PERFORMANCE OF BEAM COLUMN JOINT EXPOSED TO ELEVATED TEMPERATURE

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ABSTRACT

This study presents the experimental investigations carried out onthe effect of elevated temperature at the reinforced beam-column joint and shows the performance of the joint at elevated temperature. In this research the capacity of the joint was determined by load test at normal conditions and after heating the sample at elevated temperature. The sample was made according to the strong column weak beam concept, so that the plastic hinge develops at the joint of the beam. 150mm×150mm of beam of 900mm length with 200mm×225mm of column of 1500mm length was casted. The ultimate strength of the sample was determined by aplying increasing load on the beam. Load was applied at the end of the beam by using hydraulic jac. Deflection, temperature at different location and load applied on the beam was measured. The ultimate strength of the beam at normal condition was about 30 kN and after heating the ultimate strength of the joint was about 20 kN. The deflection of the beam at ultimate strength was 15mm for normal condition and 20mm at ultimate strength after heating at elevated temperature. After heating the sample there was hairline crack on the beam column joint. The furnace temperature was about 450^{0}C at the bottom of the beam where the surface temperature of the sample was about 400^{0}C. the temperature inside the concrete was about 120^{0}C at a depth of 75mm at beam and 125mm in the column. The location of sensor was in the middle of the beam and the column section. The heat genarates from the burner increases the temperature gadually. But for the ventilation of the furnace the maximum temperature was about 400^{0}C. the inside temperature of the concrete was about 30% of the surface temperature. The decease of temperature was very high at a small depth. From the results, it was found that the joint capacity of the sample reduces about 33% after heating at 400ºC for two hours. The reduction of the strength was less then 40%, so the concrete was not dead concrete and it can be used after heating. The cracking pattern of the beam was vertical from the top of the beam. The failure was bending failure as there was no shear crack the beam. Though there was small cracks on the beam after heating for two hours but the deflection was in aceptable limit. Failure was at the beam column joint as expected. Because according to the concept of strong column weak beam phylisophy there develpos a hing at the joint. The elasticity of the beam also increase as the deflection of the beam inceases after heating with respect to the deflection of the beam at normal condition. The volume of the sample also increses as the applied load on the top of the column increses with respect to time a the time of heating autometically. Data from the research can be used for making relationship with the thermal properties of the concret to the performance of the beam column joint. Principal effects due to elevated temperatures are loss in compressive strength, loss in weight or mass, change in colour and spall of concrete. The experimental results of normal concrete subjected elevated temperatures at 200°C, 400°C and cooling regimes viz. air cooling at normal temperature on concrete are reported in this paper. The main concern points which are elasticity of the beam and the volume expansion of the sample need to be research for getting the suitable explanation of the joint performance.

Keywords: *Fire, Beam column joint, strength, concrete, temperature.*

1. INTRODUCTION

Fire accidents have been one of the most common hazards worldwide in recent times. Numerous fire incidents of varying sizes and causes occur regularly. As a result, there is increased concern regarding structural safety both before infrastructure is built and during reinstatement or reconstruction after a building fire hazard.

For civil infrastructure, concrete is one of the main loads carrying parts of reinforced concrete structures. So, it is very important to investigate the mechanical properties (such as compressive strength, tensile strength, and ultimate strength of the component. As the beam-column joint is the most important portion to transmit the load from beam to column, so it is necessary to investigate the performance of the joint.

At the time of a reinforcement concrete structure element to high temperature due to good fire-resistant properties of concrete, it is often possible for the structure to withstand. But at the same time, concrete changes its chemical composition, physical structure, and water content. This means that in dealing with such situations, a choice may need to be made between reconstruction, reinstatement, or repair. Among them, reinstatement can be often a quicker and cheaper solution. However, before taking any of the above solutions for such treatment, it is necessary to investigate and establish whether the damage structure is suitable for such treatment or not(Rashid et al., 2019). To do this, more attention should be paid to the mechanical properties of concrete at high temperatures or to the residual properties of the concrete after exposure to high temperatures. During exposure to high temperatures, its residual capacity such as residual bond capacity, compressive strength, tensile strength, etc. for structural performance must be assessed, which requires knowledge of the properties of steel and concrete and of the bond between them at different elevated temperatures which the reinforced concrete experienced in the fire hazard (Anderberg 1997).

The behavior of a concrete structural member exposed to fire is dependent on thermal, mechanical, and as well as on deformation properties of concrete of which the member is composed. Concrete's thermo physical, mechanical, and deformation characteristics alter significantly within the temperature range linked to building fires, much like other materials do. These qualities rely on the features and composition of the concrete and change as a function of temperature. At high temperatures, the qualities of concrete are significantly influenced by its strength. Compared to Normal Strength Concrete (NSC), the characteristics of High Strength Concrete (HSC) change with temperature in a different way. For mechanical properties, which are influenced by porosity, strength, moisture content, density, heating rate, and quantity of silica fume, this variation is more noticeable(V. Kodur, 2014).
As temperatures rise, the thermal conductivity gradually drops. The moisture content and permeability of the concrete mix have a significant impact on this decrease. The variations in moisture content that occur with rising temperatures can be correlated to decreasing thermal conductivity (Khaliq and Kodur 2011).

The beams exposed to the hydrocarbon fire had the lowest fire resistance, which is roughly 25 minutes less than the beams exposed to the ASTM E119 standard fire exposure(V. Kodur & Dwaikat, 2008).
Researchers compiledmeasured specific heat of different concretes from various studies. Various studies, based on test results and different standards, shed light on how the specific heat of normal-strength concrete (NSC) changes with temperature. Concrete's specific heat tends to stay constant up to 400 degrees Celsius, then it starts increasing until around 700 degrees Celsius, after which it stabilizes between 700 and 800 degrees Celsius. The specific choice of aggregate plays a crucial role in determining concrete's specific heat, alongside other factors. The relations specified by ASCE, following ASTM C128, take into account how specific heat is affected. When it comes to concrete with carbonate aggregate, the absorption of a significant amount of energy during the decomposition of dolomite leads to an endothermic process, resulting in a higher specific heat in the temperature

range of 600–800 degrees Celsius. This increased heat capacity in concrete with carbonate aggregate proves to be beneficial(V. Kodur, 2014).
The variation in the mass of concrete as a function of temperature for concretes made with carbonate and siliceous aggregates has great influence in the strength of concrete. The mass loss is minimal for both carbonate and siliceous aggregate concretes up to about 6000C. However, the ingredients of aggregate have significant influence on mass loss in concretes beyond 6000C.Inthecase of concrete containing siliceous aggregate, mass loss is insignificant even above 6000C. However, beyond 6000C, concrete containing carbonate aggregate experiences a larger percentage of mass loss. This higher percentage of mass loss in carbonate aggregate concrete is happens due to dissociation of dolomite in carbonate aggregate at around 6000C (V. Kodur, 2014).

At high temperatures, there is a noticeable difference in the mechanical properties of concrete, particularly compressive strength. The use of various heating or loading rates, specimen size and curing, testing condition (moisture content and specimen age), and additive usage may all be responsible for the variances among experiments(V. Kodur, 2014).

For normal strength concrete (NSC), exposure to temperatures up to 400°C has a minimal impact on its compressive strength. NSC typically exhibits high permeability, facilitating the easy diffusion of pore pressure caused by water vapor. In contrast, high-strength concrete (HSC) employing various binders develops a superior and compact microstructure with reduced calcium hydroxide content, leading to a positive influence on compressive strength at ambient temperatures(V. Kodur, 2014). The best results for increasing compressive strength at room temperature are obtained using binders like slag and silica fume, which is related to a dense microstructure(Wasim Khaliq and Venkatesh Kodur, 2012). As was previously mentioned, the compact microstructure is extremely impermeable and becomes harmful at high temperatures because it prevents moisture from escaping. This causes pore pressure to build up and causes microcracks to form quickly in the HSC, which accelerates the deterioration of strength and the occurrence of spalling. Steel fibers in concrete contribute to a reduction in strength loss at high temperatures(Poon et al., 2001).

The tensile strength of concrete is much lower comparative to the compressive strength, and hence tensile strength of concrete is often neglected in strength calculations at room temperature. However, from fire resistance point of view, it is very important, because cracking in concrete occurs generally due to tensile stresses and the structural damage of the member in tension is often generated by progression in micro cracking. Under fire conditions, tensile strength of concrete can be even more crucial in cases where fire induced spalling occurs in concrete member. Thus, properties of tensile strength of HSC, which varies with temperature, is crucial for predicting fire induced spalling in HSC members(Mehta, P Kumar and Monteiro, 2014).

The decrease in tensile strength of NSC for increase in temperature is due to weak microstructure which allows initiation of micro cracks. After reaching 300°C, concrete loses about 20% of its initial tensile strength. Eventually above 300°C, the tensile strength of NSC decreases at a rapid rate and reaches to about 20% of its initial strength at 600°C(V. Kodur, 2014).

High Strength Concrete experiences a rapid loss of tensile strength due to higher temperature(Chan et al., 1999).Concrete's tensile strength is increased when steel fibers are added, and at room temperature, the increase can reach 50%. Furthermore, over the entire temperature range of 20 to 800°C, the tensile strength of steel fiber-reinforced concrete declines more slowly than that of ordinary concrete. This enhanced tensile strength is very helpful when the member is bent, as it helps slow the spread of cracks in steel fiber-reinforced concrete structural members(Purkiss, 1984).

The review of the literature reveals contradictory information about the occurrence of fire-induced spalling as well as the specific process causing it in concrete. While some studies found that concrete structural elements subjected to fire experienced explosive spalling, while other investigations found little to no spalling at all. The many variables that affect spalling and their interdependencies could be one reason for this surprising trend of results. But the majority of studies concur that excessive

temperatures and low permeability of concrete are the primary causes of fire-induced spalling in concrete(V. Kodur, 2014).
It seems that pore pressure builds up while heating is what causes spalling(V. K. R. Kodur, 2000). The exceptionally high density and compactness (as well as poor permeability) of greater strength concrete prevents the extremely high-water vapor pressure that is created during fire exposure from escaping. Concrete particles break out of the structural member when the effective pore pressure is higher than the concrete's tensile strength. According to this theory, pore pressure is what causes progressive failure. Therefore, the more fire-induced spalling, the lower the concrete's permeability. Depending on the fire and the properties of the concrete, this falling-off of chunks of concrete can frequently be explosive(Anderberg Y, 1997).

According to this hypothesis, compressive forces parallel to the heated surface emerge as a result of restricted thermal dilatation at the heated surface, which causes spalling. Concrete spalling, or brittle fracture, releases these compressive forces. An important factor in the commencement of instability that emerges as explosive thermal spalling is the pore pressure(Hertz, 2003).

All concretes may spall, but because of its limited permeability and low water-to-cement ratio, high-strength concrete is thought to be more prone to it than normal-strength concrete. Because of the high density (and poor permeability) of HSC, the high-water vapor pressure created by a sudden temperature increase cannot escape, and this pressure build-up frequently surpasses the saturation vapor pressure. The pore pressure can reach up to 8 MPa at 300°C; the HSC mix, which has a tensile strength of about 5 MPa, is frequently unable to withstand such high internal pressures(Noumowe et al., 2009)(Boel et al., 2008).

Strong pressure gradients at the surface result in the so-called "moisture clog" due to the limited permeability of concrete and the drained conditions at the heated surface. Large pieces of concrete break off from the structural part when the vapor pressure is higher than the concrete's tensile strength. It has been discovered via several test observations on HSC columns that spalling frequently has an explosive character. Therefore, one of the main issues with using HSC in building applications is spalling, which needs to be appropriately taken into consideration when assessing fire performance (Boel et al., 2008).
The extent of spalling depends on a number of factors including strength, porosity, density, load level, fire intensity, aggregate type, relative humidity, amount of silica fume, and other admixtures. Many of these factors are interdependent and this makes prediction of spalling quite complex. The variation of porosity with temperature is the most important property needed for predicting spalling performance of HSC(V. Kodur, 2014). Noumowé et al., (2009) carried out porosity measurements on NSC and HSC specimens, using a mercury porosimeter, at various temperatures.

In this study, the performance of the beam-column joint was studied by determining the capacity of the beam at normal conditions and after exposure to high temperatures. This evaluation of the percent of deterioration of the concrete properties will help engineers to decide whether the structure after being exposed to high temperature can be repaired rather than required to be demolished(Mahmud et al., 2021).

2. METHODOLOGY

To find out the effect of beam-column joint performance at elevated temperatures, 2 specimens were cast and tested. The first specimen was tested to measure the ultimate capacity of the beam-column joint at room temperature and the other specimen was heated for 2 hours at elevated temperature to measure the ultimate capacity of the beam-column joint at elevated temperature. The following flow diagram is followed for the experiment.

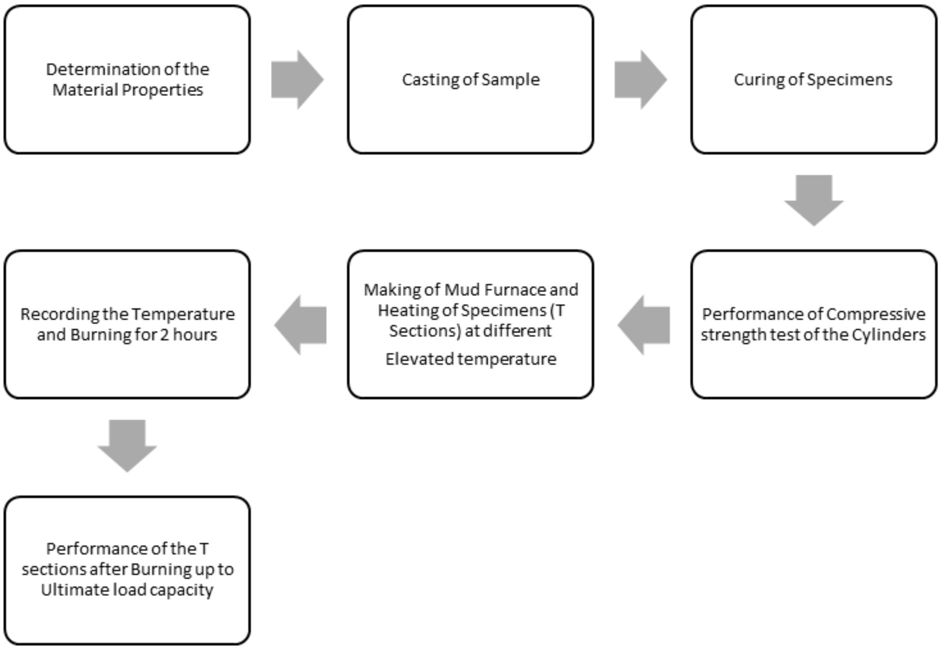

Figure 1: Flow diagram of the experimental process

2.1SpecimenDetails

Materials used for the specimen were stone, Kushtia sand, and Ordinary Portland cement reinforcement of 500W. Concrete for the specimen was prepared with the mixing ratio of 1:1.25:2.5 and w/c of 0.46(ACI-318 n. d., 2008).

The sprecimen wasdesigned with stong column and weak beam concept. For this beam column joint the rigid jone for beam was 4.5 inch inside the column which makes the speiment to fail in the beam due to deflection.

The T-type specimen was built. It was a Beam Column joint. The cross-section of the Beam was 6”×6” and theColumn cross-section was 8”×9”. The length of the Beam and Column was 6’8” and 5’ respectively. The Beam andColumn were cast monolithically.

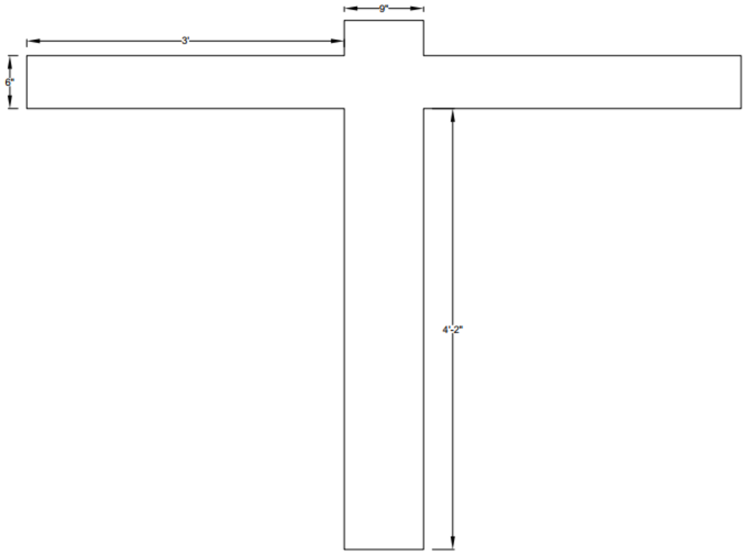

Figure 2: Specimen dimentsions

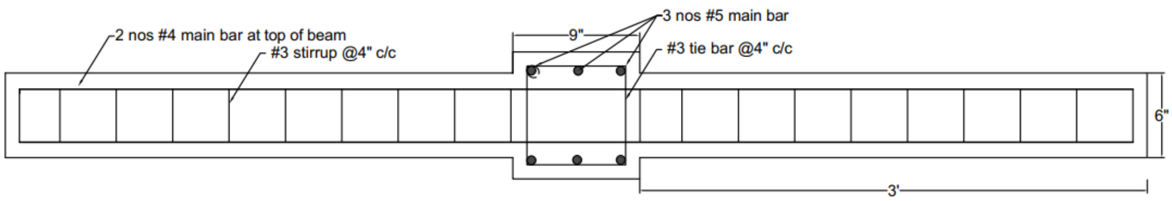

Figure 3: Reinforcement Detailing of Beam column joint (top view).

2.2Experimental setup

A mud Furnace was constructed to burn the specimen on that. It was constructed on a loadtransfer frame as the working load was applied at the time of firing. For constructing the mud In the furnace there was brick, mud, and brick shurki were used. Figure 4illustrates the making procedure of the mud-brick furnace. The height of the furnace wasabout 5 ft and the length of the furnace was about 4 ft. The width of the furnace wall was about10 in. The front of the furnace was left open. After placing the specimen in the furnace, the frontwas closed by using a steel sheet and tied with bamboo to make the furnace enclosed. Therewere few small openings in the furnace to supply enough Oxygen to make similarities with the real fire conditions.

2.3 Temperature Measurement and Load Application

A thermocouple was inserted into the specimen at different locations. There were two types of thermocouple used. One was glass braided and another was a normal thermocouple. Data was they were recorded by using a data logger. A thermocouple was placed at the surface of the specimen and inthe middle of the specimen depth.

Table 1: Location of thermocouples in the specimen

Location Number	Location name	Location	Thermocouple Type
1	1 External	12” Bottom from the beam-column joint, 4’’ inserted	Glass Braided
	1 Internal		Normal K-Type
2	1 External	At the center of the beam-column joint, 4’’ inserted	Glass Braided
	1 Internal		Normal K-Type
3	1 External	6” left from the beam-column joint, 3’’ inserted	Glass Braided
	1 Internal		Normal K-Type
4	1 External	6” right from the beam-column joint,	Glass Braided
	1 Internal		Normal K-Type

2.4Loading and Heating of Specimen

Axial load was applied on the column using a Hydraulic Jack. It was 20% of its capacity. For Heat insulation Glass wool was used around the column on the top surface of the specimen. A constant load of 40% of the Beams capacity was applied through the period of heating. Glass wool was used to protect the linear variable displacement transformer (LVDT). The load was kept constant throughout the time of heating.

A preliminary drying process was carried out for 1 hour at a temperature of 120°C in the furnace before heating the samples in the furnace. This was done to reduce the internal water content. During the time of preheating the temperature was about 100°C at the surface of the specimen. It was controlled by the regulator in the burner. At the time of preheating the load was not applied to the specimen.

For heating the specimen, the furnace was insulated using glass wool to make the furnace airtight. So that the heat does not evaporate and the desired temperature can be achieved. The gas Burner was for firing. The heating process was performed according to the rate of heating in the ASTM E119 time-temperature curve(ACI 216.1., 2007). The heating duration was 2.00-2.30 hours. There were few openings in the furnace. So, there was heat loss through the openings. As it was desired to get the temperature as ASTM E119, the furnace temperature was not as high as required(ASTM E119-00a, 2000).

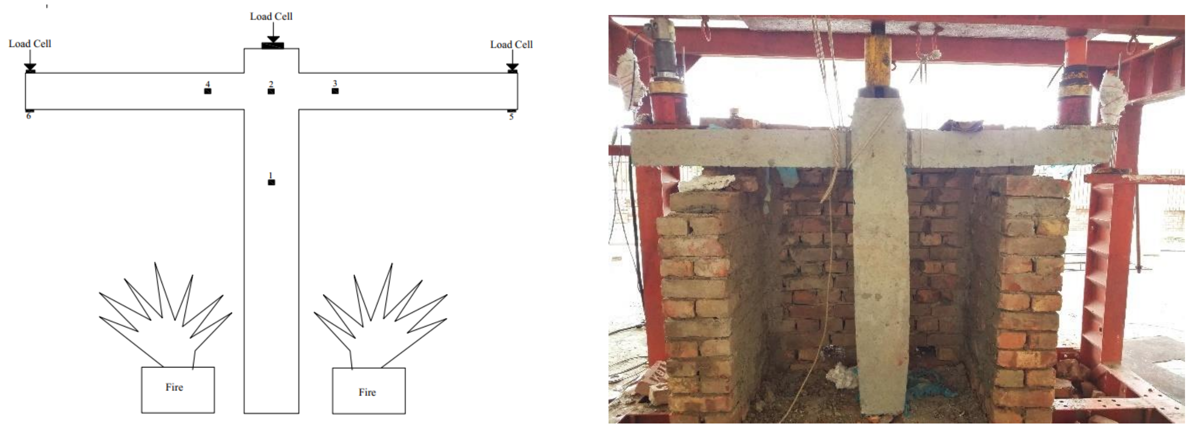

Figure 4: Mud fernace and load setup for experiment

2.5 Load Test of TheSpecimen

The specimen was cooled to room temperature before the load test. There was a crack on the specimen at the time of heating. After cooling to room temperature, the applied load was increased on the beam up to its ultimate capacity toknow the performance of the beam after heating.Another specimen was just tested without heating.

3. RESULTS AND DISCUSSION

3.1 Behaviour of Beam at Working Load

Beam under working load has limited deflection. After the increasing load, the deflection increased at a constant rate. The capacity of thebeam was found to be about 25 kN. The 40% of the load was provided as a working load. About 8.4kN load was applied gradually. The figure shown below illustrates the load vs deflectionvariation. When the working load had been applied the maximum deflection was about 2.4mm. In the two beams, the deflection was about the same and the load transmitted at the two beams was also the same.

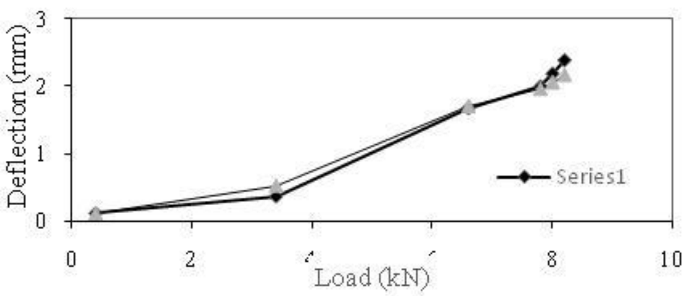

Figure 5: Deflection of Beam at working load

3.2Temperature Variation at Preheating Condition

Preheating of concrete was applied due to the removal of the water inside the concrete. Nearabout 40 min was the preheating time. The temperature of the concrete at the time of preheatingwas about 70ºC, and the outside temperature was about 200ºC. the average temperature inside theconcrete was about 50ºC. The temperature rise of the concrete was gradual. As the concretethermal property is low, the increase of temperature inside the concrete was slow.

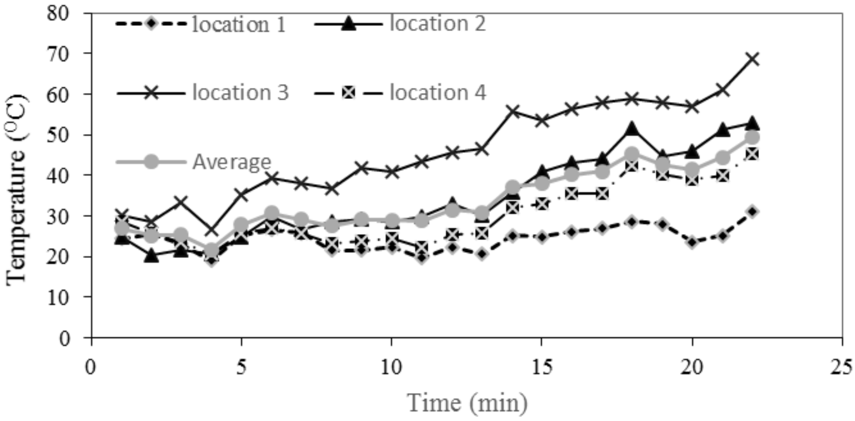

Figure 6: Temperature variation of the furnace with time

3.3 Furnace Temperature at The Time of Heating of The Specimen

At the time of heatingof the specimen, the temperature was in the range of 380ºC to 450 ºC. Maximum temperature wasrecorded at about 480ºC. There was a considerable amount of heat loss due to the opening of thefurnace. For this, the heat couldn't be increased so as the temperature did not rise more than500ºC. If the heat loss could be controlled, the temperature may rise more than this. According to ASTM E119 fire, the temperature variation is constant. The temperature should be about 1200ºC after 3 hours. If the fire condition changes the variation of temperature alsochanges. Figure 7 illustrates the variation of temperature with time. The fire temperatureincreases with time. However, fire 1 and fire 2 decreased suddenly as the furnace was not fully enclosed. Heat loss was very high.The causes of the decay are ventilation, fuel load, and lining materials.

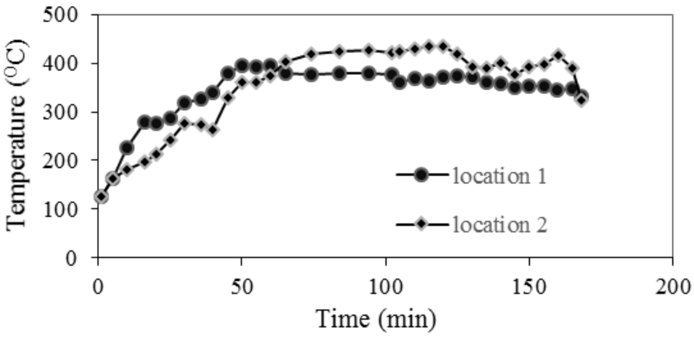

Figure 7: Furnace temperature at the time of heating

3.4 Specimen Surface Temperature

The variation of temperature at the surface of the concrete increases with time as the the furnace temperature increases. The surface temperature was less than the furnace temperature.From the figure, we can see that the temperature first rises rapidly. After a certain time, thetemperature of the sample did not rise. The average value of the temperature was 300^{0}C to 400^{0}C.

3.5TemperatureInsideThe Concrete

The most important concern of the research was the inside temperature of the concrete. The the figure shown below illustrates the inside temperature variation of the concrete. From the figure,it is clear that the temperature of the concrete increases with time. Though the outside the temperature of the concrete was about 400 ^{0}C, but the inside highest temperature was only 130 ^{o}C. Another important concern is that the temperature of the inside of the concrete did not decreases though the outside temperature of the concrete decreases. The decrease in the

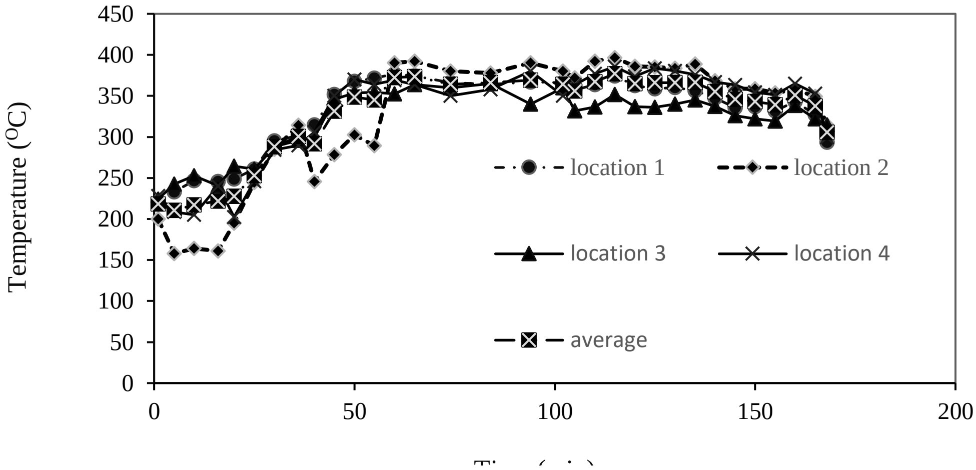

Figure 8: Surface temperature of specimens

the temperature was about 30% of the surface temperature. The conduction of heat was very low. As the heat flows upward, the heat at the bottomsurface is very high concerning the top surface. The furnace ventilation releases the heatso that the temperature cannot rise very high. At the time of heating, there was no spallingof the surface as the temperature was not as high as required for spalling.

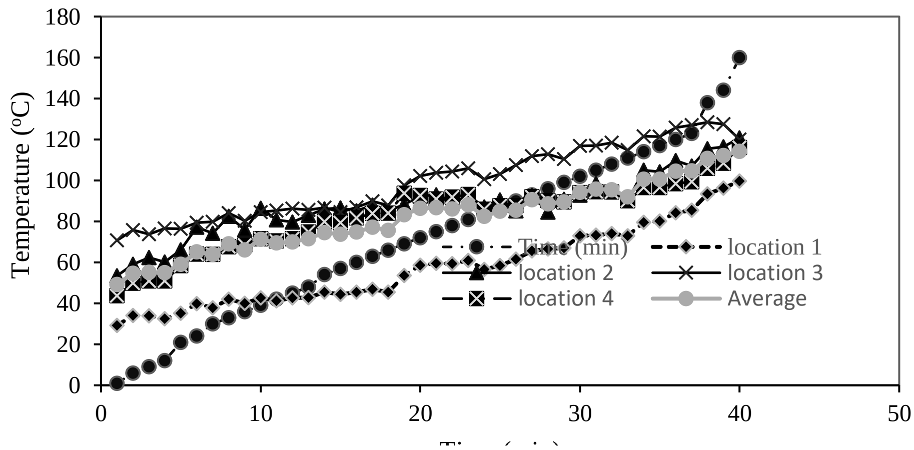

Figure 9: Temperature inside the concrete

3.6 Variation of Deflection of TheBeam Due to Increase in Temperature

The deflection of the beam with the time at the time of heating increases. The deflection of the beams increases at a considerable rate. The maximumdeflection of the beam at the time of heating was an average of 8mm. At elevated temperatures, the strength of theconcrete decreases by a considerable amount. So the deflection of the concrete increases withthe volume of the concrete. As the deflection of the beams was very high, there were linecracks at the joint. From the cracks, it was clear that the concrete is susceptible to heat.

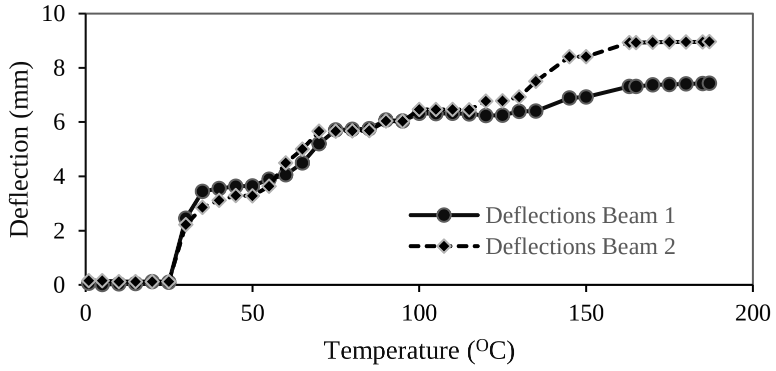

Figure 10: Variation of deflection with time due to exposed high temperature

3.7 Variation of Deflection of the Beam at Ultimate Loading

After cooling in the normal temperature exposed to air the load was applied with the hydraulicpump. With the increase of the load, the deflection was increased at a very high rate. After acertain time, the deflection rate was not so high because of reaching the ultimate capacity of thebeams. So there was not so much deflection. The total deflection of the beam was 20mm. But according to code, the maximum allowable deflection was 11.3 mm.

Figure11: Cracking of specimen

For a normal condition beam, the deflection of the beam at normal conditions was similar to that of exposure to higher temperature. However there was little variation of the deflection at the two beams.

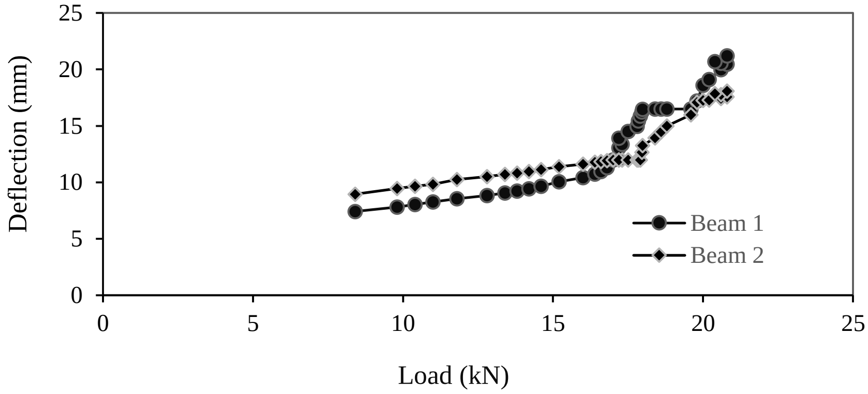

Figure 32: Variation of deflection of beam loading

There might besome mechanical error in the beam such as beam 1 might not get the equal load as the loadwas provided from the common hydraulic jack. But the ultimate capacity of the beam was veryclose. The ultimate capacity of the beams was near about 30 kN. It is greater than the ultimatecapacity of the beams exposed to higher temperatures. So, there is about a 33% loss of the ultimatecapacity as the beam is exposed to high temperature for 2 hours.

4. CONCLUSIONS

The concrete joint performance after being exposed to high temperatures has critical conditions. Thethe ultimate strength of the beam at normal conditions was about 30 kN and after heating the the ultimate strength of the joint was about 20 kN. The deflection of the beam at ultimate strengthwas 15mm for normal conditions and 20mm at ultimate strength after heating at elevatedtemperature. After heating the sample there was a hairline crack on the beam column joint. Thefurnace temperature was about 450^0C at the bottom of the beam where the surface temperatureof the sample was about 400^0C. the temperature inside the concrete was about 120^0C at a depthof 75mm at the beam and 125mm in the column. The inside temperature of the concrete was about30% of the surface temperature. The decrease of temperature was very high at a small depth.From the results, it was found that the joint capacity of the sample reduced about 33% afterheating at 400^oC for two hours. The reduction of the strength was less than 40%, so the concretewas not dead concrete and it can be used after heating. The cracking pattern of the beam wasvertical from the top of the beam. The failure was bending failure as there was no shear crackin the beam. Though there were small cracks on the beam after heating for two hours, thedeflection was within acceptable limits. The deflection of the beam increases after heating concerning the deflection of the beam at normal condition. The volume of the sample also may increaseas the applied load on the top of the column increases concerning time and the time of heatingautomatically. Based on the experimental results the following conclusions were made:

- Through properly heating the concrete, the strength of the joint decreases by about 33%from the normal condition.
- Increasing temperature has a significant effect on the deflection of the beam, 40%deflection of ultimate deflection happens at the time of heating.
- Temperatureaffects the volume expansion of concrete effecting the deformation.
- Elevated temperature shows the effect on the failure pattern of the beam as there was hairline crack after heating.

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