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Condition assessment of fire affected reinforced concrete shear wall building – A case study

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Abstract. The post - fire investigation is conducted on a fire-affected reinforced concrete shear wall building to ascertain the level of its strength degradation due to the fire incident. Fire incident took place in a three-storey building made of reinforced concrete shear wall and roof with operating floors made of steel beams and chequered plates. The usage of the building is to handle explosives. Elevated temperature during the fire is estimated to be 350^0C based on visual inspection. Destructive (core extraction) and non-destructive (rebound hammer and ultrasonic pulse velocity) tests are conducted to evaluate the concrete strength. X-ray diffraction (XRD) and Field Emission Scanning Electron Microscopy (FESEM) are used for analyzing micro structural changes of the concrete due to fire. Tests are conducted for concrete walls and roof slab on both burnt and unburnt locations. The analysis of test results reveals no significant degradation of the building after the fire which signifies that the structure can be used with full expectancy of performance for the remaining service life. This document can be used as a reference for future forensic investigations of similar fire affected concrete structures.

Keywords: fire; forensic investigation; shear wall building; non-destructive test; reinforced concrete

1. Introduction

Over the past few years, many fire incidents have taken place in civil engineering structures due to some unexpected state of affairs such as the use of explosives, accidents or other common causes of fire. Fire with high temperature is usually expected in the buildings that handle explosive devices. After a fire incident, it is important to assess the structure for available residual strength. Many fire-affected buildings in the world in recent years have been demolished while some others are left forsaken. Costs due to losses from fire are estimated at approximately one per cent of global GDP per year (The Geneva Association 2014). Assessment of fire damaged structures can always be considered an alternative to demolition as this can provide substantial savings in capital expenditure and also savings in significant losses in time, by permitting earlier reuse and re-occupation of structures (Ingham 2009). A methodical approach is necessary to evaluate any fire-affected structure to determine the level of damage and to suggest the future course of action

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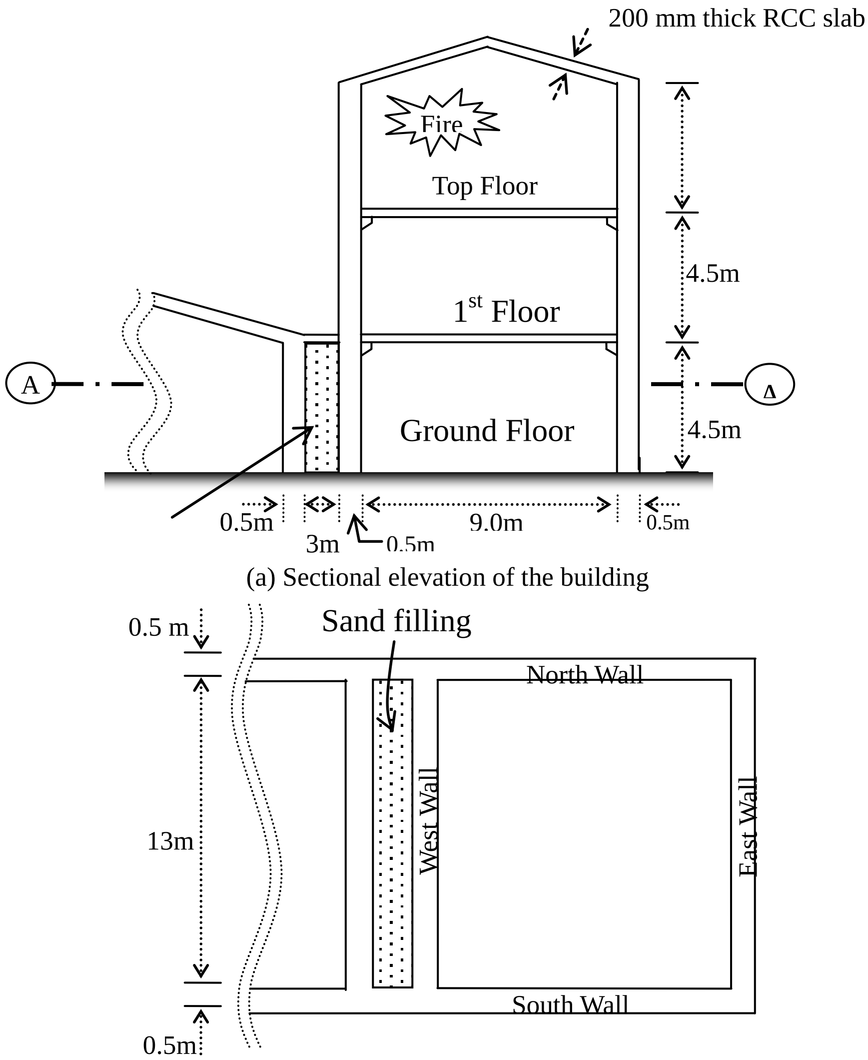
to the clients regarding its continued usage or disregard.

An extensive literature review revealed that although there are standards for design of fire resistant Reinforced Concrete (RC) structure (ACI 216.1-97, ENV 1992-1-2: 1996, IS 1642:1989 and NIST Technical Note 1842: 2014) no code, standard or federal recommendation is available for assessment of fire affected RC buildings. Being a heterogeneous material, there is an inherent scatter in the properties of concrete. Various studies were carried out during the last few decades to find the variation of different mechanical properties of concrete with temperature (Poon *et al*. 2001; Capua and Mari 2007; Willam *et al*. 2009; Prashant *et al*. 2015; Reddy *et al*. 2015; Balaji *et al*. 2015; Guo *et al*. 2016 and Wong *et al*. 2016). A number of studies (Short *et al*. 2001; Naus, 2005; Annerel and Luc 2011; Behnam *et al*. 2013; Kamath *et al*. 2015) on evaluation of fire affected reinforced concrete structures are also reported in published literature. Several defects such as cracking of concrete cover and corrosion of steel emanates after the occurrence of fire in an RC structure (Li 2011). Zhang *et al*. (2014) studied the influence of porosity, permeability and saturation of concrete exposed to fire. Kodur *et al*. (2010) has reported that the extent of strength loss of concrete under fire depends on many parameters such as type of exposure (duration, no. of faces and percentage of exposure), type of concrete (strength, aggregate type etc.), loading (level of initial load before and during exposure to fire), etc. However, the level of damage depends on several parameters, such as intensity and duration of the fire, age of the structure, amount of the clear cover, presence of high performance fire retardant/resistant coating, method of cooling, etc.

The intensity and duration of fire can be estimated by observing the damage and the field-combustible residues (Gosain *et al*. 2008). Investigation (Awoyera *et al*. 2014) of the significance of concrete cover to the reinforcement in a structural element at varying temperature revealed that the average ultimate tensile strength of steel decreased from 592 MPa at 30℃ to 314 MPa at 700℃ for concrete beam with a 25 mm cover. Therefore, there is a possibility of structural failure at elevated temperature due to the poor load carrying capacity of reinforcement steel. However, phase transformation of steel reinforcement does not occur up to a temperature of 723℃ (Singh 2010) and it gains its full strength back upon cooling to normal temperature. This fact is true even for structural steel elements. The bond between steel and concrete can be adversely affected at temperatures higher than 300℃ because of the differences in their thermal conductivity and thermal expansion properties (Haddad *et al*. 2008; Andrews 2011; Khalaf *et al*. 2016). Aslani and Bastami (2011) developed constitutive relationships for normal-strength concrete and high-strength concrete subjected to fire to provide efficient modelling and to specify the fire-performance criteria for concrete structures. Aslani and Samali (2013) developed bond constitutive relationships for normal and high-strength concrete subjected to fire. Choi *et al*. (2012) proposed a nonlinear computational modelling approach for the behaviour of structural systems subjected to fire. Ingham (2008) presented the application of petrographic examination techniques to the assessment of fire-damaged concrete and masonry structures. A reliability analysis was conducted for various reinforced concrete columns designed according to ACI code that are exposed to fire by Eamon *et al*. (2013). Colombo and Felicetti (2007) developed a new method for assessment of fire damage structure.

A detailed literature review in this area found very limited literature on the step-by-step procedure for assessment of fire affected reinforced concrete (RC) building. This paper presents a systematic evaluation of a fire damaged RC shear wall building through a case study. A methodology for condition assessment of the fire affected building is considered in three phases: (a) visual inspection to establish the intensity and duration of the fire, (b) non-destructive and destructive testing for strength evaluation and (c) microstructural studies (X-ray diffraction and



(a) Sectional elevation of the building

(b) Sectional plan of the building (A-A)

Fig. 1 Schematic diagrams of the building geometry

field emission scanning electron microscopy) to identify possible phase transformation due to elevated temperature.

2. Buildıng details

The fire damaged building considered here for the case study is constructed in the year of 1997-98. The plan dimension of the building is about 9 m × 15 m. The height of this building is about 13.5 m with three storeys of 4.5 m storey height. A schematic plan and elevation of the RC shear wall building is shown in Fig. 1. The four sides of the structure are made of RC shear walls of 500 mm thickness with an RC sloped roof (slope less than 10^0) of about 200mm thickness. The two intermediate floors are made of steel plates and steel beams supported by corbels projected from the RC shear walls on either side. A fire broke out in the second storey of this building on 30th April 2015. The approximate height, plan dimensions of the unit, number of storeys, thickness of concrete walls, the orientation of the walls, source of the fire, etc. are marked in Fig. 1. The walls are designated as per their relative positions such as North, South, and East and West walls as marked on this figure. In the absence of relevant structural drawing the clear cover of the reinforcement is measured to be 75 mm for RC wall and 25 mm for roof slab. Since the building handles explosives, both wall and roof slab were made with thick (20 mm) plaster and several layers of fire resistant coating. Also, the building is isolated from the adjacent unit by providing a sand layer of width 3 m between the walls to enclose possible blast.

3. Visual inspection

Following the fire explosion in the second storey of this building, a visual investigation of the building has been conducted and the observations are documented. The dark textured burnt zones of the interior face of the building are shown in Fig. 2. The duration of the fire and possible maximum temperature was established based on apparent damage to the concrete (surface colour, cracking and spalling) and the field-combustible residues.

The important observations during the site inspection are summarised below:

(a) The fire was extinguished by water jet introduced through the ventilation holes. Previous literature (Mandal 2015) reported the residual strength of concrete is lesser in case of water cooled structure compared to that of the air cooled structure. This is due to the higher temperature gradient associated with water cooling. Tests by Poon *et al*. (2001) suggested that with a post-fire duration of 56 days along with air/water re-curing, the concrete may regain certain amount of strength.

(b) The cement mortar plasters were found to be spalled off at certain places (Fig. 2). The surfaces of the structural components were found to be blackened due to the deposit of soot and smoke.

Fig. 2 Dark patches of burnt zones on the interior walls

Fig. 3 Burnt wooden frame found in the proximity of the source of the fire

(c) There was no indication of cracking, spalling and change in colour found in the concrete. No reinforcement bar is found to be exposed out of the concrete.

(d) The fire was found to cause no significant displacement, change in size and shape of the cross section of structural steel members. It was observed that the fire had apparently no significant effect on the structural steel element.

(e) A few of the insert plates connecting the steel attachments (such as monorail, etc.) to the concrete were found to fall loose fit due to the fire.

The possible maximum temperature due to the fire is correlated qualitatively from the field-combustible residues observed. For example, a burnt wooden frame was found in the proximity of the source of the fire as shown in Fig. 3. The cross section of the timber elements is approximately 50 mm × 25 mm and the element was exposed to fire in all four faces. The depth of the surface char layer formed was found to be negligible (5 mm or less) at the middle of the element. The charring of wood depends on the type of wood, duration of the fire and the temperature profile. It was informed that the fire lasted for at least four hours before being extinguished. Published literature (Frangin and Fontana 2003, Yudong and Drysdale 1992) has indicated that fire-point temperatures for different types of wood lie in the vicinity of 350-360℃ and the surface combustion starts well before the initiation of flaming. Therefore, it can be concluded that surrounding temperature due to the fire has not gone beyond 350℃. Investigation of effect of temperature on electrical fittings in the room such as plastic covers of electrical wires, bulbs made of glass and others also support this fact.

4. Detailed investigation

A detailed investigation is carried out after the visual inspection. This includes the assessment of concrete strength through non-destructive as well as invasive tests and assessment of the material passes through microstructural studies. No structural details with regard to the quality of the concrete, grade of the steel, etc. was available prior to the detailed investigation. Therefore, tests were carried out on concrete from burnt location as well as on similar concrete taken from neighboring unburnt locations. Assessment of damage is done by comparing the relative characteristics.

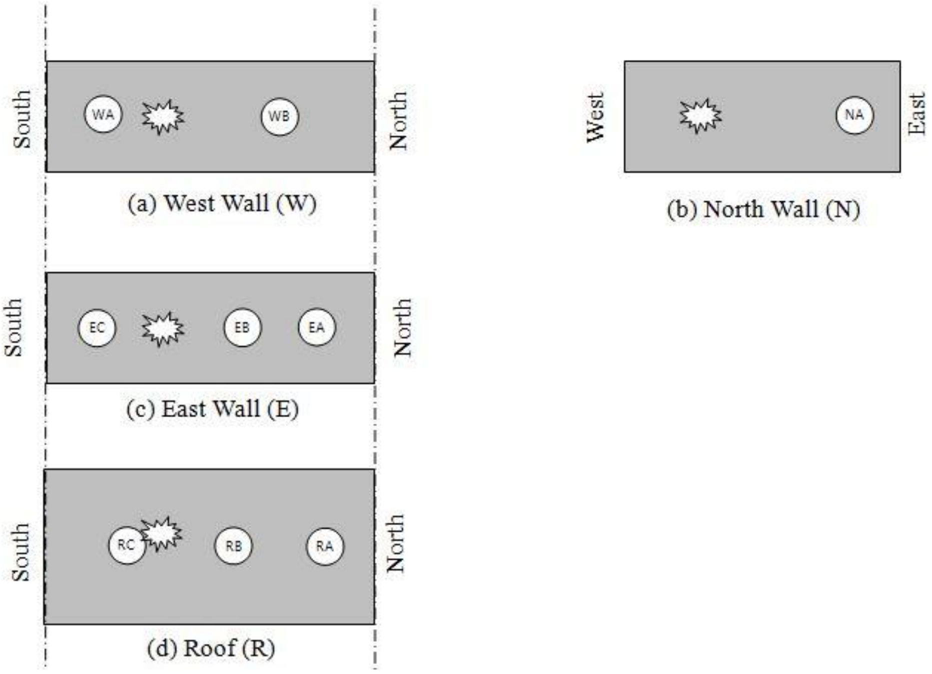


Fig. 4 Test points in burnt areas in second floor

The strength of all engineering materials, including steel reduces as their temperature increases. However, a major advantage of steel is that it is incombustible and it can fully recover its strength following a fire upto a certain temperature upon cooling to normal temperature. Previous literatures (Singh 2010; Tide 1998 and Raghavan 2006) indicate that heating of steel within 650℃ results in metallurgical changes that are predominantly temporary. Steel members which have slight or no distortions following a fire may be put to continued use with the full expectancy of performance with its specified mechanical properties (Kumar and Kumar 2015). There were no significant displacement, misalignment and distortion of structural steel members found in the present case study during the visual inspection. Also, the possible maximum temperature is estimated to be approximately 350℃. Therefore, it can be concluded that all the structural steel members (beams, floor decks, attachments, equipment, etc.) can be used with full expectancy of performance for the remaining service life. With this, the steel components of the building are kept out of the scope of detailed investigation and this section focus on the results of the concrete elements of the building. Although the visual inspection did not reveal any serious damage to the concrete structure, it was planned to carry out a detailed evaluation of the concrete walls and roof based on non-destructive testing (NDT) as well as more direct assessment involving concrete core sampling and testing. The details of selected test points and test results are presented in the following sections.

4.1 Test points for detailed investigation

The assessment of the damage to the concrete is carried out by comparing different parameters of the fire affected concrete (burnt concrete) with the associated parameters of the unaffected

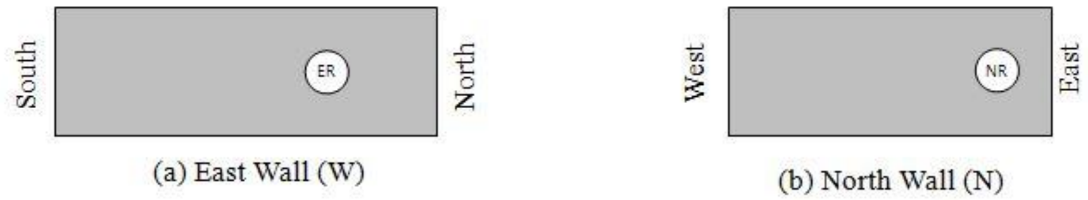


Fig. 5 Test points in unburnt areas in shear walls of ground storey

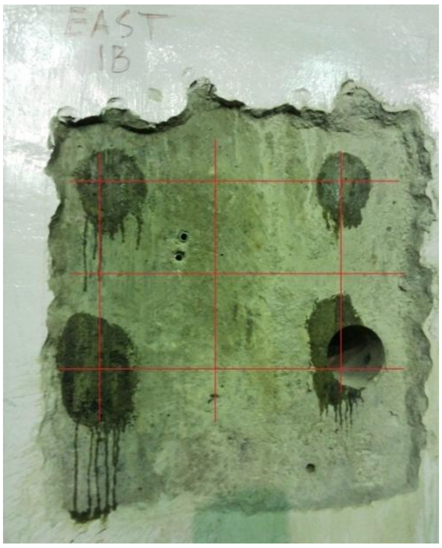


Fig. 6 Typical test point arrangement for NDT and core drilling

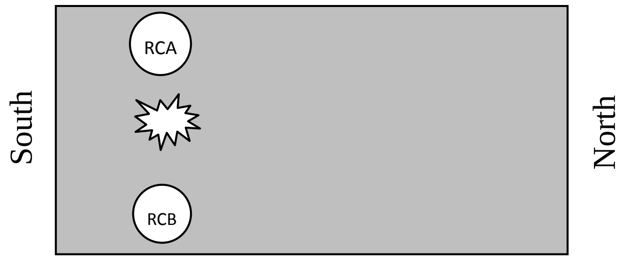


Fig. 7 Core test points in the burnt area of RCC sloped roof

concrete (unburnt concrete) of the same type of members (shear wall/slab) of the building. The test points in the walls and roof at the second storey of the building adjacent to the fire source are considered for the characterization of burnt concrete. The points considered in three walls of this storey, namely, west (WA, WB), north (NA) and east (EA, EB and EC) are marked in Figs. 4(a), 4(b) and 4(c) respectively. South wall could not be investigated due to inaccessibility. The points considered in the roof (RA, RB, RC) are marked in Fig. 4(d).

The fire occurred in the second storey and it was observed in the visual inspection that the ground storey is unaffected by the incident. Hence, to characterize the concrete from the unburnt areas, it was decided to consider points located in the shear walls of the ground storey (unaffected

Table 1 Rebound number of concrete at various test points

Sl.#	Location										
	Burnt									Unburnt	
	WA	WB	NA	EA	EB	EC	RA	RB	RC	ER	NR
1	47.0	42.5	51.5	50.5	41.5	39.0	51.5	45.5	31.0	35.0	43.5
2	48.5	37.5	41.0	47.5	44.8	43.5	47.0	52.0	29.5	-	47.5
3	43.0	48.0	34.5	41.5	43.5	43.0	40.0	49.5	36.0	34.5	45.5
4	40.0	51.0	27.5	43.5	48.0	43.5	43.5	41.0	39.5	39.5	-
5	47.5	39.5	40.0	38.0	40.8	46.0	43.5	43.5	33.0	39.0	48.0
6	41.0	32.5	47.0	34.0	47.0	46.5	34.0	45.5	-	44.0	41.5
7	43.0	32.0	43.0	39.5	45.5	44.0	51.0	43.0	-	-	50.5
8	47.0	35.5	43.0	39.5	39.5	45.5	47.5	38.5	-	-	-
9	49.0	39.0	48.0	46.0	43.5	53.0	37.0	44.0	-	33.5	51.0
10	48.0	41.0	40.5	33.5	51.5	-	40.0	56.0	-	35.5	48.0
11	53.5	42.5	35.5	43.5	43.5	-	41.0	45.5	-	33.0	38.0
12	52.5	36.0	38.5	36.5	50.5	-	-	49.0	-	37.0	47.5
Avg.					42.91					42.02	

by fire) to represent the concrete of unburnt state. Figs. 5(a) and 5(b) present the selected reference points in, east (ER) and north (NR) walls respectively. Fig. 6 presents photograph of a typical test point after the test. Although the above test points for burnt and unburnt locations are considered for testing, in order to avoid the regions of maximum bending moment in roof slab, core samples were taken from the locations marked as RCA and RCB in Fig. 7.

4.2 Results of rebound hammer test

At each test location, nine points with 200 mm grid spacing were considered and rebound hammer tests were conducted as per Indian standard IS 13311:1992 (Part -2) using Concrete Test Hammer (Proceq) with 2.2 N-m impact energy. Table 1 summarizes the rebound hammer numbers (RH) obtained from the test points from burnt and unburnt areas, ignoring the outliers as per Indian standard IS 8900:1978. The difference between the average rebound hammer value of burnt and unburnt concrete are found to be negligible. Associated average cube compressive strength of the concrete from burnt and unburnt areas are found to be 16.81 MPa and 17.16 MPa with suitable calibration.

As concrete is a heterogeneous material there is an inherent variability in its properties. In order to assess the relative changes in the properties of burnt and unburnt concrete considering this variability, probabilistic distributions of rebound numbers for both burnt and unburnt areas are fitted from the data of experimental values. Figs. 8 and 9 present the normal probability distributions for burnt and unburnt concrete respectively. The comparison of the rebound hammer numbers of burnt and unburnt concrete is plotted in Fig. 10. The trend of the probability distributions shows that the strength of burnt concrete is almost not affected due to fire in a mean sense.

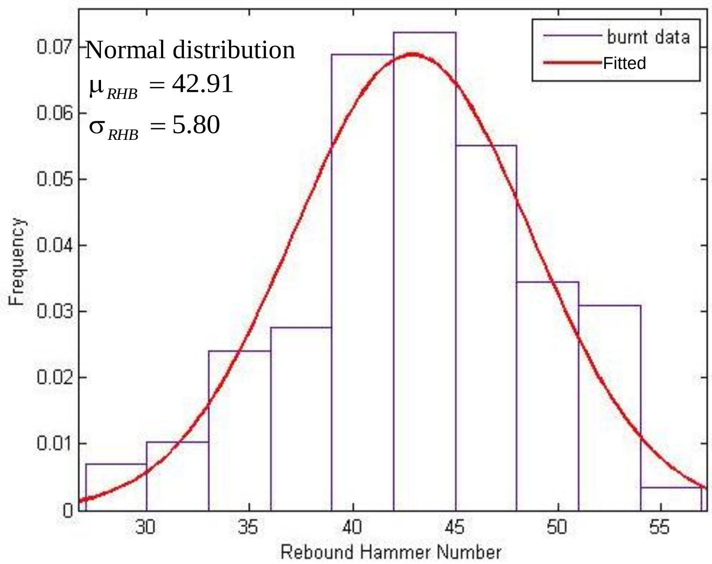


Fig. 8 Probability distribution and parameters fitted to RH data of the burnt area

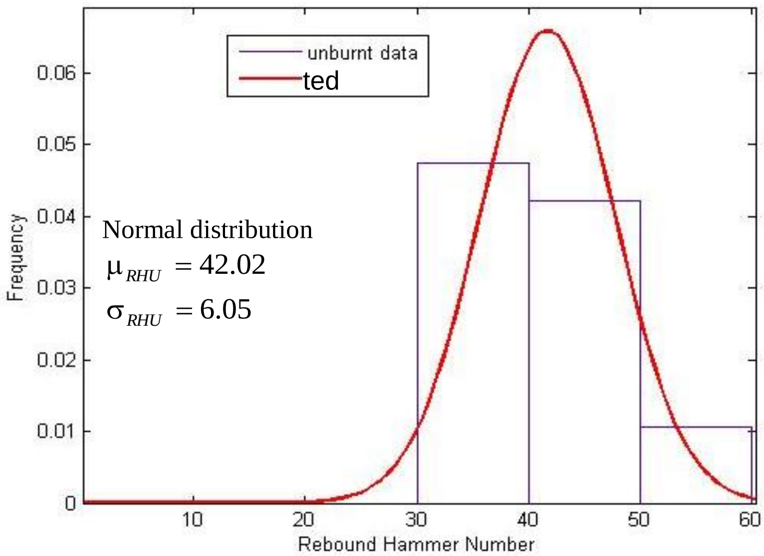


Fig. 9 Probability distribution and parameters fitted to RH data of unburnt area

4.3 Results of ultrasonic pulse velocity test

Similarly, four ultrasonic pulse velocity (UPV) readings were taken at each test location. This test was done by placing the transmitting transducer and receiving transducer on the same face of the concrete members (surface probing) as two opposite faces of the concrete member are not being accessible for measurements. The UPV test is conducted as per Indian standard IS 13311:1992 (Part-1) using Ultrasonic Test Equipment (Pundit Lab Plus). The path length is

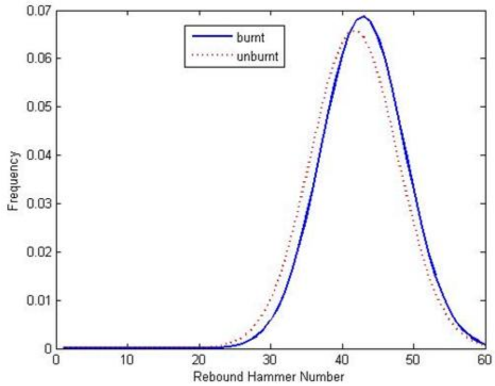
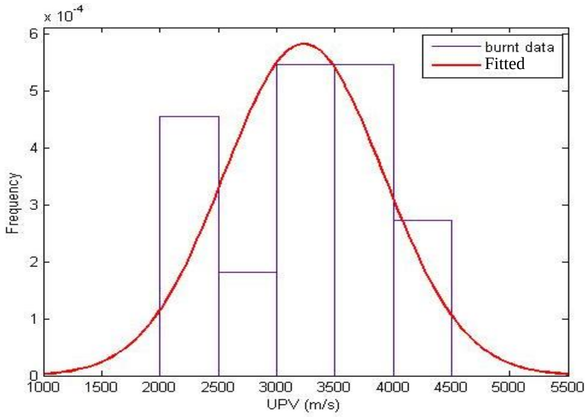


Fig. 10 Comparison of probability distributions for RH values

Table 2 Ultrasonic pulse velocity (m/s) of concrete at various test points

Sl. #	Location										
	Burnt									Unburnt	
	WA	WB	NA	EA	EB	EC	RA	RB	RC	ER	NR
1	3707	2068	3132	2913	3735	3431	-	-	-	3784	4107
2	4065	-	2045	3098	4107	3605	-	-	-	4206	3247
3	4338	2138	2436	3777	3472	3350	-	-	-	3568	3795
4	3431	-	2187	2994	3506	3633	-	-	-	3613	3428
Avg.	3235									3718	



Normal distribution

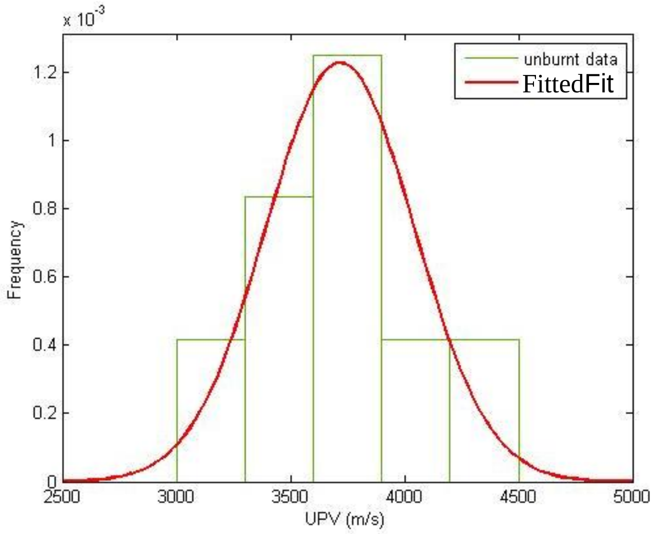
$\mu_{UPVB} = 3235\,m/s$

$\sigma_{UPVB} = 685\,m/s$

Fig. 11 Probability distribution and parameters fitted to UPV data of the burnt area

kept 400 mm with a transducer frequency of 150 kHz for all the UPV tests conducted in this study. Table 2 summarizes the UPV values obtained from all selected points ignoring the outliers as per IS 8900:1978. This is to be noted that the UPV test was not carried out in the roof slab due to insufficient accessibility.

Probability distributions for ultrasonic pulse velocities of both burnt and unburnt concrete are fitted and presented in Figs. 11 and 12. Fig. 13 presents both these distributions together to



Normal distribution

$$\mu_{UPVU} = 3718\,m/s$$
$$\sigma_{UPVU} = 625\,m/s$$

Fig. 12 Probability distribution and parameters fitted to UPV data of unburnt area

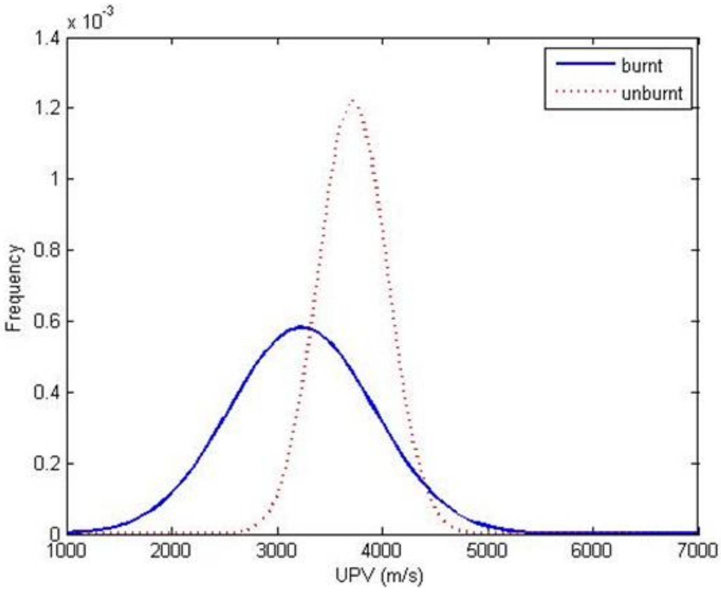


Fig. 13 Comparison of probability distributions for UPV values

compare them. The trend of the probability distributions shows that the burnt concrete has marginally lower mean (with higher dispersion) compared to the unburnt concrete. This may attributed to the dry pores within the burnt concrete.

4.4 Compressive strength from core tests

Cylindrical core samples of 100 mm diameter were extracted from the selected test points and the compressive strengths were obtained as per Indian standard IS 516:1959. As the aspect ratio

Table 3 Compressive strength calculation of core samples

SL#	Weight (kg)	d_1 (mm)	d_2 (mm)	d_{avg} (mm)	A (mm^2)	l_1 (mm)	l_2 (mm)	l_{avg} (mm)	l_{avg}/d_{avg}	η	F_{ult} (T)	$f_c^{'}$ (MPa)	$f_{c,\,cor}$ (MPa)	f_{ck} (MPa)
WB	1.14	90	90	90.0	6364	70	76	73.00	0.81	0.87	4	-	-	-
NA	1.60	93	94	93.5	6869	99	97	98.00	1.05	0.90	10	14.28	12.79	15.98
EA	1.86	93	93	93.0	6796	112	113	112.50	1.21	0.91	11	15.88	14.50	18.12
EB	1.58	93	93	93.0	6796	99	98	98.50	1.06	0.90	11	15.88	14.24	17.79
RCA	2.38	93	93	93.0	6796	144	143	143.50	1.54	0.95	10	14.44	13.71	17.14
RCB	2.48	93	94	93.5	6869	147	146	146.50	1.57	0.95	16	-	-	-
ER	1.00	95	94	94.5	7017	56	62	59.00	0.62	0.85	11	15.38	13.05	16.31
NR	1.80	94	94	94.0	6943	103	101	102.00	1.09	0.90	11	15.54	13.98	17.47

d_1 and d_2 = diameter of the core; d_{avg} = average diameter of the core; A = cross sectional area of the core; l_1 and l_2 = length of the core; l_{avg} = average length of the core; η = Correction factor; F_{ult} = ultimate failure load under direct compression; $f_c^{'}$ = cylindrical compressive strength; $f_{c,cor}$ = corrected cylindrical compressive strength and f_{ck} = equivalent cube compressive strength

Table 4 Summary of compressive strength (MPa) of core samples

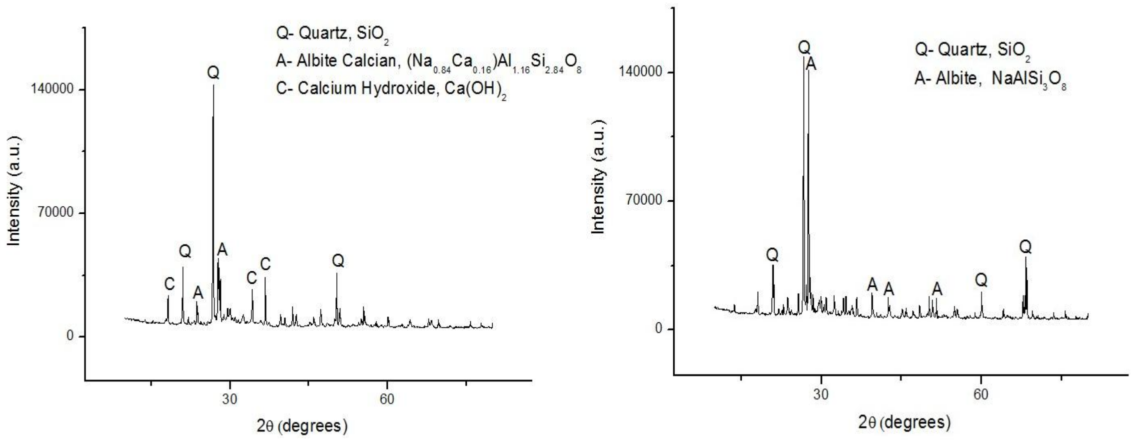
Sl. #	Location									
	Burnt								Unburnt	
	WA	WB	NA	EA	EB	EC	RCA	RCB	-	ER	NR
1	-	-	15.98	18.12	17.79	-	17.14	-	-	16.31	17.47
Avg.	17.26								16.89	

(height-to-diameter) of each core samples were different, suitable correction factors were applied. The calculation of compressive strength of core samples, including the correction factors is presented in Table 3. Table 4 summarizes the resulting compressive strength values with the average of all core samples for burnt and unburnt concrete ignoring the outliers as per IS 8900:1978. Comparison of average compressive strength of concrete at burnt (17.26 MPa) and unburnt (16.89 MPa) locations indicate that the fire has made no effect on the compressive strength of the concrete. It can be seen that the values of compressive strength obtained from core test is consistent with that estimated from rebound hammer test.

4.5 XRD Analysis result

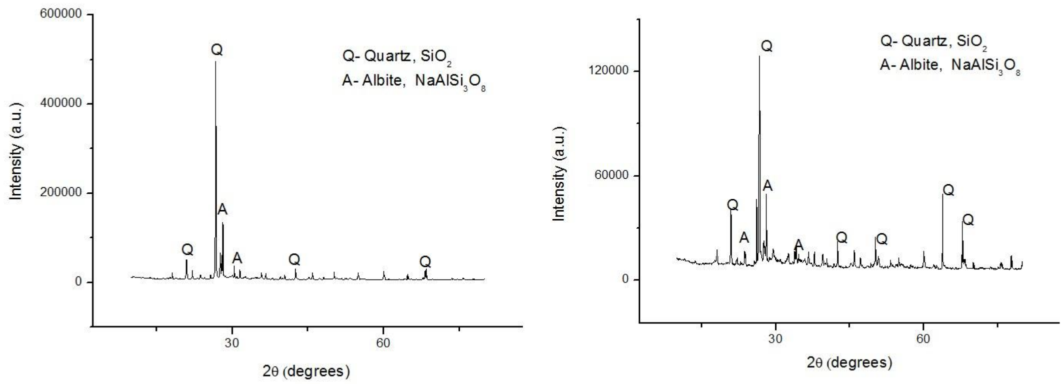
X-Ray Diffraction (XRD) is an analytic technique primarily used for phase identification of a crystalline material. In this study, XRD was performed for burnt and unburnt concrete specimens in order to identify any possible changes of the material phases or chemical compounds in concrete due to fire. This test was done using Rigaku Japan ULTIMA-IV multipurpose X-ray diffraction system. The specimens were grounded into a fine powder, sieved and then they were placed in a sample holder which was put in machine for testing. The testing was done with a scanning range of 10° to 80°, scanning rate of 0.05 degree/sec and step size of 20°/min.

The burnt specimens were taken from two locations, EA and EB (refer Fig. 4), and while the unburnt specimens were taken from ER and NR (refer Fig. 5). The XRD patterns of all the



(a) EA

Fig. 14 XRD pattern of burnt concrete specimens



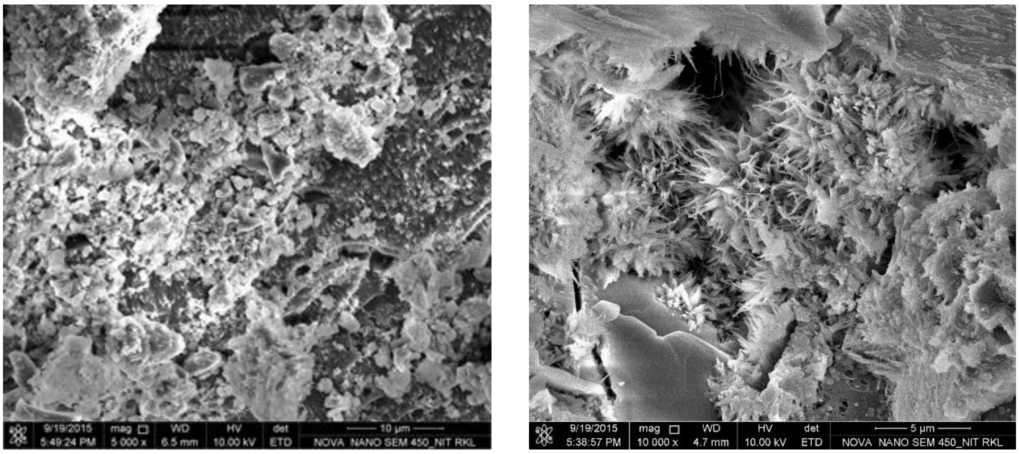
(a) ER(b) EB (b) NR

Fig. 15 XRD pattern of unburnt concrete specimens

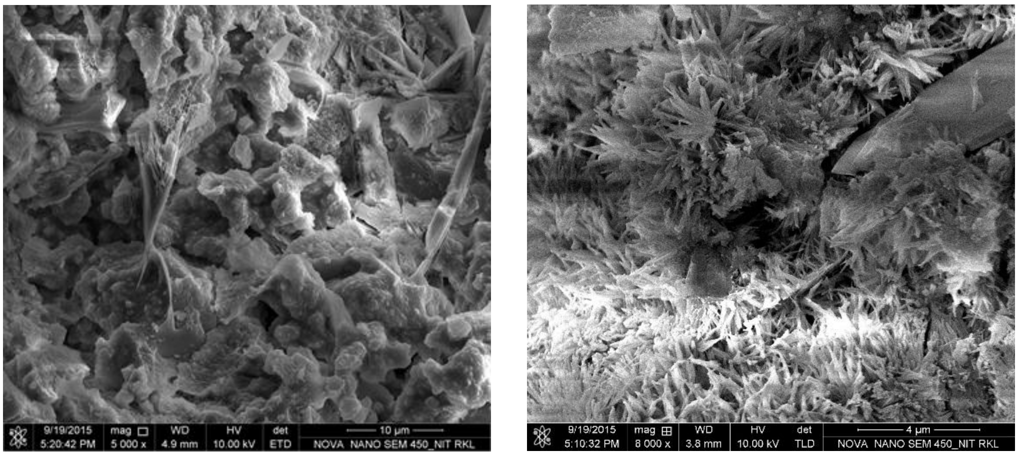
specimens are shown in Figs. 14 and 15. The concrete specimens show amorphous phase with intensified peaks of quartz (SiO_2) and albite ($NaAlSi_3O_8$). The peaks of quartz and albite compounds are predominantly found in all the burnt and unburnt specimens. Quartz is a common mineral compound in concrete because of its presence in cement, fine and coarse aggregates. Similarly, albite is feldspar mineral present in the aggregates. It is observed from the XRD analysis that there is no significant variation between the chemical properties of burnt and unburnt concrete.

4.6 FESEM analysis results

In order to detect any possible changes in the surface structure of concrete at molecular level due to fire, Field Emission Scanning Electron Microscopy (FESEM) was conducted on both burnt and unburnt samples of concrete using Nova Nano SEM/FEI model microscopic machine. Prior to



(a) Burnt concrete



(b) Unburnt Concrete

Fig. 16 Comparison of FESEM images of burnt and unburnt concrete specimens

testing, the specimens were coated with an electrically conductive gold material to avoid the charging effect. The coated samples were placed on carbon tape attached to sample holder which was then fixed in the machine. The micrographs were taken at different magnification levels for clear view of the microstructure.

The FESEM images of burnt and unburnt concrete specimens taken at different magnification levels are shown in Fig. 16. The figure shows the general hydration products of concrete with spiked-crystals of calcium based compounds embedded on silica based compounds, which are represented by rough patches. There is no significant difference observed between the burnt and unburnt concrete samples based on the surface structure of specimens.

4.7 Assessment of steel reinforcement

Under high temperatures, both strength (proportional limit) and stiffness of steel reinforcement deteriorate. However, considering the clear cover to the main reinforcement of 75mm in the concrete wall (25 mm in the roof slab), plaster thickness of 20 mm and fire retardant paint, it is unusual for the temperature of the steel within the concrete member to go beyond a tolerable limit to cause any damage to the bond or the strength of the steel. It is reported that ribbed high yield strength deformed bar does not lose strength up to 300^0C (Haddad *et al*. 2008, NIST Technical Note 1842: 2014, Kigha *et al*. 2015). Also, the reduction of bond strength between the deformed bars and surrounding concrete is negligible up to a temperature of 300^0C (Willam *et al*. 2009, Lublóy and György 2014). Therefore, the reinforcing bars in the present study are assumed to be not affected by the fire incident.

5. Summary and conclusions

Case study of post-fire investigation of a reinforced concrete shear wall building is discussed in this paper. The methodology for the post-fire forensic investigation involves visual inspection, NDT (rebound hammer and ultrasonic pulse velocity), more direct core sample extraction and testing, and microstructural study (XRD and FESEM).

It is concluded from the visual inspection that the probable elevated temperature is approximately 350℃. The assessment of the damage to the concrete is carried out by comparing the properties of the burnt concrete with that of the unburnt concrete of the same building. The trend of the probability distributions that describes the inherent variability of NDT values shows that the strength of burnt concrete is almost unaffected due to fire in a mean sense. Comparison of average core compressive strength (17.26 MPa for burnt and 16.89 MPa for unburnt) of concrete also justifies the above observation. Neither XRD based chemical properties, nor FESEM based images of burnt and unburnt parts of concrete shows any significant differences. This implies that the fire incident did not change the properties of the materials and also the structure as a whole. Hence, it can be concluded that the building can be used with full expectancy of performance for the remaining service life. In absence of any codes and standards this case study can be used as a guideline to conduct assessment of RC buildings affected by fire systematically.

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