Performance of repaired fire affected RC beams

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Structural concrete is one of the most commonly used construction materials in the world. Concrete elements exposed to fire experience temperature gradients and, as a result, undergo physical changes or spalling, thereby exposing steel reinforcement. The structural property of concrete that has been most widely studied as a function of temperature exposure is compressive strength. Relatively few studies have been undertaken on flexural strength of reinforced cement concrete (RCC) beams and their repair. Therefore, this study was carried out to generate experimental data on residual flexural strength of heated RCC beams and their strengthening using various repair techniques. A total of 25 RCC beams were cast with similar cross-sectional details, length and grade of concrete and clear cover. Twenty beams were meant for fire exposure and the remaining five were used as companion beams. The beams were heated in two stages. In the first stage, two beams were kept at each temperature for 3 h between 100°C and 1000°C, in increments of 100°C. Beams exposed to temperature ranging between 100 and 500°C were repaired by applying paint. The beams exposed to temperature ranging between 600 and 1000°C were repaired for spalling. In the second stage, all repaired specimens were again heated. These test specimens were tested for flexural strength after bringing them to room temperature. The variation of flexural strength of repaired RCC beams with increase in temperature has been studied and the flexural strength of beams before and after the repair was compared.

Keywords: Fire exposure, flexural strength, paint, repair technique, temperature gradient.

CIVIL and structural engineers are often not familiar with fire safety issues, and fire protection design is currently being overlooked by structural consultants in building projects. However, the structural design codes recommend nominal cover, minimum dimensions, etc., for fire-exposed structural elements.

In recent years many engineers find themselves increasingly involved in structural fire resistance decisions. After the tragic World Trade Center, Pentagon events of 11 September 2001, and the nightclub fire in West Warwick, Rhode Island, USA on 20 February 2003, building owners and professionals pay more attention to fire safety in general and structural fire resistance in particular. The field quality of fire protection materials is now getting deserved attention. It is a positive trend that engineers work closely with architects and fire engineers to produce 'fire-safe' structures, and the responsibility for fire-resistant design is delegated to the structural engineers.

The study of the behaviour of concrete at elevated temperatures and rehabilitation and repair of structures has assumed great importance in recent times because the accumulated annual loss of life and property due to fires is comparable to the loss caused by earthquakes and cyclones. This necessitates development of fire-resistant design and proper repair of damaged structures. A brief review of the existing literature on the behaviour of reinforced cement concrete (RCC), and its constituents under elevated temperatures considering parameters like compressive strength, flexural strength, temperature distributions and other properties is presented.

Hansen and Jansen[1] conducted three series of beam tests at the Norwegian Fire Research Laboratory. The test specimens were reinforced and prestressed concrete beams having dimensions of 150 × 200 × 2850 mm. Three types of concrete were used: ND (normal density concrete), LWA (lightweight aggregate concrete – Liapor aggregate), and LWAF (lightweight aggregate concrete with s-Fibrin type 1823). The concrete were designed to have target 28-day cube (100 × 100 × 100 mm) strengths of 75 and 95 MPa. It was concluded that severe spalling (exposed reinforcement) occurred more in higher strength lightweight aggregate beams as observed in reinforced and prestressed LWA75 beams, while spalling without exposed reinforcement occurred more in high-strength, normal weight ND beams. Shallow or no spalling was observed in higher strength lightweight concrete beams with LWAF. No spalling was observed in lightweight beams with s-Fibrin type and with passive protective coating (LWAFP75).

Sanjayan and Stocks[2] conducted fire tests on two full-scale *T*-beams, one made of high-strength concrete with silica fume (105 MPa) and the other made of normal-strength concrete (27 MPa). It was observed that high-strength concrete appeared to be more prone to spalling in fire than normal-strength concrete. Spalling occurred in 200 mm thick flange where the cover to the steel was large (75 mm). In the 150 mm thick flange, the cover was 25 mm and there was no spalling. No spalling occurred in the web, possibly because in the web the concrete was exposed on three sides and therefore, moisture can escape through all the exposed surfaces. The higher moisture content found in high-strength concrete indicated that it had slower drying rate than normal-strength concrete.

Kumar *et al.*[3] investigated the behaviour of RCC beams cast with M20-grade concrete after exposure to elevated temperatures for 1, 1.5, 2 and 2.5 h duration. The results showed that M20 RCC beam with clear cover 25 mm under its own weight was unable to resist a fire exposure of

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about 2.5 h as it failed in serviceability criterion. Spalling of concrete was observed at many places, which increased further with time. Even 2 h fire exposure was critical for such a beam as it withstood about 50% load of the companion beam. M20 RCC beam behaved reasonably satisfactorily when exposed to fire for 1 h duration, as its strength was found to be about 83% of the companion beam.

Crozier *et al.*[4] conducted tests to establish the residual bending strength and residual shear strength of reinforced high-strength concrete beams, following exposure to temperatures between 200 and 800°C. From this investigation, it was concluded that the degradation of bending strength was relatively minor at or below 400°C. The degradation of bending strength was not as severe as degradation of concrete compressive strength. More residual strength was gained from a beam in which top reinforcement was provided. The degradation of the bending strength was a combination of concrete compressive strength reduction and degradation of the strength of the tensile reinforcement. Significant reduction in residual shear strength occurred at temperatures beyond 200°C. The degradation of concrete compressive strength significantly influenced the shear strength.

Ellingwood and Lin[5] studied the flexure and shear behaviour of reinforced concrete beams during fire. The results showed that all beams developed shear cracks as early as 90 min after the start of the fire. Flexural cracks formed in the positive moment region approximately 30 min later and extended rapidly; as a result, all the beams failed in flexure rather than shear. The most important factor affecting the behaviour of a properly designed reinforced concrete flexural member is the temperature history of the reinforcement.

In the present work, the flexural strength of repaired fire-affected RCC beams was evaluated after exposing them to elevated temperatures for 3 h duration.

The reinforced concrete beams were 1200 mm long and 112 × 240 mm in cross-section. They were designed as under-reinforced beams[6], conforming to IS 456–2000. The concrete mix proportion was 1 : 1.296 : 3.33 with water cement ratio of 0.48. The cement was Ordinary Portland Cement (OPC)[7], conforming to IS 8112-1989. The fine aggregate was natural river sand[8], conforming to zone III of IS 383-1970. The coarse aggregate was crushed hard blue granite passing through IS 20 mm sieve and retaining on 4.75 mm sieve. Potable water with pH value 6.72 was used.

High Yield Strength Deformed bars (Fe415) conforming to IS 1786-1985 were used as longitudinal reinforcement[9]. Each beam was reinforced with three 10 mm diameter bars at the bottom face and two 10 mm diameter bars as hanger bars. Fe250-grade steel bars of 6 mm diameter were provided in the form of two-legged stirrups at a spacing of 60 mm centre-to-centre, throughout the span. A clear cover of 15 mm was maintained throughout for all the beams using cover blocks. The reinforcement detailing is shown in Figure 1.

Twenty beams were meant for high-temperature exposure and the remaining five were used as companion beams. These beams were demoulded after 24 h and were cured in a water tank for 28 days. After 28 days of curing, all the beam specimens were stored under laboratory air-drying conditions prior to high-temperature exposure or for load test. Cubes were also cast along with each batch and were demoulded the next day. Then these were stored under water at room temperature until tested at the age of 28 days. The average 28 days cube compressive strength of concrete at room temperature was 30.1 N/mm^2.

To determine the flexural strength after exposure to elevated temperatures, the beams were heated in a furnace. After exposing to specified temperatures of 100–1000°C for 3 h, the beams were cooled to room temperature. Later, these beams were tested for flexural strength on a loading frame.

A custom-built electric furnace was used to heat the RCC beams under study (Figure 2). The heating arrangement in the furnace is as according to ISO 834 specifications[10]. The furnace is designed in two parts. One part can be pulled out after loosening the screws provided for tightening the two parts together. It rests on wheels at its bottom and can be moved to the required location by moving it on rails. It also consists of a bed of refractory bricks of 110 mm size, on which the beam is to be placed.

The second part is the heating chamber. This chamber measures 650 mm in height, 500 mm in width and 1800 mm

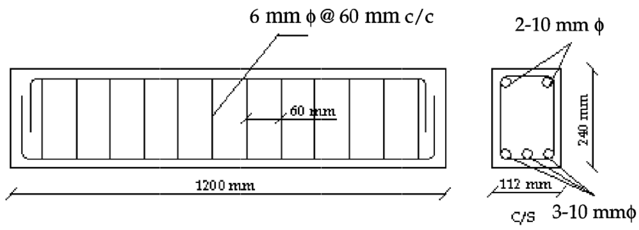

Figure 1. Reinforcement detailing of beams.

Figure 2. Furnace for heating beams.

in length, with the sides and top lined with electrical heating coils embedded in refractory bricks.

Temperature in the furnace is controlled by means of a dedicated control panel which houses the power supply and circuit switches for the furnace. The control panel also has an oven temperature controller to prevent damage to the furnace by tripping out, if the temperature inside the furnace exceeds the specified temperature. The maximum operating temperature of the furnace is 1000°C. Air vents are also provided at the top, to allow vapours to escape from the heating chamber and to allow any thermocouple leads, if provided, to be connected to a digital temperature logger.

The test specimens were subjected to temperatures from 100 to 1000°C, at 100°C intervals, for 3 h duration at each temperature. The beams were heated individually to the specified target temperatures.

The weight of the beam was noted before exposure to any temperature. Then the beam was placed on the platform of the moveable part of the furnace. The beams were exposed to temperatures in such a manner that only the top and side faces were exposed to heat. The cast surface of the beam rested on refractory bricks. Each of the target temperatures was set on the control panel after the beam was placed inside the furnace. Following attainment of the desired temperature, the exposure continued for 3 h. Then the furnace was switched-off and the beam was taken out of the furnace and allowed to cool to room temperature. The weight of the beam after cooling to room temperature was noted.

Two types of repair techniques were adopted for the fire affected specimens.

Type-I: Application of heat-resistant paint – When no spalling was observed for exposure to 100–600°C, heat-resistant paint was used. The surfaces of the beams were cleaned thoroughly so that there was no dust or oil stains. Two layers of Brushbond TI Flexicoat were applied on all the faces. The cubes were also repaired along with the beams.

Type-II: Repair of beams with repair materials – This procedure is adopted for the specimens where spalling was observed in the temperature range 600–1000°C. The cover concrete provided for the beams was removed on all sides using a demolishing chipper until the reinforcement was exposed. Then the surface was thoroughly cleaned first by water jetting and then by sand blasting till the surface is made clean without any loose material. After the surface was thoroughly cleaned, a primer (Nitobond SBR) was applied on the wet surface. This was applied to provide good bonding between the substrata and the new material. After the primer was applied, the specimen was placed in the mould again. The gaps between the specimen and mould were filled with cement sand mortar (1 : 3) with Nitobond AR. The cement mortar was then hand-compacted into the moulds with the help of a tamping rod. After tamping was thoroughly done, the top layer was wiped-off. The specimens were demoulded and cured for 28 days.

Each of the 25 beams was tested for flexural strength under static two-point loading set-up. The load was applied through a hydraulic jack in stages of ten divisions in the proving ring, until failure.

In this investigation, the physical changes and weight losses of the beams due to heating and the effect of temperature on residual flexural strength have been studied. The repaired beams exposed to fire were also studied.

The following physical changes due to heating were studied. (i) Thermal crack pattern: Surface cracking of the beam specimens was the most obvious sign of physical damage resulting from high temperature exposure. Thermal cracks were the result of tensile stress induced by differential movement of moisture. This differential movement was the result of higher moisture loss in the outer layers of the test specimens compared to the inner layers. Up to 400°C, there were no significant changes in the beam. However, between 500 and 700°C thermal cracks developed. At 800°C, thermal cracks of width 0.2 mm were observed on the sides and top surface of the beam. However, 0.4 mm wide cracks were observed at 900°C and 0.5 mm cracks were observed at 1000°C. These cracks developed on all faces of the beam between 900 and 1000°C. Severe spalling of the concrete in the cover portion was observed in the beam exposed to 1000°C. The concrete could be chipped-off from the beam even with a slight touch.

(ii) Colour change: Visual examination of the test specimens revealed that the colour of the concrete gradually lightened as the temperature of exposure increased. Beams exposed to temperatures 800–1000°C became red hot in colour. Later, on cooling, the beams turned into pale white colour.

Figure 3 shows the variation of weight loss in the beams with temperature. No weight loss was observed at 100°C, and weight loss increased as the temperature increased to 1000°C. At elevated temperatures, the formation of cracks allows additional passages for the vapours to dissipate and thus reduces the weight of the beams. This disintegration process was most pronounced in the beam exposed to 1000°C temperature. The process is attributable to the breakdown of the calcium hydrosili-

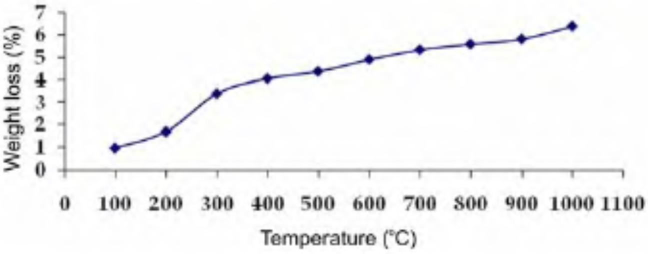

Figure 3. Weight loss vs temperature.

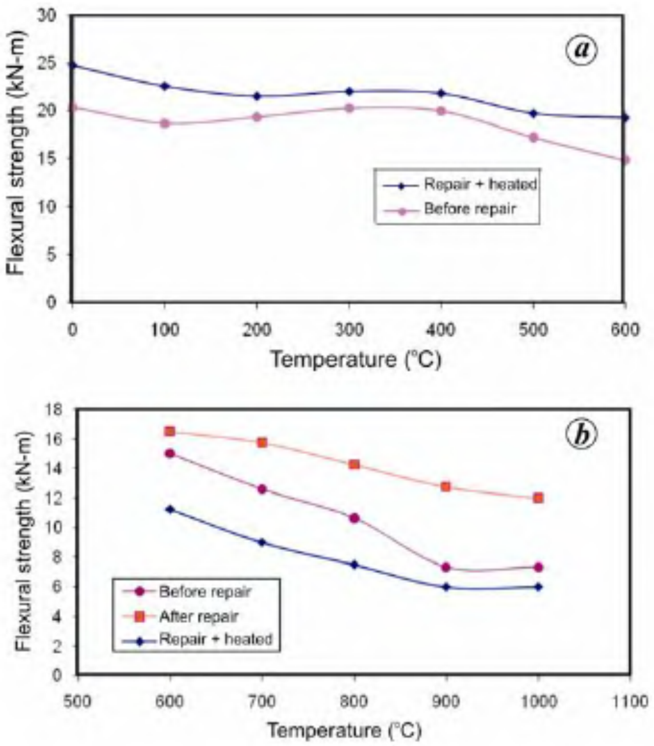

Figure 4. Flexural strength vs temperature. ***a***, Beams repaired up to 600°C by using Type-I technique. ***b***, Repair of beams from 600 to 1000°C using Type-II technique.

cates and the dehydration of chemically bound water from lime. The increase in weight loss contributes to reduction in flexural strength.

Figure 4 *a* and *b* shows the flexural strength of all the heated beams plotted against temperature. In the temperature range 100–600°C, repaired beams after exposure to temperature exhibited improvement in flexural strength compared to companion beams exposed to temperature. The flexural strength of companion beams and the repaired beams exhibited reduction in strength with increase in temperature up to 1000°C. The decrease in flexural strength was almost linear beyond 600°C. However, the flexural strength of all the heated beams beyond 600°C was found to be lower than that of companion beams. It was noticed that the unheated companion beam failed at a load of 135 kN, while the beam exposed to temperatures from 100 to 600°C failed at 123, 117.5, 120,119, 107.5 and 100 kN respectively. The repaired heated beams failed at 105, 95, 85 and 80 kN respectively, at room temperature and the same repaired beams when exposed to 600–1000°C failed at 75, 60, 50, 40 and 40 kN respectively. This can be attributed to the failure of the bond between the substrate and repair material. The type of technique adopted and the repair material used have failed between temperatures 600°C and 1000°C.

The following conclusions have been drawn from this investigation.

1. The normal-strength concrete could sustain up to 500°C without any visible distress.
2. The weights of the beams were reduced when heated to higher temperatures. The loss of water at elevated temperatures contributes to a decrease in flexural strength.
3. Visible thermal cracks were formed beyond 600°C and they increased in width at increasing temperatures. Spalling of cover concrete was observed in beams exposed to 1000°C before repair. Severe spalling of concrete was observed in the temperature range 700–1000°C, in beams which were heated after repair.
4. The beams exposed to temperatures between 100 and 500°C failed in flexure. Shear cracks were predominant and the beams failed in shear beyond 600°C.
5. The residual strength in the beams between temperatures 100 and 400°C was unclear. The strength was found to decrease almost linearly beyond 400°C.
6. Increased flexural strength was observed at temperatures between 100 and 600°C in the case of beams repaired by applying paint.
7. Unheated, repaired beams exhibited improved flexural strength. They failed to show the same performance when exposed to 600–1000°C.
8. Within the experimental limitations it can be concluded that application of heat-resisting paint up to 600°C performs better. Repair technique adopted beyond 600°C was found to be beneficial under room temperature, but failed under heat after repair.

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Assessment of intra-specific variability at morphological, molecular and biochemical level of *Andrographis paniculata* (*Kalmegh*)

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In the present investigation, 15 *Andrographis paniculata* genotypes collected from Chhattisgarh and adjoining states were grown in the field for the variability studies. Three different variability parameters namely morphological, molecular and chemotyping of active ingredient content were employed. Wide variations were recorded with regard to quantitative characters and RAPD profile. Ten polymorphic RAPD primers produced a total of 37 amplicons, which generated 70.27% polymorphism. The number of amplified products ranged from 2 to 7 for different primers, whereas the percentage of genetic similarity for the studied primers ranged from 51.4 to 97.0. There were two major clusters formed in the genotypes studied. The andrographolide content ranged from 0.69 to 1.85% and the genotype KI-2 had the highest estimated content. The study demonstrated that simultaneously morphological, molecular and biochemical analysis are useful for characterizing genetic diversity and defining relationships between *kalmegh* germplasm. It also gave possible indications to the phytochemical variation of different genotypes which were due to the genetic differences.

Keywords: *Andrographis paniculata*, andrographolide content, genetic diversity, germplasm, RAPD.

ANDROGRAPHIS PANICULATA (*Kalmegh*) is an erect growing annual medicinal herb and well known for its multiple health-promoting properties. It is found throughout India and other Asian countries namely China, Java, Thailand, etc. It was credited a wonder drug in 1919 for arresting the spread of the contagious disease 'global flu' epidemic. *Kalmegh* is also reported to possess antihepatotoxic[1], antibiotic[2], antimalaria[3], antihepatitic[4], anti-inflammatory[5] and anti-snakevenom[6] properties to mention a few, besides its general use as an immune stimulant agent[7]. It is used as a wonder drug in traditional Siddha and Ayurvedic systems of medicine as well as in tribal medicine in India and several Asian countries for multiple clinical applications.

A recent study conducted at Bastyr University, USA confirms anti-HIV activity of andrographolide[8]. It is widely distributed and exploited as medicinal plant in almost all regions of India (Himachal Pradesh, Andhra Pradesh, Chhattisgarh, Madhya Pradesh, etc.). It is placed at 17th position among the 32 prioritized medicinal plants of India with a demand of 2,197.3 tonnes (2005–06) and annual growth of 3.1%[9]. Several active components have been identified; two of them are andrographolide and neoandrographolide. Andrographolide (the diterpenoid lactones) is the main bitter principle found in high concentrations in the leaves of *kalmegh*[10]. Andrographolides are extracted for their use as drug. The aim of the present study was to find out whether phenotypic variations in the genotypes grown in different locations are merely epigenetic or genetic. Furthermore, if the same are genetically different, then their medicinally active principles have to be estimated to find out possible quantitative variation, which accord for the variation in medicinal activity of the plants collected from different locations. In general, genetic diversity can be measured at any functional level from blueprint (DNA) to phenotype[11]. In this study, possible genetic variation among *A. paniculata* genotypes collected from different locations was analysed using RAPD (random amplified polymorphic DNA) to

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