Pisco Med Publishing

Evaluation of Residual Strength of RC Columns in Fire-Damaged Buildings: Case Study

Vail Karakale[1], Ramadan E. Suleiman[2], Fathi M. Layas[2]

[1] Civil Engineering Department, Faculty of Engineering, Marmara University, Goztepe Kampusu, Kadikoy, Istanbul, 34722, Turkey

[2] Civil Eng. Dept., Faculty of Eng., University of Benghazi

Abstract: Prediction of residual strength of structural members in a RC fire-damaged building is an important step in taking a decision about restoration or demolition of the building. In this paper a finite element model was developed to evaluate the distribution of temperature within the cross-section of a RC column during a fire. Then the results were used to estimate the residual strength of RC columns in a fire damaged building in Libya. The building was used as a court yard for the public and the fire visibly damaged some of its columns. Material tests were conducted in situ and in the laboratory in order to evaluate the material strength after fire. Finite element analysis results and Euro code formulations were used in the prediction of material properties during fire .The predicted properties shows good agreement with material test results. Furthermore analysis results shows that up to 60 min of fire duration the column do not lose considerable amount of its strength, however at 120 min fire duration it will lose about 35 percent of its axial load and moment carrying capacities.

Keywords: Fire-Damaged Buildings; RC Column; Restoration; Residual Strength

1. Introduction

Restoration of fire damaged reinforced concrete buildings is of great interest both for the owners and insurance company in terms of reducing the capital cost and restarting the business due to earlier reoccupation of the buildings. Columns are the primary structural elements that transfer the loads of a building vertically to the foundation. Several studies and experimental works in the literatures shows that when RC columns are exposed to fire, the material properties of concrete and the reinforcing steel change as a result of the temperature increases[1-3,5,9]. The decreases in yield strength and modulus of elasticity reduce the overall strength of the column. In this paper an evaluation of the residual strength of RC columns in a fire damaged –building in Libya is presented. The building was used as a central court building of Al-Baida city in Libya. The fire occurred in the right wing of the ground floor of the building, and as reported by witnesses it lasted approximately for an equivalent continuous time 2 to 3 hours, involving a lot of paper documents, furniture and other materials. More reliable information about temperatures is obtained during on site surveys. All furniture, windows, doors, infrastructure are severely affected by fire. Most of the structural elements are grey to black in color. Examinations of rubble show all floor tiles are dismantled and burned, window glass is shattered but not melted. The electric wires are burned however the copper in electric wires had not been softened. These observations indicate that the temperature is in the range of 400 to 600 $_{o}$C[6,7]. This is also assured by the grey to black color of the structural concrete elements surfaces. Concrete color provides a broad, general guide of temperatures[8]. The fire caused

doi: 10.18282/adr.v1i1.557

readily visible surface damage, as shown in **Figure 1**. The residual strength evaluation process used in this research involves calculating the distribution of temperature within the concrete cross-section, then calculating the axial load and the moment capacities of the member using residual strength versus temperature relationships suggested by Euro code. Furthermore material tests were conducted in situ and in the laboratory in order to evaluate the material strength of the RC members after fire and the results were compared with the Euro code relationships. The following sections show the details of building description, material tests, heat transfer analysis, and residual load carrying capacities calculations.

Figure 1; General view of the building after fire**.**

2. Architectural plans and structural system

The building consists of ground floor and upper 3 typical floors, built in the early1970s; each story is 3 m clear height and 1460 m^2 plan area. The structural framing made of conventional cast in- situ reinforced concrete as it is shown in **Figure 2.** The framing system consists of grid of columns mainly of40 cm square size, as shown **Figure** (**2**). The columns are reinforced with 8 ϕ18 longitudinal mild steel plain bars and ϕ 8 lateral reinforcement spaced at 15 cm center to center. The maximum spacing between columns is 6m center to center in short direction and 4.5m spacing in long direction. The slabs of the building floors are hollow block type with total thickness 25 cm comprising (approximately) 5 cm topping and 20 cm thickness hollow block. The ribs are 15 cm thickness and spaced at 55 cm center to center with 2 ϕ18 mm bars to resist negative and positive bending moments. The ribs are mainly constructed continuously in 3 spans 6m, 2m and 6m long respectively except at entrance lobbies where it is 6m long single spans. The main beams supporting the ribs are embedded beams with 25 cm thickness and 10 mild steel plain bars combination of ϕ 18 and ϕ 20 top and bottom provided with ϕ 8 double stirrups spaced at15 cm center to center. Conventional local type hollow block of 20 cm thickness is used for partitions.

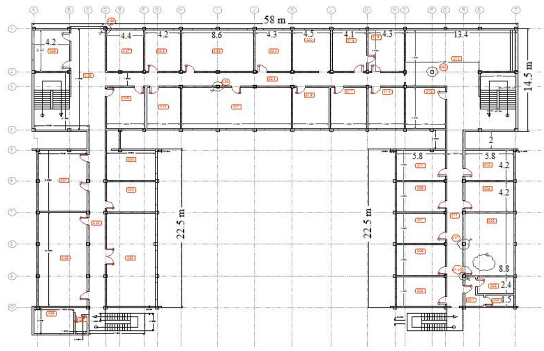


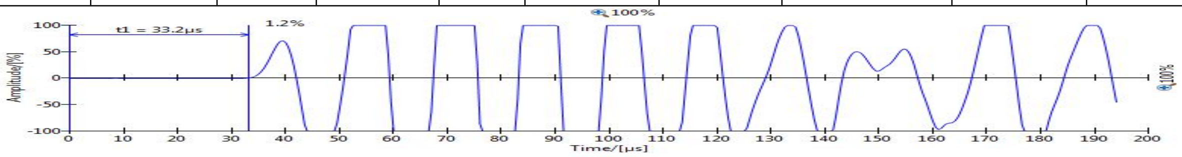
Figure 2; Plan of the first floor**.**

3. Material tests

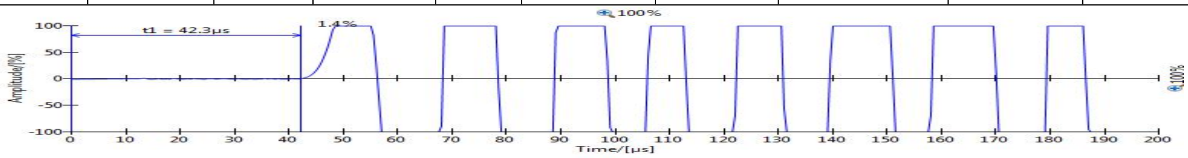
Tests were conducted in situ and in the laboratory in order to determine concrete material properties. The non destructive rebound hammer and pulse velocity tests were conducted in situ according to the recommendation given in ACI 228-R10 and ASTM C 805[10]. In the rebound hammer test all affected structural members are divided into grids with designation numbers and labels then the hammer test is performed and the rebound number is recorded. An average of at least ten rebound readings are taken for one test, see **Figure** (**3c**). In the pulse velocity test method the estimation of the compressive strength of concrete is based on measuring the time it takes for a pulse of vibration energy to travel through a concrete member. The tests were performed according to ASTM C 597 standard[11]. Some test results are shown in **Table 1** and **Figure 3.** 20 core samples have been extracted from the different structural elements affected by fire. Locations of the concrete core samples are established based on results of non destructive tests and visual inspection of the structure. Strength of concrete cores taken from the structural elements is determined in accordance with ASTM C 42[12]. For core length-to-diameter ratios different than 2.0, the appropriate strength correction factors given in ASTM C 42 were applied[12]. Two types of concrete cores are extracted, full core length diameter 70 mm, and smaller size of diameter 25 mm. The small size cores are used to depict the concrete strength along the structural member from the near surface to fire and the far surface to estimate the extent of fire damage to thecross section. The equivalent specified concrete strength is calculated according to ACI 562-12[13]. Typical results are shown in **Table 2**. Test results shows that the average compressive strength of fire unaffected concrete is about 17 MPa. However the most fire affected structural element (i.e Column C007 of the ground floor) has an average compressive strength of 9.6 MPa. Furthermore tests on core samples taken from different depths of a fire affected column indicate that the concrete strength near the column surface is about 11.8 MPa , and at the center of the column's cross section is about 15.3MPa.

Figure 3; Some photos during testing a) core sampling, b) ultra sonic test, c) rebound hammer**.**

Element No	**Element Location**	**Meas. Type**	**Velocity [m/s]**	**Time [µs]**	**Distance [m]**	**Ambient Temp. [°C]**	**Compressive Strength [MPa]**	**Amplitude [V]**	**Frequency [kHz]**
S042	(8/9-A/G)	Direct	3765	33.2	0.1250	11	10.8	500	200



S005	(7/9-R/T)	Direct	3979	42.3	0.1683	14	12.6	500	200



C030	(2-P)	Direct	4094	97.7	0.4000	12	15.0	500.000	200.00

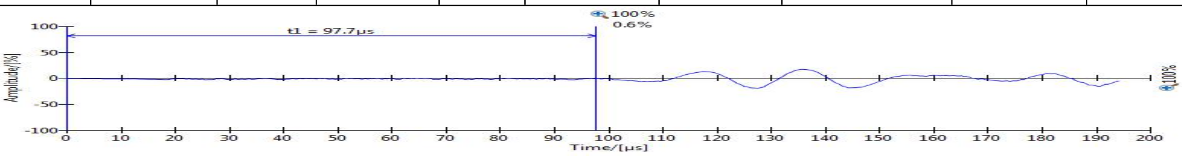
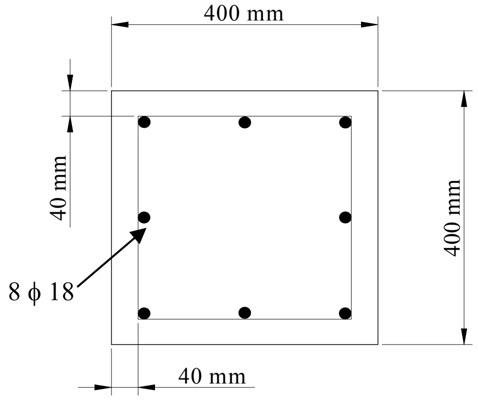


Table 2. Typical result of compressive strength testing

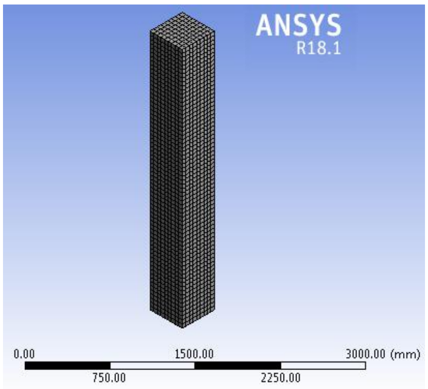
Element No.	Height	Dim.	Ratio	Volume	Wight	Density	Equivalent Comp. Strength	Ultrasonic velocity
	mm	mm		cm^3	grm	grm/cm^3	Mpa	
C192	169	67.8	2.49	610.4	1345.5	2.204	11.03	3773.00
S111	165	67.8	2.43	595.9	1407.4	2.362	13.25	4014.00
S142	121	67.8	1.78	437.0	979.7	2.242	16.51	4187.00

4. Transient thermal analysis

All the columns in the case study building have the same dimensions and steel reinforcement details as it is shown in **Figure 4 a**. During fire some of these columns were exposed to fire from four sides and the rest were exposed to fire from three or two sides. The finite element analysis program ANSYS was used to carry out a 3-D transient thermal analysis on one of the building's columns[14]. The concrete column was modeled using an 8-node linear heat transfer brick elements as shown in Figure. 4b. Modeling of reinforcement bars was neglected and it is assumed that reinforcement bar temperature is equal to the surrounding concrete temperature. The developed FE model takes into account the nonlinear temperature dependant of concrete material properties such as specific heat, and thermal conductivity. The variation of concrete thermal conductivity with temperature specified by EN 1992-1-2:2004 code was used in the model (see **Figure 5**)[4]. The concrete density is assumed to be constant and equal to $2300kg/m^3$. As a worst case the model was exposed to fire from four sides using ISO834 standard fire curve. From analysis results the distribution of temperature in the column section at different time intervals was obtained and presented in **Figure 6.** To illustrate the thermal predictions from the model, the temperature variation is plotted as a function of fire exposure time at various locations of the column cross section in **Figure 7**. The temperature at various depths of concrete, as well as in reinforcement bars locations, increases with fire exposure time. As expected, the predicted temperature decreases with increasing distance from the fire exposed side.



(a)



(b)

Figure 4; a) Column section details, b) FE model**.**

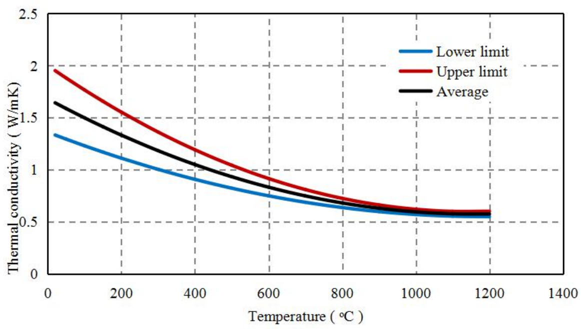


Figure 5; Variation of concrete conductivity with temperature (EN 1992 standard)**.**

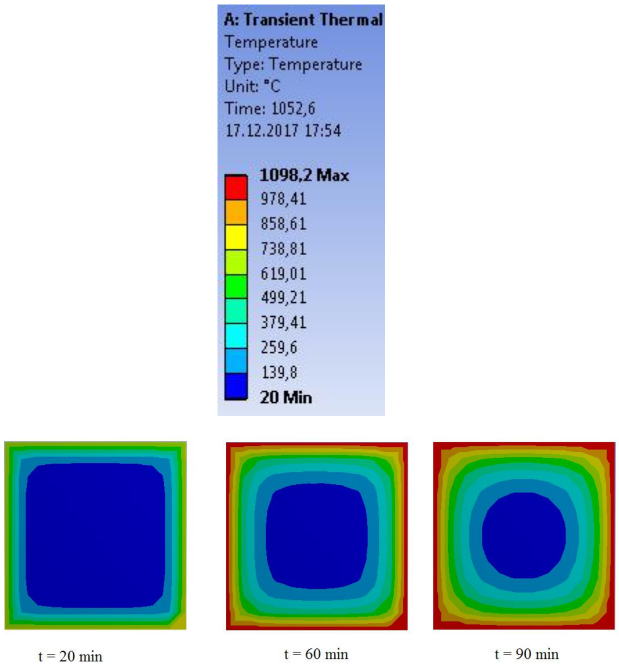


Figure 6; Temperature distribution.

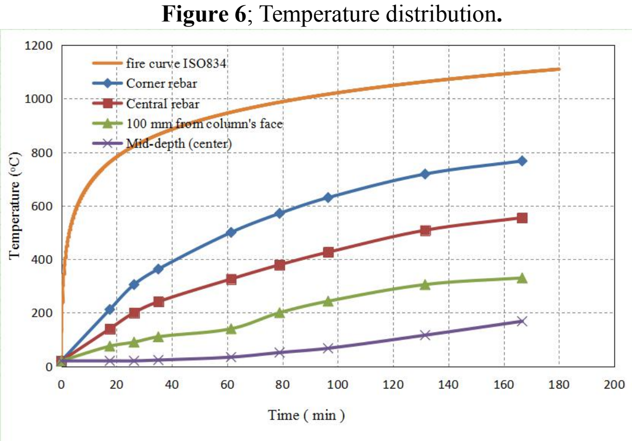
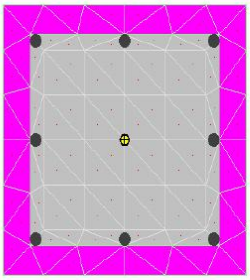


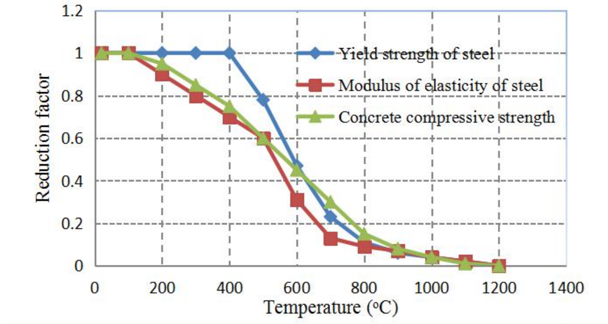
Figure 7; Temperature as a function of time at different locations of cross section.

5. Residual strength prediction

The cross sectional temperatures generated from thermal analysis are used as input to the strength analysis (i.e. generation of moment-curvature relationship and axial force-moment interaction diagram). In the strength analysis a finite element model was developed using XTRACT software as shown in **Figure 8a**[14]. In the analysis it is assumed that plane sections before bending remain plane after bending and there is no bond-slip between steel reinforcement and concrete. The fire unaffected concrete compressive strength of 17 MPa which is obtained from material tests was used as initial value. The initial value of reinforcement bars yield strength was taken equal to 420 MPa. As the fire duration increase the temperature will increase inside the column cross section causing reduction in the compressive strength of each concrete layer of the XTRACT model and also cause a reduction in the reinforcement bars stress-strain relationship. The reduction in material properties of concrete and reinforcement steel due to elevated temperatures was taken into account using reduction factors proposed by the Eurocode (see **Figure 8b**)[4]. The proposed reduction factors are in good agreement with the experimental material tests results. This can be shown from the results in **Figure 7** and **Figure 8**b as follows: it is shown from **Figure 7** that at 120 min fire duration and at a depth of 100 mm from the face of the column, the concrete temperature was equal to 300ºC; and at this temperature according to **Figure 8**b the reduction factor of concrete strength is about 0.8; that is the concrete strength becomes (17 x 0.8 = 13.8 MPa) and this value are in good agreement with the experimental results given in **Table 1** and **Table 2**. The axial force-moment capacity diagrams and moment–curvature curves for the column at various time durations were obtained and presented in **Figure 9** and **Figure 10** respectively. From the Figureures it is clearly that both moment and axial load carrying capacities of the column decreases with increasing time of fire exposure. This is due to the deterioration in the material strength and stiffness as a result of increased temperatures in concrete and steel. Furthermore analysis results shows that up to 60 min of fire duration the column do not lose considerable amount of its strength, however at 120 min fire duration it will lose about 35 percent of its axial load and moment carrying capacities.



(a)



(b)

Figure 8; a) XTRACT FE model ,b) Strength reduction factor according to Euro code[4].

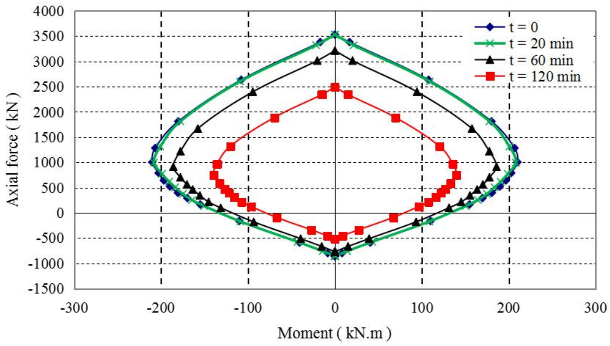


Figure 9; Axial force –moment interaction diagrams at various times for the RC column under fire exposure.

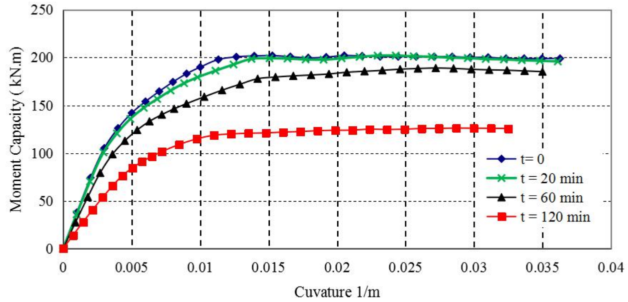


Figure 10; Moment–curvature curves at various times for the RC column under fire exposure (The axial load = 800).

Conclusions

Based on the experimental and analytical results of this study, the following conclusions can be drawn:

1. From transient thermal analysis results shown in **Figure 7** it is clearly that the reinforcement bars at the column corners are more affected by fire than the central reinforcement bars. Furthermore at 60 min fire duration the average temperature of the reinforcement bars is about 400°C and according to Euro code this will reduce the elastic modulus of steel by about 30%.
2. At 120 min fire duration the average temperature of the reinforcement bars is about 550° C and according to Euro code this will reduce the strength and elastic modulus of steel by about 32% and 40% respectively.
3. At 120 min fire duration and at a depth of 100 mm from the face of the column, the concrete temperature was equal to 300°C; and according to Euro code this will reduce the concrete strength by about 20%;that is the concrete strength becomes (17 x 0.8 = 13.8 MPa) and this value is in good agreement with the experimental test results given in Table 1 and Table 2
4. Strength capacity analysis results shows that up to 60 min of fire duration the column do not lose considerable amount of its strength, however at 120 min fire duration it will lose about 35 percent of its axial load and moment carrying capacities.
5. Depending on material test results, thermal analysis results and witnesses report it is clear that the building was exposed to fire up to 2 or 2.5 hours. And this will reduce the axial and moment capacities of the columns by more than 35%. Hence the building's columns need to be strengthened in order to be able to support future earthquake loads. Steel cages or RC jackets can be used for strengthening.

References

1. O. Arioz, "Effects of elevated temperatures on properties of concrete," Fire Safety Journal, vol. 42, pp. 516–522, 2007.
2. Nikhil Raut and KodurVenkatesh, (2012), Behavior of circular reinforced concrete columns under fire conditions, Journal of structural fire engineering, Volume 3 · Number 1, pp37-55.
3. Z. Tao, and X. Wang, "Stress-strain curves of structural and reinforcing steels after exposure to elevated temperatures," Journal of Materials in Civil Engineering, vol. 25, no. 9, pp. 1306-1316, 2013.
4. EN 1992-1-2:2004. Eurocode 2: Design of Concrete Structures, Part 1-2: General Rules – Structural Fire Design, European Committee for Standardization, Brussels, Belgium.
5. J. Stern-Gottfried, and G. Rein, "Travelling fires for structural design–Part I: Literature review," Fire Safety Journal, vol. 54, pp. 74–85, 2012.
6. "Fire Safety of Concrete Buildings," Cement Concrete & aggregate, Australia, 2010
7. BERNARD, E., WILLIAM, G. H., WILLIAM, H. k., "Evaluating fire damage to concrete structures" Part II of a three-part series on Repair of Fire Damage, 1972, The Aberdeen Group.

8. Yüzer, N.,Aköz, F. and Özturk, L.,D., “Compressive Strength-Color change relation in Mortars at high temperature” Cement and Concrete Research, V.34, No. 10, Oct.2004, pp. 1803-1807.
9. Kodur VKR, McGrath R. (2001), Performance of high strength concrete columns under severe fire conditions. In: Proceedings Third International Conference on Concrete under Severe Conditions, Vancouver, BC, Canada, p. 254–68.
10. ACI Committee 228.1R-03: “In-Place Methods to Estimate Concrete Strength”, American Concrete Institute, Farmington Hills, Mich., USA, 1994.
11. ASTM C597, “Standard Test Method for Pulse Velocity Through Concrete,” American Society for Testing and Materials, West Conshohocken, PA, 2009.
12. ASTM C42/C42M, “Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete,” American Society for Testing and Materials, West Conshohocken, PA, 2004.
13. ACI Committee 562-12 “Code Requirements for Evaluation, Repair, and Rehabilitation of Concrete Buildings and Commentary”, American Concrete Institute, Farmington Hills, Mich., USA, 2012.
14. ANSYS Software ,Southpointe 275 Technology Drive Canonsburg, PA 15317.
15. XTRACT Software , Imbsen and Associates, USA.