Assessment of Residual Strength Based on Estimated Temperature of Post-Heated RC Columns

MUHAMMAD YAQUB*, IMRAN A. BUKHARI**, AND USMAN GHANI*

RECEIVED ON 01.12.2010 ACCEPTED ON 21.06.2012

ABSTRACT

The experience shows that fire-damaged concrete structures both technically and economically can be reinstated after fire due to high fire resistance and high residual strength. The residual strength of fire-damaged concrete structural member depends on the peak temperature reached during fire, fire duration and the distribution of temperature within the structural member. The assessment of the residual strength of post-heated concrete structural members in a professional way is a prime factor to take a decision about the reinstatement or demolition of fire-damaged structure. This paper provides an easy and efficient approach to predict the residual strength of reinforced concrete columns based on the estimated temperature which may have occurred within the concrete cross-section during a fire. A finite element model was developed to evaluate the distribution of temperature within the cross-section of the reinforced concrete columns. Twelve reinforced concrete square columns were heated experimentally up to 500°C at 150°C/hour. A comparison of the experimental temperature values of the tested columns was made with the model results. A good agreement was found between the experimental and the finite model results. Based on the temperature distribution obtained from the finite element model, the residual strength of concrete and reinforcement could be evaluated by using the relationships for concrete, steel and temperature proposed by various researchers.

Key Words: Columns, Residual Strength, Temperature, Finite Element Model.

1. INTRODUCTION

Generally the temperatures higher than 900°C are common in buildings during fire [1-2]. The concrete structure due to high fire resistance, hardly ever results in a serious damage due to fire [3]. The repairing of fire damaged concrete structure 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 [3]. In fire damaged concrete structures, the knowledge of the residual properties of materials are needed as a basis for the decision of reconstructing or repairing and for the design of the repair structure. This paper provides a practical and a valuable guidance to the practicising engineers to take a decision for the fire damaged concrete structures whether it can be reinstated or it must be demolished. To evaluate the residual strength of fire damaged structural member, the easy and efficient approach presented in this paper consisted of three steps.

* Assistant Professor, Department of Civil Engineering, University of Engineering & Technology, Texila.
** Director, Research & Development Organization, Government of Pakistan, Islamabad.

The first step involves the judgement of the peak temperature reached during fire and the duration of fire. The second step is to make heat transfer analysis finite element method to evaluate the distribution of temperature within the concrete cross-section based on the estimated temperature of surface and fire duration. The third step is to assess the residual strength based on the temperature distribution within the cross section of structural members using residual strength versus temperature relationships suggested by various researchers.

1.1 Step-1: Estimation of Maximum Temperature Reached and Duration of Fire

There are many factors which contribute in the reduction of concrete strength during fire. However, the primary effect of fire on concrete structures depends on the peak temperature reached during fire and the duration of fire [4].The maximum temperature and duration of a fire depends on fire load, ventilation conditions, geometry and material properties of the compartment. After making sure that the structure is safe to enter after fire, probable reasons of the fire should be carefully recorded before the removal of debris. In a real fire situation, the severity of fire is not uniform throughout the building and may have remained localized for a long time; the rate of temperature rise may have been faster, or slower, than in the standard test, or extensive spread may have occurred [5]. Different parts of the building may have reached different peaks of temperatures due to different fire intensities. Therefore, the building should be divided into horizontal and vertical zones so that the severity of fire can be assessed in detail for each zone. There are four feasible approaches to assess the severity of fire and the duration of fire [6].

The first approach is to use fire brigade records and the evidence of eyewitnesses which gives an indication about the times of starting and extinguishing of fire and the effort to control the fire. It gives only a qualitative judgment but not the quantitative assessment, as it provides the information whether the fire was small or large, more or less damaging, of long or short duration and which particular areas had higher temperatures than others. The second approach is to examine the maximum temperature of the debris reached during fire. In buildings there is range of metallic and non metallic materials, each of which has a different temperature at which it suffers physical or chemical changes [7]. An examination of the debris may not give the actual picture of the fire temperature and it is difficult to predict the actual temperature from the examination of the debris due to local fluctuations of fire [8]. However, Table 1 gives an approximate guidance to the assessment of temperature reached by selected materials and various components in building fires [8]. This gives only clue about maximum particular temperatures but it does not give the direct indication regarding the actual temperature and the total duration of exposure to that temperature.

In the third approach, the duration and severity of the fire may be obtained from the depth of charred timber that has remained in place throughout the fire. For all practical purposes, the timber will char at a constant rate on each face in the standard furnace tests. Generally, the char increases at 40mm per hour in the standard fire test. The information is less relevant if the timber had not remained in place and has fallen to the ground with other debris [8]. Attention should be given to the species of timber, position of timber and the nature of fire load. The timber fixed at higher levels in a room for instance would have been exposed to more severe conditions as compared to near the floor [7]. The rates which are given in Table 2 are based on Section 4.1 of BS:5268 [9] and allow an assessment to be made in terms of an equivalent fire resistance time.

The fourth method is to examine the colour of concrete. The colour of concrete changes during heating and it is irreversible therefore; colouration of concrete at various depths allows an estimation of fire severity and the equivalent fire duration [10]. The normal grey colour of

TABLE 1. MELTING TEMPERATURES FOR VARIOUS MATERIALS [7]

Substance	Typical Examples	Conditions	Approximate Temperature (°C)
Paint		Deteriorates Destroyed	100 150
Polystyrene	Thin-wall food containers, foam, light shades, handles, curtain hooks, radio casings	Collapse, Softens Melts and flows	120 120-140 150-180
Polyethylene	Bags, films, bottles, buckets, pipes	Shrivels Softens and melts	120 120-140
Polymethyl methacrylate	Handles, covers, skylights, glazing	Soften Bubbles	130-200 250
PVC	Cables, pipes, ducts, linings, profiles, handles, knobs, house ware, toys, bottles(Values depend on length of exposure to temperature)	Degrades Fumes Browns Charring	100 150 200 400-500
Cellulose	Wood, paper, cotton	Darkens	200-300
Wood		Ignites	240
Solder lead	Plumber Joints, plumbing, sanitary installations, toys	Melts Melts, sharp edges rounded drop formation	250 300-350 350-400
Zinc	Sanitary installations, gutters downpipes	Drop formations Melts	400 420
Aluminium and alloys	Fixtures, casings, brackets, small mechanical parts	Softens Melts Drop formation	400 600 400
Glass	Glazing, bottles	Softens, sharp edges rounded Flowing easily, Viscous	500-600 800
Silver	Jewellery, spoons, cutlery	Melts Drop formation	900 950
Brass	Locks, taps, door handles, clasps	Melt(particularly edges) Drop formation	900-1000 950-1050
Bronze	Windows, fittings, doorbells, ornamentation	Edges rounded Drop formation	900 900-1000
Copper	Wiring, cables, ornaments	Melts	1000-1100
Cast iron	Radiators, pipes	Melts Drop formation	1100-1200 1150-1250

Ordinary Portland Cement Concrete changes to light pink at around 300°C and becomes darker attaining the maximum intensity at about 600°C [7]. The temperature of 300°C is significant for three reasons. Firstly, a pink coloration occurs at this temperature [11]. Secondly, below that temperature the effect of heat on concrete strength is likely to be structurally insignificant [12]. Thirdly, a pink discolouration above 300°C, indicates the inception of significant strength loss due to heating [8]. In the siliceous aggregates, colour change to pink tends to be more prominent. However the calcareous and igneous crushed rock aggregates did not give clear indication of colour change due to fire [11].

The change in colour is due to the transformation of ferric compounds present in aggregate or in the sand as impurities to ferric oxide [7]. However, these salts are not found in concrete in some of the cases. It is therefore, the concrete that does not show pink colour on heating, is not necessarily undamaged by fire [8]. The intensity of colour depends upon the level of impurity and the colour changes have been noticed even with limestone aggregate concrete when river sand is used as fine aggregate [7]. After removal of pink coloured concrete it may be assumed that remaining concrete has an average strength not less than 80% of the strength before fire [12]. Fig. 1 shows the changes in colour of concrete at different temperatures [5].

TABLE 2. CHARING RATE OF WOOD [8]

Timber	Depth of Charring in 30 Minutes	Depth of Charring in 60 Minutes
All Structural Species Except Those Listed Below	20mm	40mm
Western red cedar	25mm	50mm
Hardwoods Baving a Nominal Density Not Less Than 650kg/m³ at 18% Moisture Content	15mm	30mm

To examine the change in colour, a piece of concrete from the damaged section should be removed, including the exposed surface in order to establish the spectrum of colour changes. The furthest depth at which pink colour can be seen may be taken as the boundary for the 300°C isotherm [7]. Where it is difficult to assess the depth of the pink layer, a small diameter core can be extracted from existing fire damaged portion of concrete in order to examine it accurately. The depth of discolouration shows the fire severity in terms of the equivalent duration of standard fire test.

The fifth approach is based on the equations proposed by Lie, [13] and BS EN [14] for determining equivalent fire resistance. Estimations have to be made for each and every compartment, or floor, depending on the layout of building. This estimation needs the size of the compartment, size of openings (ventilation factor) and the fire load density. This approach is based on the assumption that the whole fire load is burnt without any interruption and the whole ventilation is available from the start of the fire. Therefore, the result of these equations may not be totally accurate for a large compartment fire. Practically none of the above approaches gives completely reliable results but the combination of all approaches gives a reasonable prediction about the maximum temperature and the duration of fire.

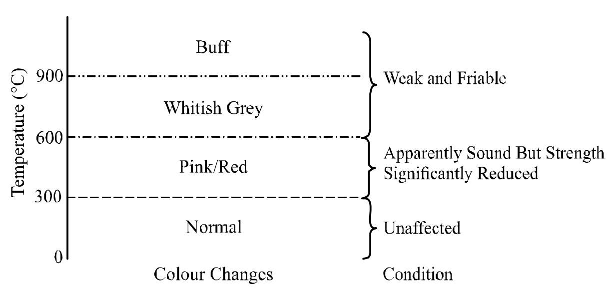

FIG. 1. COLOUR CHANGES IN CONCRETE [4]

1.2 Step-2: Heat Transfer Analysis

The heat transfer analysis is important for the prediction of temperature distribution within the cross-section of the structural member. There are three approaches. First is to use the charts for temperature distributions in dense concrete elements based on standard fire tests given in BS EN [15]. Secondly to use simple formulae proposed by Wickstrom, [16]. The third one, which is considered a more realistic approach, is to estimate temperature time distributions within the cross-section of structural members using finite element methods for heat transfer analysis. In the present work to validate the heat transfer analysis finite element model, twelve reinforced concrete square columns were constructed and heated within the Structural Engineering Laboratory at the University Manchester, UK. The experimental programme and the finite element analysis for heat transfer are discussed in the following sections.

2. EXPERIMENTAL PROGRAM

2.1 Construction of RC Square Columns

Twelve reinforced concrete square columns were constructed within the structural engineering laboratory. All square columns were cast in a horizontal position using steel moulds for the formwork. The steel moulds were properly oiled on the inner sides for easy removal of the specimens at the time of demoulding. The cage of reinforcement consisting of 8 No. 10 longitudinal bars and 6mm diameter deformed bars were used as a link bars spaced at 100mm centres. The prepared cage of reinforcement was kept in the moulds carefully. Concrete spacers of 25mm size were used to maintain 25mm concrete cover to the main reinforcement using binding wire. The concrete was poured in three layers and compaction of each layer was carried out using a vibrating table. In order to record the temperature at the time of heating, two type K-thermocouples were embedded in each column. One was placed at mid height in the centre of the column, whereas the second K-thermocouple was attached to the longitudinal reinforcement. After 24 hours of casting, the specimens were cured using moist sacking. To prevent the loss of moisture, the specimens were covered with plastic sheet and curing was continued for fourteen days. The specimens were then kept in the laboratory environment until the day of heating and testing.

2.2 Heating of RC Square Columns

Twelve reinforced concrete square columns were heated in an electric furnace having internal size of 1.6 x1.2x1.5m. The arrangement of post-heated square reinforced concrete columns in an electric furnace is shown in Fig. 2. A maximum of six columns were heated at a time with the average heating rate used in the experiment was 150°C/hour. The square columns were heated to a uniform temperature of 500°C. The temperatures within the furnace were controlled with the help of two type-K thermocouples attached at mid-height and at the top of the electric furnace.

FIG. 2. ARRANGEMENT OF POST-HEATED SQUARE REINFORCED CONCRETE COLUMNS IN FURNACE

When the average furnace temperature reached 500°C, the furnace temperature was kept constant until the temperature at the centre of columns reached the same temperature. The duration of heating of columns to achieve the point of uniformity of temperature inside the furnace and at the centre of columns was eight hours. After achieving the point of uniformity of temperature inside the furnace and at the centre of columns, the furnace was switched off and all the columns were cooled down naturally within the boundary of furnace.

2.3 Modelling of RC Columns Using FEM

The computer software ABAQUS 6.10.1 was used to perform the numerical heat transfer analysis for the heated reinforced concrete square columns. ABAQUS is an advanced nonlinear finite element analysis programme and is used for stress, heat transfer and other types of analysis in civil and other related engineering applications.

A thermal 3-D analysis was carried out of the reinforced concrete column using the a finite element analysis program ABAQUS [17]. The concrete column was modelled using an 8-node linear heat transfer brick elements called DC3D8 available in ABAQUS/Standard. The model was partitioned into 25mm sized elements in the longitudinal and transverse directions, as shown in Fig. 3 in order to apply the structured meshing technique to the entire model for making the analysis faster. The cubical mesh size of 12.5mm was used in the model. In the thermal analysis reinforcement is neglected taking the reinforced temperature equal to the surrounding concrete temperature because all columns were heated uniformly. A convective coefficient of 25W/m^{2}K was assumed for the fire exposed and 9W/m^{2}K for the unexposed surface. The radiative heat flux was estimated using a concrete emissivity value of 0.7. The density is assumed to be constant at 2300kg/m^3 for concrete. The concrete moisture content, specific heat and thermal conductivity was taken from the Eurocode 2 [15]. Figs. 4-5 show the values of specific heat and thermal conductivity used in the model. Fig. 6 shows the distribution of temperature in the finite element model of reinforced concrete square columns.

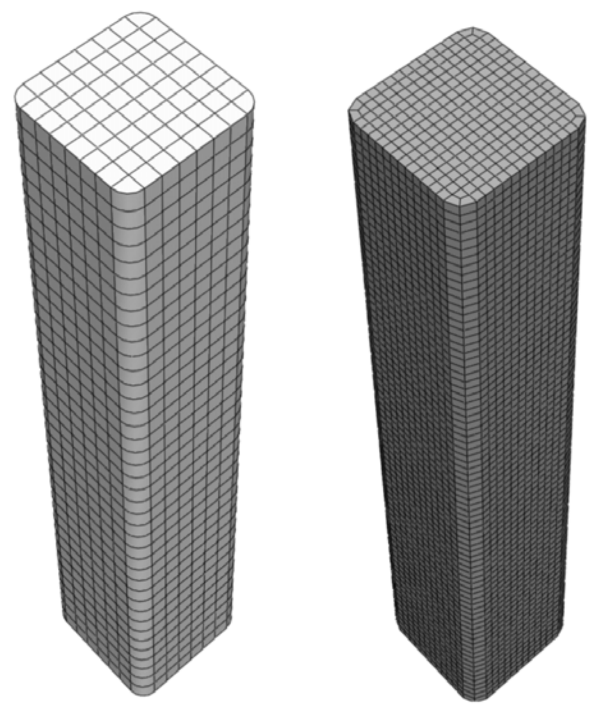

FIG. 3. PARTITION AND STRUCTURAL MESHING OF SQUARE REINFORCED CONCRETE COLUMNS FOR HEAT TRANSFER ANALYSIS

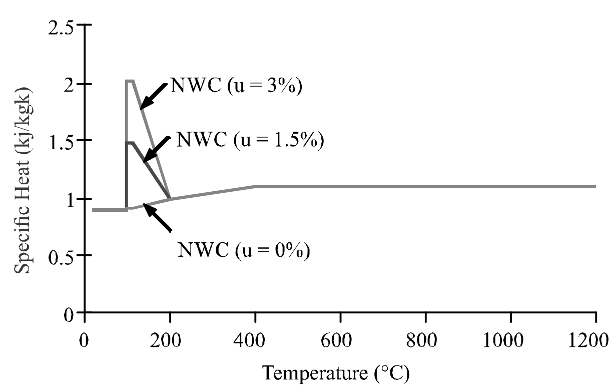

FIG. 4. VARIATION OF SPECIFIC HEAT WITH TEMPERATURE AND MOISTURE CONTENT [15]

3. RESULTS AND DISCUSSION FROM THE THERMAL ANALYSIS

The comparison of the time-temperature curves obtained from the finite element analysis and the measured experimental data at mid height in the centre of square columns are shown in the Fig. 7. The time-temperature curves obtained from the finite element (ABAQUS heat transfer) analysis model agree quite well with the experimental measured data for the concrete columns.

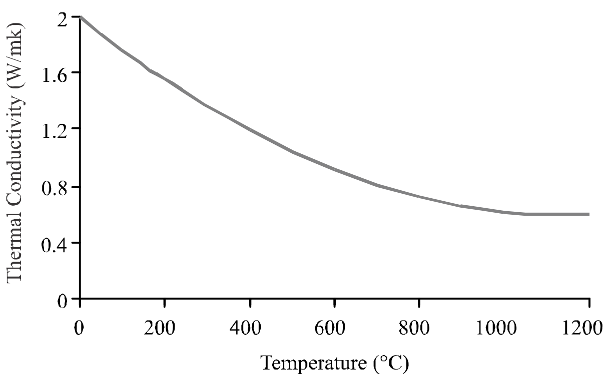

FIG. 5. UPPER BOUND LIMIT OF VARIATION OF CONDUCTIVITY OF CONCRETE WITH TEMPERATURE [15]

However, the peak temperature values in the finite element model were slightly lower compared to the experimental values, as shown in the Table 3. It could be due to variation in the moisture content.

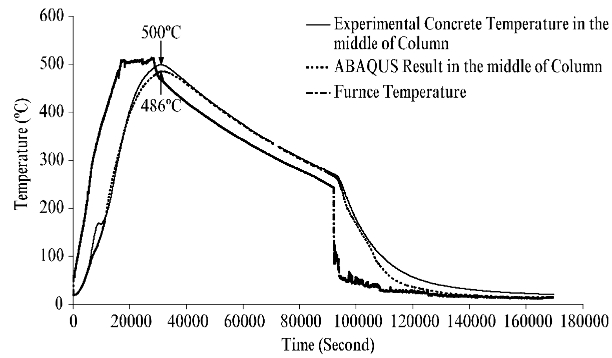

FIG. 7. COMPARISON OF EXPERIMENTAL MEASURED TIME TEMPERATURE CURVE WITH THE NUMERICAL MODEL RESULTS IN SQUARE COLUMNS

TABLE 3. COMPARISON OF EXPERIMENTAL PEAK TEMPERATURES WITH THE FINITE ELEMENT MODEL

Square Columns	Experimental Temperature (°C)	Finite Element Model Temperature (°C)	Difference (%)
Group-1 (Six Columns)	500	486	3
Group-2 (Six Columns)	511.6	492	4

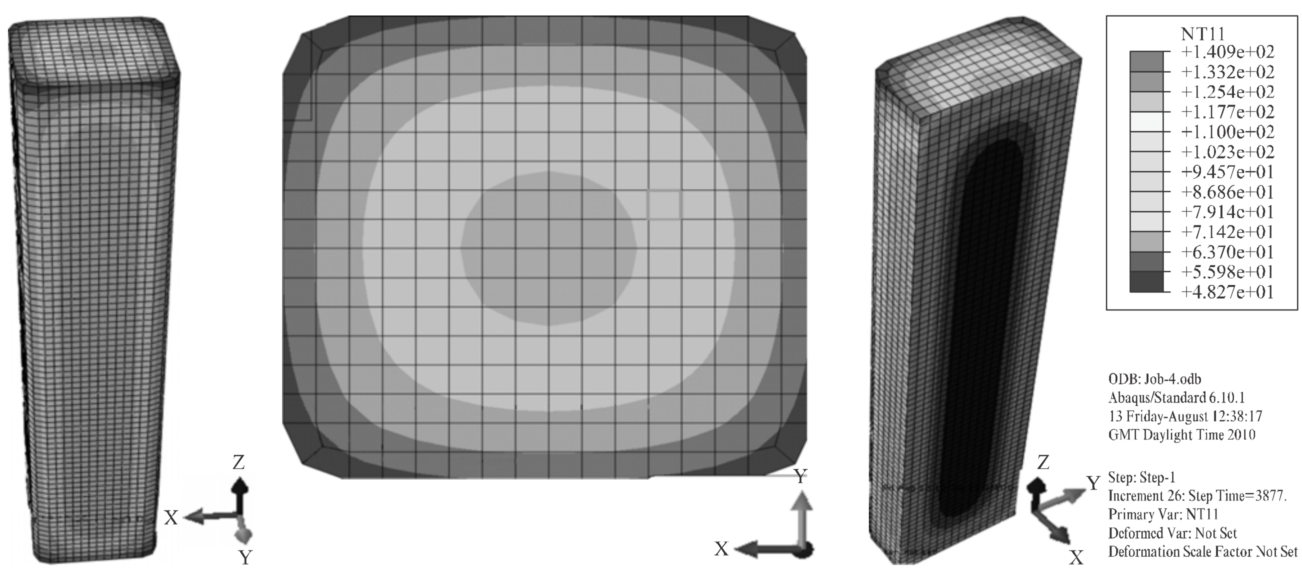

FIG. 6. DISTRIBUTION OF TEMPERATURE IN FINITE ELEMENT MODEL OF REINFORCED CONCRETE SQUARE COLUMN

3.1 Step 3-Assessment of Residual Strength of Concrete and Reinforcing Steel

The residual strength of concrete can be estimated by applying the residual strength and temperature relationships reported by Hertz, [18], Chang, et. al. [19], BS EN [20], Concrete Society [8,11], CIB W14 report [2], Marchant, [21]. The residual strength of reinforcing steel can be estimated by applying the relationships reported by Hertz, [18] and the Concrete Society [8,11]. To determine the average damage in concrete structures, the structural members can be divided into inner, intermediate, and outer zones according the temperature distribution, as shown in Fig. 8. The average damage factor of 1.0 could be applied for all concrete subjected to temperatures less than 100°C (inner zone refer Fig. 8) [2].

For the concrete heated in the range of 100-300°C (intermediate zone refer Fig.8), the damaging factor of 0.85 could be used for the evaluation of residual strength of fire damaged concrete [2]. However, the damage factor of 0.6 and 0.5 could be applied for the concrete heated between 300-400°C and between 400-500°C respectively (outer zone refer Fig.8) [21]. A damage factor of 0.20 could be used for concrete subjected to temperatures ranging from 500-600°C. At higher temperatures from 600-900°C, concrete becomes weak and friable and may be considered in the range of 100% strength loss. However, for the sake of assessment of fire damaged concrete structure on the safer side, the Concrete Society TR68 [8] assumed the zero strength of concrete above 300°C [outer zone refer Fig. 8]. After the removal of pink coloured concrete (above 300°C) it may be assumed that the remaining concrete has an average strength not less than 80% of the strength before fire.

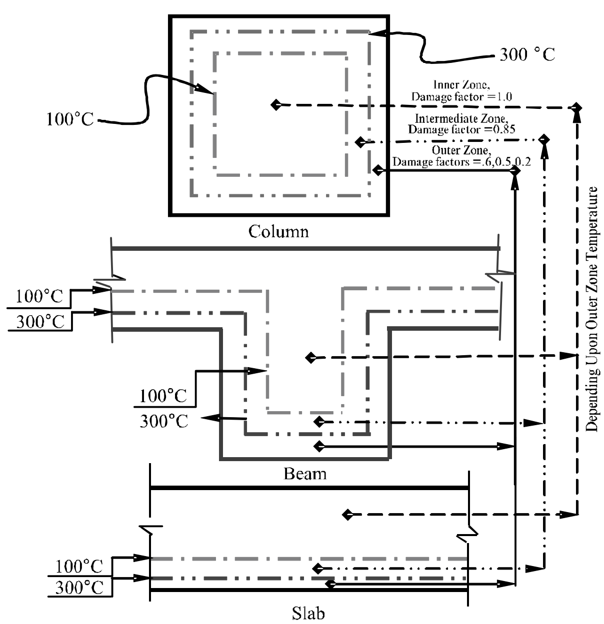

FIG. 8. FIRE DAMAGE FACTORS FOR COLUMNS, BEAMS AND SLABS BASED ON TEMPERATURE PROFILE [10]

The Concrete Society is very active in producing comprehensive updates to assess the strength and rehabilitation of fire damaged concrete structures. The Concrete Society TR15 [1] suggested a strength reduction curve based on the research of unstressed residual strength of concrete, as shown in Fig. 9. It has been found that the strength of the loaded specimens is generally higher than those of unloaded specimens during heating [18]. Therefore the un-stressed residual strength test is considered more conservative for assessing the post-fire or residual properties of concrete [6]. In reality, all concrete structures would be stressed, at least under dead load, at the time of heating. Therefore, the Concrete Society had made some modifications in the TR33 report [11] and suggested the strength reduction curve shown in Fig. 10 based on the residual strength of heated stressed concrete after cooling.

The Concrete Society published a report TR68 [8] in which it is assumed that the concrete heated to above 300°C has lost all of its strength. It has been found that the normal grey colour of Ordinary Portland Cement

Concrete turns to light pink at around 300°C [10]. Therefore the 300°C temperature is considered as the boundary for the pink colour and it identifies the limit of damaged concrete. The change of colour is not apparent for all types of aggregates. Therefore, the concrete exposed to more than 300°C should be removed prior to repairing. For the sake of assessment of fire damaged concrete structure on the safer side, the Concrete Society TR68 [8] assumed the zero strength of concrete above 300°C, as shown in Fig. 11. Marchant, [21] reported that concrete retains 50% of its compressive strength after exposing to temperature 500°C, as shown in Fig. 12. It is worth mentioning here that the colour changes shown in Figs. 9-12 are applicable for siliceous aggregate and may not be applicable for calcareous or crushed flint aggregate.

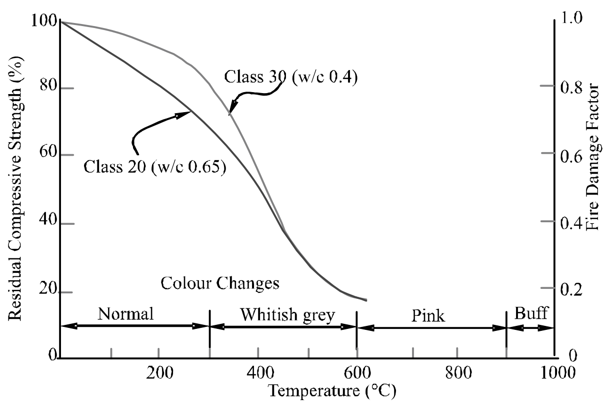

FIG. 9. RESIDUAL STRENGTH OF HEATED UN-STRESSED DENSE AGGREGATE CONCRETE AFTER COOLING [1]

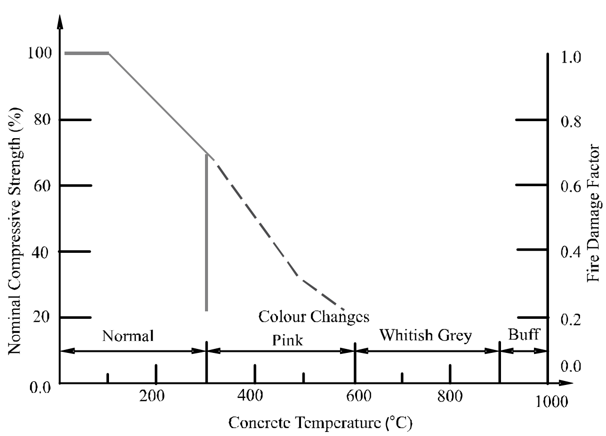

FIG. 10. RESIDUAL STRENGTH OF HEATED STRESSED DENSE AGGREGATE CONCRETE AFTER COOLING [10]

The variation in residual compressive strength of concrete reported by various researchers depends on many factors which contribute in the strength loss. Some of them are described in the following sections.

3.2 Effect of Methods of Cooling on the Compressive Strength of Concrete

The relative residual compressive strength of concrete after various cooling regimes decreases monotonically.

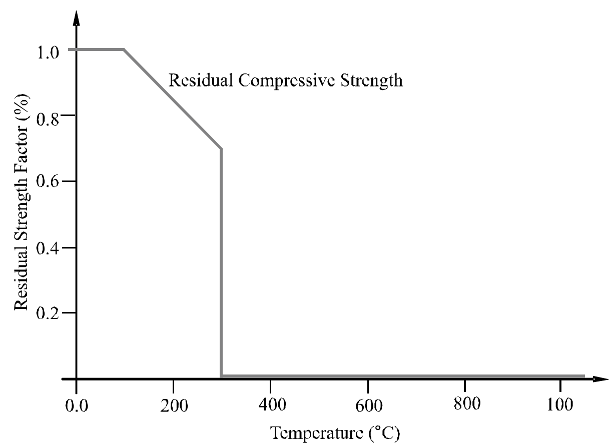

FIG. 11. RESIDUAL COMPRESSIVE STRENGTH OF CONCRETE AFTER COOLING [7]

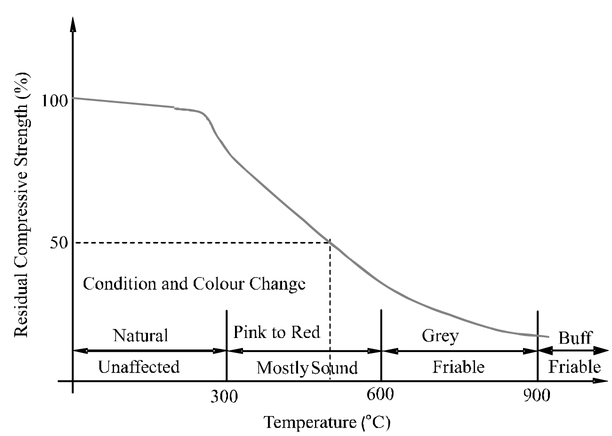

FIG. 12. RESIDUAL COMPRESSIVE STRENGTH OF CONCRETE AFTER COOLING [20]

After cooling, the material takes time to come to equilibrium with its surroundings [18]. It is found that the compressive strength measured immediately after cooling is somewhat lower than that at high temperature [18,23] and there is further post-cooling variations due to re-absorption of moisture to the concrete [18,23]. The post-cooling changes in strength that occurs as a concrete element approaches equilibrium with its surroundings are accompanied by dimensional changes within a structure. These changes would cause further redistribution of stress and possibly additional damage during the period immediately after fire [23].

The strength of a fire damaged concrete structure should be investigated after one week of fire, since the compressive strength is reduced a further 20% after cooling [18,22]. The loss of strength of concrete with water cooling below 400°C is more than that of cooling in air. However above 600°C, the effect of cooling regime (either water or air) on the compressive strength is not significant [24]. It is also found that the damage of carbonated concrete heated above 500°C becomes more when cooled with water [25]. In water cooling, the strength loss is even greater with a abrupt drop of the temperature since a part of cement may be hydrated in this case [25]. However, the residual strength of air cooled concrete continues to decrease for a period with some slow recovery [26]. On cooling in the furnace, the strength loss of concrete is lesser as compared to concrete cooled in open air and in water [25,26].

The application of water in fire causes more reduction in compressive strength due to setting up of larger temperature gradients in the concrete [27]. The cooling of heated concrete with tap water for five minutes reduces about 10% more the compressive strength of hot siliceous concrete [28]. Zoldners [29] found that at temperatures up to 500°C, the specimens that were cooled by quenching in water for 5 min had much lower compressive strength than the specimens cooled down slowly overnight in the closed furnace.

3.3 Effect of Rate of Heating and Cooling on the Compressive Strength of Concrete

The higher the rate of heating and cooling, the greater would be the reduction in residual compressive strength of concrete [18,29]. The residual strength is smaller for small temperatures at rapidly heating rate because the matrix does not have the same time to creep when the aggregate expands and the matrix shrinks [18]. The matrix therefore appears to be more brittle giving more micro cracks, when the specimen is heated rapidly. On the other hand for higher temperatures, the rapidly heated concrete is stronger because it takes time for the calcium hydroxide to decompose and damage the concrete. More rapid heating usually occurs near the surface of a structure [18].

For temperatures lower than 600°C, the heating and cooling rates have a significant effect on the residual strength of heated concrete. However, no significant effect had been found when concrete exposed to 600°C and above [24] because most of the micro cracking has been taken place and most of the moisture has been removed up to 600°C and above. If the heated concrete is allowed to cool down faster, the reformation of calcium hydroxide will take place at a slow rate in the days after cooling, and the concrete reaches its minimum strength several days after the fire. The speed of this regeneration process depends upon the moisture content of the ambient air and the size of the structure [18].

3.4 Effect of Loading on the Compressive Strength of Concrete

Under a combination of load and heat, which is likely the case during actual fire, the concrete retains a high percentage of its compressive strength. The presence of compressive stresses in stressed specimens reduces the growth of cracks [22]. It was observed that the strength of the loaded specimens was generally 5-25% higher than those of unloaded specimens during heating [18,22]. The strengths of specimens stressed in compression during heating were not significantly affected by the applied stress level, which ranged from 25-55% of the original strength [22]. The effect of loading should be considered only when the compressive load is known throughout the fire exposure period of the concrete, otherwise it is more conservative to use the values obtained from tests of unloaded specimens instead of the transient values of the compressive strength [6,18].

3.5 Effect of Shape and Size of Specimens on the Compressive Strength of Concrete

The shape and size of the specimens may affect the compressive strength of Portland Cement Concrete at high temperatures. Cubes have greater residual strength than prisms [30]. The temperature at the centre of the larger size specimen will be lower than those of smaller size specimens at the same time due to the delay of the heat transfer. Consequently, the loss of strength would be higher in the smaller size concrete specimen than in the larger size specimens when exposed to same fire duration [30]. However, the effect of specimen size on the retained compressive strength of concrete is not manifested when heated uniformly [31].

3.6 Effect of Long Term Exposure to High Temperature on the Compressive Strength of Concrete

The longer the period of exposure to high temperatures, the greater would be the deterioration in compressive strength due to crack generation and material decay. Most of the reduction occurs within the first 30 days of long term exposure [27,32]. The residual strengths after one hour of exposure at 200, 400, 600 and 800°C were about 80, 70, 60, and 30% respectively while the residual strengths after 2 hours or more exposure were found to be about 70, 60, 45, and 25% [24]. At 300°C, residual compressive strength of about 65% was found after two days and 50% at the end of four months [32].

3.7 General Expressions for Residual Compressive Strength of Fire Damaged Concrete

A simple expression for deterioration of concrete proposed by Hertz, [18] is given as:

$$\xi = \frac{1}{1+\frac{T}{T_1}+\left(\frac{T}{T_2}\right)^2+\left(\frac{T}{T_8}\right)^8+\left(\frac{T}{T_{64}}\right)^{64}} \tag{1}$$

Where ξ is the ratio between the residual compressive strength at a given temperature T°C and the original un-heated compressive strength of concrete at 20°C. The parameters T_1, T_2, T_8, T_{64} with the unit °C are given in Table 4.

The residual compressive strength of concrete can be estimated using the relation given in BS EN [20]. The equation for estimating the residual compressive strength of unstressed concrete reported by Lie, et. al. [33] can be used to calculate the residual strength of fire damaged concrete. This equation is also confirmed analytically and experimentally by Lin, et. al. [34]. The expression for concrete after exposure to fire proposed by Lie, et. al. [33] is given as:

$$f_r = f_{co}(1\text{-}0.001T) \text{ for } 0^oC \leq T \leq 500^oC \quad (2)$$

$$f_r = (1.375\text{-}0.00175T)\, f_{co} \text{ for } 500^oC < T \leq 700^oC \quad (3)$$

$$f_r = 0 \text{ for } T > 700^oC \quad (4)$$

The stress-strain relationship for concrete after exposure to fire may be represented by the following [34]:

$$f_c = f_r\left[1-\left(\frac{\varepsilon_o-\varepsilon_c}{\varepsilon_o}\right)^2\right] \text{for } \varepsilon_c \leq \varepsilon_o \quad (5)$$

$$f_c = f_r\left[1-\left(\frac{\varepsilon_c-\varepsilon_0}{3\varepsilon_o}\right)^2\right] \text{for } \varepsilon_c > \varepsilon_o \quad (6)$$

In which,

$$\varepsilon_o = 0.0025 + (6.0T+0.04T^2)x10^{-6} \quad (7)$$

Where f_{co} is Cylinder strength of concrete not exposed to fire, f_{cr} is Residual strength of concrete after fire, f_c is Concrete stress, ε_c is Concrete strain, ε_o is Concrete strain corresponding to f_{rc}, and T isHighest temperature attained by the concrete.

Chang, et. al.[19] suggested the following temperature-dependent residual compressive strength equation for unstressed concrete.

$$f'_{cT}/f_c{}' = 1.01 - 0.00055T \text{ for } 20^\circ C < T \leq 200^\circ C \quad (8)$$

$$f'_{cT}/f'_c = 1.15 - 0.00125T \text{ for } 200^\circ C < T \leq 800^\circ C \quad (9)$$

Where f'_{cr} is the residual compressive strength of concrete after exposure to temperature is maximum temperature that concrete has been exposed to, before cooling, and f'_c is the concrete compressive strength at ambient temperature (20°C).

The findings of Chang, et. al. [19] were very close to the results of Abrams [22] and BS EN1994-1-2 [20]. The curves for the residual compressive strength of concrete after fire reported by Marchant [21] and Concrete Society Reports TR15 [1] and TR33 [11] could also be helpful in predicting the residual strength of concrete exposed to various temperatures. None of the above relations could predict the exact value of residual strength. However, the combination of all could give a reasonable prediction about the residual strength of fire damaged concrete.

TABLE 4. PARAMETERS FOR CONCRETE WHILE HOT AND IN COLD CONDITION [17]

Type of Aggregates and Conditions of Testing	T_1	T_2	T_8	T_{64}
Siliceous Concrete (Hot)	15000	800	570	100,000
Siliceous Concrete (Cold)	3500	600	480	680
Main Group aggregate Concrete(Hot)	100,000	1080	690	1000
Main Group Aggregate Concrete (Cold)	10,000	780	490	100,000
Light Aggregate Concrete (Hot)	100,000	1100	800	940
Light Aggregate Concrete (Cold)	4000	650	830	930

4. RESIDUAL STRENGTH OF REINFORCING STEEL

The effect of high temperature on the yield strength of typical reinforcing steel bars while hot and after cooling is shown in Figs. 13-14. Significant loss of strength occurs when the reinforcing steel is at high temperature and it is generally responsible for any excessive residual deflection. The yield strength reduces at temperatures above 300°C and retains 50% of the original yield strength at 550°C [8, 11], as shown in Fig. 13. The reduction in yield strength will occur further with increasing temperatures above 550°C.

The original yield strength after cooling is almost completely recovered from temperatures up to 450°C for cold worked steel and 600°C for hot rolled steel, as shown in Fig. 14. However, above these temperatures, there will be a loss in yield strength after cooling [11]. On cooling from 800°C, the yield strength is reduced by 30% for cold worked bars and 5% for hot rolled bars [23]. It is believed that the temperature rise of the reinforcing bars in the sides of the concrete columns is considerably lower than for those at the corners. This could be attributed to the fact that the concrete cover to the side bars remains in position longer than that over the corner bars [35] because the corners of columns heat up quicker. The actual loss in strength depends on the heating conditions and type of steel. However, the conservative values given in Fig. 14, for temperatures up to 700°C, would be sufficient for most purposes [11].

4.1 A Simple Expression for Residual Strength of Fire Damaged Reinforcing Steel

A simple expression for deterioration of reinforcing steel proposed by Hertz [18] is given below:

$$\xi = K + \frac{1-K}{1+\frac{T}{T_1}+\left(\frac{T}{T_2}\right)^2+\left(\frac{T}{T_8}\right)^8+\left(\frac{T}{T_{64}}\right)^{64}} \qquad (10)$$

Where ξ is the ratio between the residual tensile strength of reinforcing steel at a given temperature T°C and the original un-heated tensile strength of reinforcing steel at 20°C. The parameters T_1, T_2, T_8, T_{64} with the unit °C and K are given in the Table 5.

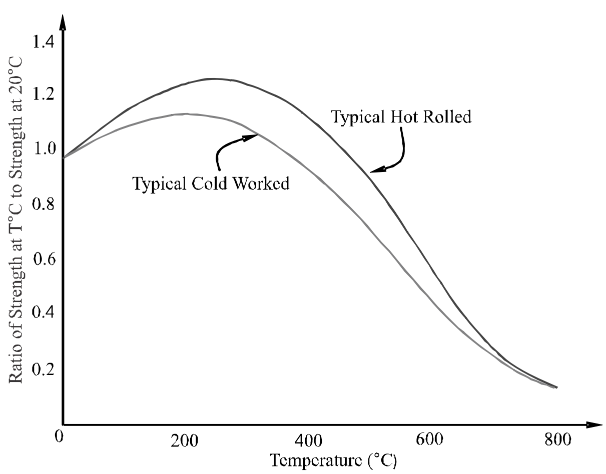

FIG. 13. YIELD STRENGTH OF STEELS TESTED AT ELEVATED TEMPERATURES [2]

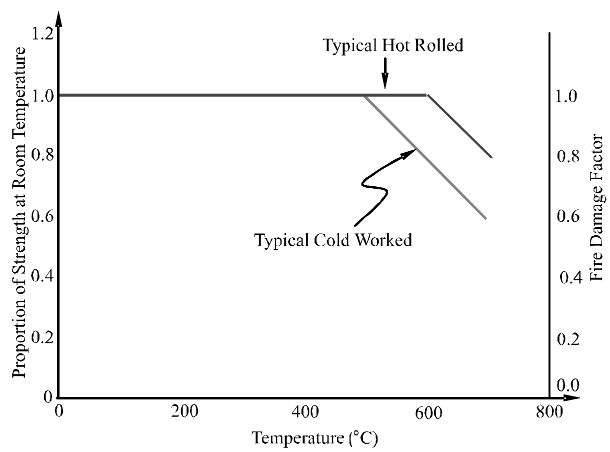

FIG. 14. YIELD STRENGTH OF REINFORCING STEELS AT ROOM TEMPERATURE AFTER HEATING TO AN ELEVATED TEMPERATURE [10]

TABLE 5. PARAMETERS FOR REINFORCING STEEL TESTED IN COLD CONDITION [17]

Type of Steel and Conditions of Testing	K	T_1	T_2	T_8	T_{64}
Hot Rolled Bars, 0.2% Stress	0.00	6000	620	565	11,00
Hot Rolled Bars, 2.0% Stress	0.00	100,000	100,000	593	100,000
Hot Rolled Bars, 0.2% Residual Stress	1.0	100,000	100,000	100,000	100,000
Hot Rolled Bars, 2.0% Residual Stress	1.0	100,000	100,000	100,000	100,000
Cold-Worked Bars, 0.2% Stress	0.00	100,000	900	555	100,000
Cold-Worked Bars, 2.0% Stress	0.00	100,000	5000	560	100,000
Cold-Worked Bars, 0.2% Residual Stress	0.58	100,000	5000	590	730
Cold-Worked Bars, 2.0% Residual Stress	0.52	100,000	1500	580	650

5. CONCLUSION

The residual strength of fire-damaged concrete structural members is a key factor in the decision of reinstating or demolishing of fire damaged concrete structures. The load carrying capacity of reinforced concrete structures depends upon the strength of the main structural members. The evaluation of the residual strength of reinforced concrete column is important from strength point of view because the load carrying capacity of the whole reinforced concrete structures depends upon the strength of reinforced concrete columns. In the evaluation of residual strength of fire damaged concrete structures field testing is involved ranging from simple hammer testing to costly load testing. This research paper facilitates the practising engineers who are involved in the field of repairing and strengthening of fire damaged concrete structures to evaluate the residual strength of fire damaged reinforced concrete columns without involving time consuming and costly field testing.

ACKNOWLEDGEMENTS

The authors acknowledge the financial support offered by HEC (Higher Education Commission), Pakistan. Authors would like to offer special thanks to his supervisor Prof. Colin Bailey, for his technical suggestions and cooperation in this project. Authors are also grateful to the technical staff in the Structural Lab., University of Manchester, UK, especially John Mason, whose contribution in the experimental work is really appreciated.

REFERENCES

[1] Concrete Society Technical Report No.15 "Assessment of Fire-Damaged Concrete Structures and Repair by Gunite", 1978.

[2] CIB W14 Report, "Repairability of Fire Damaged Structures", Fire Safety Journal, Volume 16, No. 4, pp. 251-336, 1990

[3] Ingham, J., "Forensic Engineering of Fire-Damaged Structures", Proceedings of the Institution of Civil Engineers, Civil Engineering, Volume 162, No. 5, pp. 12-17, 2009.

[4] Zhongcheng, M., and Mäkeläinen, P., "Parametric Temperature-Time Curves of Medium Compartment Fires for Structural Design", Fire Safety Journal, Volume 34, No. 4, pp. 361-375, 2000.

[5] Tucker, D.M., and Read, R.E.H., "Assessment of Fire-Damaged Structures", BRE Information Paper IP 24/81, Building Research Establishment, Watford, 1981.

[6] Purkiss, A.J., “Fire Safety Engineering Design of Structures”, 2nd Edition, 2007.

[7] Malhotra, H.L, “Design of Fire-Resisting Structures”, 1st Edition, 1982.

[8] Concrete Society Technical Report No.68, "Assessment, Design and Repair of Fire-Damaged Concrete Structures", Camberley, UK, 2008.

[9] BSI:5268, "Structural Use of Timber, Part-4: Fire Resistance of Timber Structures, Section 4.1: Method of Calculating Fire Resistance of Timber Members", British Standard Institution, London, 1978.

[10] Bessey, G.E., "The Visible Changes in Concrete Mortar Exposed to High Temperatures", Technical Paper No. 4, Part-II, National Building Studies, HMSO, London, 1950.

[11] Concrete Society Technical Report No. 33, "Assessment and Repair of Fire-Damaged Concrete Structures", 1990.

[12] A Cembureau Report Prepared with the Assistance of FIP "Concrete for Fire Resistant Construction", 1979.

[13] Lie, T.T., "Characteristic Temperature Curves for Various Fire Severities", Fire Technology, Volume 10, No. 4, pp. 315-326, 1974.

[14] BS EN-1991, "General Actions-Actions on Structures Exposed to Fire", Parts-1-2, 2002.

[15] BS EN 1992, "Design of Concrete Structures General Rules-Structural Fire Design", Parts-1-2, 2004.

[16] Wickstrom, U., "A Very Simple Method for Estimating Temperature in Fire Exposed Concrete Structures", Fire Technology Technical Report, SP-RAPP, Volume 46, National Testing Institute, Swedish, 1986.

[17] ABAQUS User Manual, 2010.

[18] Hertz, K.D., "Concrete Strength for Fire Safety Design", Magazine of Concrete Research,. Volume 57, No. 8, pp. 445-453, 2005.

[19] Chang, Y.F., Chin, Y.H., Sheu, M.S., and Yao, G.C., "Residual Stress-Strain Relationship for Concrete After Exposure to High Temperatures", Cement and Concrete Research, Volume 36, No. 10, pp. 1999-2005, 2006.

[20] Eurocode-4: BS EN 1994, "Design of Composite Steel and Concrete Structures”, Part 1-2, General Rules-Structural Fire Design, 2005.

[21] Marchant, E.W., "A Complete Guide to Fire and Buildings”, Medical and Technical Publishing, London, 1972.

[22] Abrams, M.S., "Compressive Strength of Concrete at Temperatures to 1600F Fire", PCA, Research and Development Bulletin, 1977.

[23] Report of a Joint Committee of the Institution of Structural Engineers and the Concrete Society, "Fire Resistance of Concrete Structures", 1975.

[24] Mohamedbhai, G.T.G., "Effect of Exposure Time and Rates of Heating and Cooling on the Residual Strength of Heated Concrete", Magazine of Concrete Research, Volume 38, No. 136, pp. 151-158, 1986.

[25] Xiao, J., and Konig, G., "Study on Concrete at High Temperature in China: An Overview", Fire Safety Journal, Volume 39, No. 1, pp. 89-103, 2004.

[26] Lee, J., Xi, Y., and William, K., "Properties of Concrete After High-Temperature Heating and Cooling", ACI, Materials Journal, Volume 105, No. 4, pp. 334-341, 2008.

[27] Neville, A.M., “Properties of Concrete”, 4th Edition. Pearson Prentice Hall, 2007.

[28] Nassif, A., "Post Fire Full Stress-Strain Response of Fire-Damaged Concrete", Fire and Materials, Volume 30, pp. 323-332, 2006.

[29] Zoldners, N.G., "Effect of High Temperatures on Concretes Incorporating Different Aggregates", ASTM, Volume 60, pp. 1087, 1960.

[30] Komarovskii, A.N., "Design of Nuclear Plants", 2nd Edition, 1965.

[31] Arioz, O., "Retained Properties of Concrete Exposed to High Temperatures: Size Effect", Fire and Materials, Volume 33, No. 5, pp. 211-222, 2009.

[32] Kassir, M.K., Bandyopadhyay, K.K., and Reich, M., "Thermal Degradation of Concrete in the Temperature Range from Ambient to 315°C", United States Department of Emergency Report, Washington DC, October, 1996.

[33] Lie, T.T., Rowe, T.J., and Lin, T.D., "Residual Strength of Fire-Exposed Reinforced Concrete Column", ACI, Structural Journal, Volume 92, pp. 153-174, 1986.

[34] Lin, C.H., and Chen, S.T., and Yang, C.A., "Repair of Fire Damaged Reinforced Concrete Columns", ACI, Structural Journal, Volume 92, No. 4, pp. 406-411, 1995.

[35] Thomas, F.G., and Webster, C.T., "The Fire Resistance of Reinforced Concrete Columns Investigations on Building Fires", National Building Studies Research Paper No. 18, Part-VI, 1953.