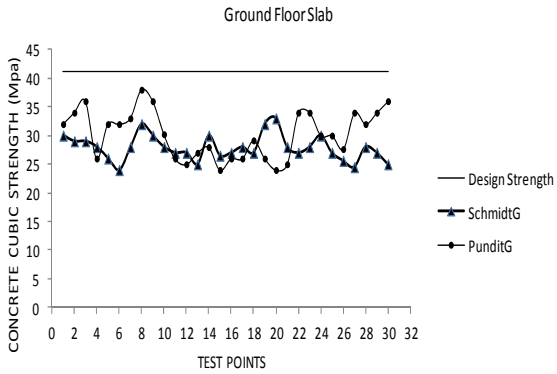

Figure-11. Comparison between nondestructive cubic strength and Design Strength at Ground Floor Slab.

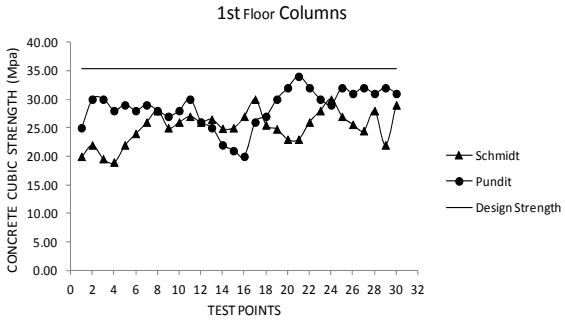

Figure-12. Comparison between Nondestructive cubic strength and Design Strength for 1st Floor Columns.

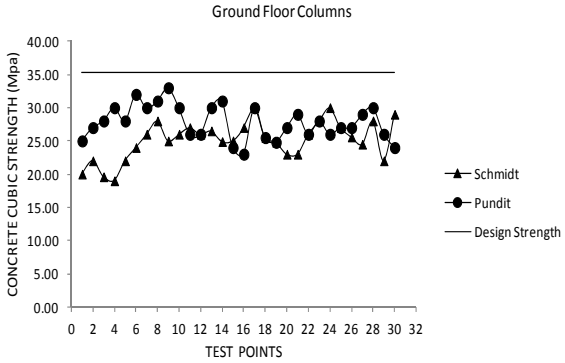

Figure-13. Comparison between Nondestructive cubic strength and Design Strength for Ground Floor Columns.

Laboratory analysis of concrete cores for the structure in zone 3

Concrete core samples were taken for determination of compressive strength as shown in Figure-9. Such cores were taken where the concrete surface was still intact and contained the outermost layer of discolored concrete. Concrete cores taken from beams and columns were tested for compressive strength in accordance with ASTM C42 "Standard test method for obtaining and testing drilled cores". Concrete core strength results are listed in Table-3.

Table-3. Concrete core strength.

Structural member	MPa
1st Floor Slab	20
Ground Floor Slab	24
1st Floor Columns	18
2nd Floor Columns	20

In general they are in agreement with the results obtained from nondestructive testing, illustrated in (Figures 10, 11, 12, 13). Concrete core strength results are close to about 50% of the concrete characteristic strength.

CONCLUSIONS

The study presents an overview of how to conduct a structural evaluation of a fire damaged structure.

Being able to assess the material strength and prestress loss is crucial to estimate the flexural strength of the slender pre-tensioned slab panels. Nonetheless the interactions between thermal expansions, stress level in strands, affected by pre-stress losses due to creep related relaxation at elevated temperatures, are not well known. Accordingly, the residual flexural strength of the slab panels could not be determined.

Structural design parameters for the building project were taken from the structural design drawings, and are presented in Tables 4, 5

Table-4. Structural Concrete Design Parameters.

Concrete design parameters as given in design drawings	
Structural Element	**MPa**
Lean Concrete	14.7
Columns and footings	35.3
Concrete slabs Beams	41.2 41.2

Tracing the extension and depth of cracks and spalling of concrete, resulted in identifying the locations of the mostly distressed concrete. The cracks and spalls were created by the pressure of fire vaporized trapped concrete pore water, resulting in internal tensile stresses, that were ultimately relieved by cracks and spalls that extended to the surface. Ignoring to comply with the minimum concrete cover in the slender slab panels deprived the prestressing strands from the proper fire protection. Localized heating at high temperature resulted in significant losses of the effective prestressing forces leading to considerable reduction in moment capacity at mid span for many slab panels. Eventually resulting in collapse of slab panels, the supporting beams and the columns remained in position.

Utilizing both pulse ultrasonic non-destructive and Schmidt hammer testers enhanced the credibility of the non-destructive testing that was carried out throughout the structure in zone 3. Obtained results from destructive testing, carried out in selected locations within the structure in zone 3, were in agreement with nondestructive testing results. Both results indicated that post fire concrete residual strength in slab panels was as low as 50% of the characteristic concrete strength in several locations. The slab panels flexural strength was also impaired by the decrease in strands stresses due to creep related relaxation at elevated temperature.

Based on such comprehensive assessment, the study concluded that the part of the building in zone 3, as well as the remaining parts in zone 2, and zone 1 are to be demolished

Table-5. Steel Design Parameters.

Reinforcement parameters as given in design drawings	MPa	Grade
Main reinforcement, up to 18 mm diameter	412	60
Main reinforcement 20 mm diameter	515	75
Secondary Reinforcement	412	60
Wire Mesh	412	60
Prestressing low relaxation strands	1850	270

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