Effect of Fire Flame (High Temperature) on the Behaviour of Axially loaded Reinforced SCC Short Columns

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Abstract:

Experimental research was carried out to investigate the effect of fire flame (high temperature) on specimens of short columns manufactured using SCC (Self compacted concrete).

To simulate the real practical fire disasters, the specimens were exposed to high temperature flame, using furnace manufactured for this purpose. The column specimens were cooled in two ways. In the first the specimens were left in the air and suddenly cooled using water, after that the specimens were loaded to study the effect of degree of temperature, steel reinforcement ratio and cooling rate, on the load carrying capacity of the reinforced concrete column specimens. The results will be compared with behaviour of columns without burning (control specimens).

The results showed that, the ultimate load capacity of columns exposed to fire decreases with increasing the fire flame temperature. At burning temperature $300^{\circ}C$, $500^{\circ}C$ and $700^{\circ}C$, the average residual ultimate load capacity for gradually cooled specimens were 91%, 81% and 71% respectively. By increasing the ratio of longitudinal reinforcement 44% , the maximum improvement in the ultimate load capacity was 24% and 17% for the gradually and sudden cooling respectively at $500^{\circ}C$. For the same longitudinal reinforcement ratio and fire burning temperature, the ultimate capacity for the sudden cooling specimens was less than that of gradually cooled specimens by about 10%.

Key word: self compacted concrete (SCC), elevated temperature, fire flame.

الخلاصة:

بحث عملي اجرى لدراسة تاثير درجات الحرارة العالية (الحريق) ،على نماذج من الاعمدة الخرسانية القصيرة المسلحة والمصنعة باستخدام الخرسانة الذاتية الرص. النماذج عرضت الى لهب بدرجات حرارة عالية ، باستخدام فرن صنع لهذا الغرض.

لمحاكات كارثة الحريق الحقيقية في الموقع تم تبريد النماذج بطريقتين ، تبريد بطيء وتبريد فجائي باستخدام الماء.

ومن ثم يتم تحميلها لدراسة تاثير :

تاثير لهب الحريق في درجات حرارة مختلفة ، نسب حديد مختلفة بعد الحرق ، ومعدل التبريد (تدريجي و فجائي).

على قوة تحمل الاعمدة الخرسانية المسلحة ومقارنة النتائج مع نماذج السيطرة(الاعمدة التي لم تتعرض للحريق).

اظهرت النتائج ، ان مقاومة تحمل العمود للاحمال تقل كلما زادت درجة الحرارة، حيث وصلت الى 71%، 81% و 91% للنماذج المعرضة الى $300^{\circ}C$ ، $500^{\circ}C$ و $700^{\circ}C$. بزيادة نسبة حديد التسليح الطولي بمقدار 44% كانت اعلى زيادة في قوة تحمل العمود 24% و 17% للنموذج المبرد تدريجيا وفجائيا بالتتابع عند درجة حرارة $500^{\circ}C$ ..

وجد انه عند استخدام نسبة مماثلة من الحديد الطولي ولنفس درجة الحرارة فان الاعمدة المبردة فجائيا اقل تحملا للاحمال من المبردة تديجيا بمقدار 10%.

Introduction:

Many researchers studied the effect of fire on concrete, reinforcement, bond strength between them and the behavior of different reinforced concrete members. Most of tests were done using electrical furnaces to expose such members to elevated temperature, where only heat is supplied, while in real fire, the members are subjected to the flame which accompanied with different gasses. Another problem must be simulated which is the sudden cooling of the reinforced concrete members due to the way of controlling the fire by using water. Most of the researchers a greed that the damages in the concrete mix started at temperature above $300^\circ C$, because the porosity of cement paste increased rapidly to formation of micro cracks [Piasta et al 1984] due to dehydration of some cement paste compounds. Beyond $500^\circ C$ additional increase in porosity caused by liberation of water from the dissociation of $Ca(OH)_2$ in the range $450\text{-}550^\circ C$ and liberation of CO_2 as a result of $CaCO_3$ decomposition above $600^\circ C$ [Piasta et al 1984]. For reinforced concrete members, [Robston T.D. 1962] found that till $100^\circ C$ the thermal expansion in the steel reinforcement is approximately similar to that of the normal concrete, above this temperature the expansion of the steel will increase while the concrete will suffering from shrinkage due to the dehydrate of cement, this will cause loosens in the bond between concrete and steel reinforcement and cracks will started to form and grow. [Harada et al 1972] they found that the residual bond strength between the concrete and the reinforcement was 44% of the control specimens at a temperature of $300^\circ C$ and dropped rapidly to 10% at $450^\circ C$, while the residual strength of compressive strength was 60% at the similar temperature.

Material Properties:

- The cement used in this study was Ordinary Portland Cement complying with ASTM C150-02. Test results are shown in **Tables 1** and **2** for the chemical and physical properties respectively.
- The coarse aggregate used was natural aggregate with 10mm maximum size of aggregate. The grading obtained from the results of sieve analysis of the aggregate lies within the range defined by ASTM C33-03.
- The results of sieve analysis which was carried out on fine aggregate lies also within the range defined by ASTM C33-03. The chemical and physical test results for gravel and sand are shown in **Tables 3** and **4** respectively.
- Glenium 51: (modified polycarboxylic ether) was used as a water reducing agent plus stabilizing agent with a specific gravity of 1.1, at 20^oC, PH = 6.5 as issued by the producer.
- Silica fume mineral admixture or micro silica**:** composed of ultrafine, amorphous glassy spheres of silicon dioxide (SiO_2), produced by Crosfield Chemicals, Warrington, England.

Concrete mix proportions:

Several trail mixes were used. The final mix proportions used is 1:1.5:1.6 with water cement ratio 0.5 in addition to 3 liters of glenium-51 admixture for each 100kg of cement was used. The mixture proportions are summarized in **Table 5** .

The slump flow for the self compacted concrete was 685mm (using cone test-ASTM C1611-05) and the slump test for the normal concrete was 100mm (ASTM C143-00).

Deformed steel bars of diameter 10mm and 12mm were used as longitudinal reinforcement. While for the ties reinforcement 3mm smooth bar diameter was used. Determine their tensile properties according to ASTM

615-05a. The results are shown in **Table 6**.

The mixing of concrete was carried out in a tilting pan type mixer of $0.1m^3$ capacity. In all the mixes, the aggregates and cement were first mixed dry for about 90seconds. The water, silica fume and the superplasticizer together were mixed externally in a pan then added to the pan mixer, after that mixing continued, for a further 90seconds. With each beam six (100mm x 100 mm x 100mm) cubes were cast to determine the compressive strength of the hardened concrete.

Experimental Program:

Twelve reinforced concrete columns were tested, with overall length of 700mm and cross-sectional area of 100 x 100mm as shown in **Fig. 1-A**. All columns specimens have a top and bottom bearing hat with a square tied ring all made of 2mm thick steel plate to prevent end bearing failure of the two ends and to be insure that the load are distributed uniformly overall the column ends. Specimens were tested in the structural lab of Al-Mustanseria University. All specimens were reinforced with four longitudinal steel bars, as shown in **Table 7.** Specimens C_1 to C_6 were reinforced with 4 - $\phi\,10$ mm, while C_7 to C_{12} were reinforced with 4 - $\phi\,12$ mm. Ties were made of 3mm smooth bar diameter and spaced at 100mm in all the specimens and the clear cover was 6mm. To prevent the deference in concrete strength among the specimens, all column specimens castled in the same period.

The furnace was manufactured by using 3mm thick steel plate to burn one specimen in each time, as shown in **Fig. 2**, the clear space was 800mm height by 500mm width and 400mm length, appropriate with the specimens dimensions, to keep enough space around the specimen to reach the fire from the fire sources (nozzle) to the specimen. The nozzles were eccentrically positioned, four in each side of the furnace, as shown in **Fig. 2-A**, to distribute the flame along the specimen height. The specimen was positioned as shown in **Fig. 2-B** in the furnace to divide the flame on two faces of specimen on each side, so, the fire flame was subjected directly to the specimen on its four sides, by using a network of methane burners. Two column specimens were left without burning as control specimens C_1 and C_2. The specimens were cast, then moist curried for seven days, after that dried by air in the laboratory. Ten specimens were subjected to burning by fire flame at age of 45 days at three temperature levels 300, 500 and $700^{\circ}C$ and for similar exposure period of 1houre after reaching the target temperature. After this period, the fire flame was turn off , the case of the furnace removed and the specimen was cooled gradually by left the specimen in the air such as specimens C_2 and C_3, or suddenly by using splash of water till reaching the normal temperature as in specimens C_4 and C_6. The temperature was monitored by using digital thermometers inside the furnace and a thermocouple wire (Type K) made of Nickel-Chromium covered with cement to resist the temperature, with a digital temperature reader. The thermocouple wire poisoned at the specimens mid height fix with longitudinal reinforcement during manufacturing the specimen, as shown in **Fig. 1**.

Results and discussion:

Compressive strength:

The results show that the compressive strength was varying with the fire flame temperature as shown in **Fig. 3**, it decreases with increasing the exposure temperature. The average percentage of residual compressive strength after exposure to $300^{\circ}C$, $500^{\circ}C$

and $700^{\circ}C$ was 82%, 65% and 43% respectively, for the specimens cooled gradually. The results agree with that obtained by other researchers for normal concrete, [Nevile and Brooks 1987] and [Al-Kafaji 2010]. The decrease in compressive strength of concrete is due to the breakdown of interfacial bond due to incompatible volume change between the concrete components during heating and cooling [Venecanin 1977]. While for the specimens which cooled suddenly (high rate of cooling), the residual compressive strength was slightly lesser than that, they were 61% 39% for the exposure temperature of $500^{\circ}C$ and $700^{\circ}C$ respectively. This may be due to the grading progression of decreasing temperature (cooling), which will never be uniformly through the concrete cross-section, because losing temperature will delay for the inner concrete than that of the outer concrete, this process will create internal damaged stresses, and it will be worse with increasing the cross-section of the concrete member. [Mohamedbhai 1986] conclusion's agreed with these results (for the normal concrete) till $500^{\circ}C$ but in contrast with that at $700^{\circ}C$, his conclusion was, cooling rate affects on the residual concrete strength till $600^{\circ}C$ temperature, but it had no affect at more than this temperature, this may be because of using electrical furnace which can not allow to control the real cooling rate because of the delay time between the end of the exposure temperature and the cooling process i.e. no allowance to sudden cooling immediately after the exposure period.

Cracks due to burning:

Cracks were observed on the concrete surface of the column specimens after burning and cooling the specimens, and these cracks become deeper with more intensity as the fire flame temperature increase, as shown in **Figs. 4-A** and **4-B** for columns C_4 and C_6 respectively. The column specimens which were exposed to $700^{\circ}C$ the concrete of the corners were split as shown in **Fig. 4-B**. This explains the decreases in the compressive capacity of the concrete with increasing the temperature. Comparing the formation of cracks in the two cooling conditions as shown in **Figs. 4-B** and **4-C**, the sudden cooling in specimen C_6 has greater effect on the cracks formation than specimen C_5 which cooled gradually. This is because the rate of increasing temperature (according to the standard fire) ASTM E119-02 was less than that of decreasing temperature (sudden cooling), which had much worse effect on the interfacial bond between the concrete components. Also, the color of the concrete changes to pink, this may be due to hydration of iron oxide component and other mineral of cement and the aggregate [Al-Kafaji 2010] and [Neville 1995].

Fig. 5 shows the percentage of loss in weight versus the fire temperature, as shown in this figure the percentage of loss increased with increasing the fire temperature. [Mohammed 1987] recorded that till 300^{o}C only the free water will be lost after that, the loss in weight caused by the chemical change in the aggregate properties. These two types of losses increase the cracks formation.

Mode of failure after the loading test:

For the control column specimens C_1 and C_2 few longitudinal fine cracks were observed at the outer thirds of the specimens, at about 105kN and 120kN respectively. With increasing load, new cracks were formed and the earlier cracks become wider. In general, the observed cracks were forming and progressed parallel to the longitudinal axis of the specimen then turned toward the edges of the cross-section. Failure occurred when the concrete crushed in one of the two outer thirds of the specimens and the reinforcement buckled.

For the other column specimens which were burned and cooled before the loading test, the cracks (burning and cooling cracks) were observed before applying the load. So, the first crack load can not be recognized. The cracks formed and grew randomly. With increasing load, the concrete cover spalls in some places near the specimens corners. This phenomenon was observed at earlier loading stages in the specimens which exposed to high temperature ($700^{\circ}C$), also in the specimens which were cooled suddenly more than the specimens which were gradually cooled, this means spalling happened due to the exposure to high temperature but was delayed because it happened in limited places. In contrast, this observation was recorded by [Khoury 2000], [Al-Kafaji 2010] and others, they tested specimens of normal reinforced concrete, the spalling happened during the exposure to high temperature in wide areas, lead to split the concrete cover and exposing the reinforced to the direct fire flame and decrease the concrete cross section, this causing reduction in the strength of the reinforced concrete element. Failure happened in all the burned specimens by crushing the concrete at different axial load as shown in **Table 8**. specimens C_5 and C_6, failed at the middle third of the specimen with splitting the concrete diagonally, as shown in **Figs. 6-A** and **6-B**, while specimens C_{11} and C_{12} failed in the same manner but at the outer third as shown in **Figs. 7-A** and **7-A**.

Table 8 and **Fig. 8** show that, the axial ultimate load capacity decreases with increasing the fire flame temperature. **Table 8** shows, at burning temperature $300^{\circ}C$, $500^{\circ}C$ and $700^{\circ}C$, the average residual ultimate load capacity for gradually cooled specimens were 91%, 81% and 71% respectively. Because, as the temperature increased the cracks formation and growing increased this lead to decrease the bond strength between the concrete components as well as between the concrete and the reinforcement bars. Also, the expansion of the steel will increase while the concrete will suffer from shrinkage. **Fig. 8** shows, at $500^{\circ}C$ and for the same longitudinal reinforcement ratio, the ultimate load capacity for the sudden cooling specimens were less than that of gradually cooled specimens, by about 5% for C_4 with respect to C_3 and 10% for C_{10} with respect to C_9. While at $700^{\circ}C$ the sudden cooling specimens of both the two longitudinal reinforcement ratio were less than the gradually cooled specimens, by about 32%, as mentioned before the sudden cooling had worse effect because of the high probability of cracks formation due to the difference in temperature between the inner and outer concrete during the cooling process as well as the burning process. In another hand the concrete components had different thermal expansion lead to breakdown of interfacial bond due to incompatible volume change between the concrete components during heating and cooling. This causes a reduction in the ultimate load capacity of the specimens.

Also, it can be seen, the decrease in the ultimate load capacity at $300^{\circ}C$, $500^{\circ}C$ and $700^{\circ}C$, were 5%, 28% and 32% respectively for specimens with 4-$\phi\,10\,\text{mm}$ ($\rho = 0.0314$) and 4%, 23% and 26% respectively for specimens with 4-$\phi\,12\,\text{mm}$ ($\rho = 0.0452$). This means the rate of decreasing ratio at 500 $^{\circ}C$ was higher than that at $700^{\circ}C$, this agreed with [Mohammed, A. S., 1987], they concluded that rich mixes losses most of the ultimate strength till about $400^{\circ}C$ because the cement past will effect first, while the poor mixes losses most of their strength at higher than

$400^{\circ}C$, because the aggregate effect by the temperature at this rang.

Fig. 8 shows that, increasing the ratio of longitudinal reinforcement will improve the ultimate load capacity. Comparing the ultimate load capacity of the specimens had ratio of longitudinal reinforcement of $\rho = 0.045$ with that of $\rho = 0.031$ at the same burning and cooling conditions, as shown in **Table 9**. The percentage of increase was 10%, 10%, 24% and 19% at the exposure temperature of $35^{\circ}C$, $300^{\circ}C$, $500^{\circ}C$ and $700^{\circ}C$ respectively for the gradually cooled specimens, while it was 17% and 14% at $500^{\circ}C$ and $700^{\circ}C$ respectively for the sudden cooled specimens. This comparison leads to proof, increasing the ratio of longitudinal reinforcement will increase the ultimate load column capacity of burned reinforced concrete column and it's more effective at the higher burning temperature, because, the reinforcement bars less effected by fire flame than the concrete till about $600^{\circ}C$. As concluded by [Harada 1972], [Nuri 1983] and [Fahemi and Asa'ad 1991], the reinforcement had less affected by the burning temperature till about 600^{o}C and at 750^{o}C the residual yield strength of the reinforcement will be about 85% . Also, it can be seen that the ratio of sudden cooled specimens were less than that of the gradually cooling at the same burning temperature. Because of the propagating of cracks due to cooling process, as well as the burning process. **Figs 9** and **10** show the column specimens axial deflection versus the axial load. In general the control column specimens C_1 and C_7 showed slightly higher rigidity than the burned specimens and it decreased as the burning temperature increased. This may be due to the burned specimens which suffering from crack formation due to burning and cooling process before the loading test. Also, the confinement of lateral explosive of the reinforced concrete column is a contribution of the tensile strength of the concrete and much than that of the reinforcement ties. [Al-Ausi and Faiyadh 1985] concluded that the splitting tensile strength of the concrete is mach effect by the fire temperature than the compressive strength. So, as the burning temperature increased the tensile strength decreased or vanished and the ties will confine all the lateral deformations.

Conclusions:

- The compressive strength was varies varying with the fire flame temperature. It decreases with increasing the exposure temperature.
- The average percentage of residual compressive strength after exposure to $300^{\circ}C$, $500^{\circ}C$ and $700^{\circ}C$ was 82%, 65% and 43% respectively, for the specimens cooled gradually. -While for the specimens which cooled suddenly (high rate of cooling), the residual compressive strength was slightly lesser than that, they were 61% 39% for the exposure temperature of $500^{\circ}C$ and $700^{\circ}C$ respectively.
- Cracks were observed on the concrete surface of the column specimens after burning the specimens, and these cracks become deeper with more intensity as the fire flame temperature increase, and the sudden cooling had great effect on cracks formation.
- The percentage of loss in weight of the specimens increased with increasing the fire temperature, it reached to about 3% at$700^{\circ}C$.
- The probability of spalling the concrete cover near the specimens corners increased as the burning temperature increased. This phenomenon was observed at earlier stage of loading test of the specimens which exposed to high temperature than the others, also in the specimens which were cooled suddenly more than the specimens which were gradually cooled.

- The ultimate load capacity of the column specimens decreases with increasing the fire flame temperature, at burning temperature $300^{\circ}C$, $500^{\circ}C$ and $700^{\circ}C$, the average residual ultimate load capacity for gradually cooled specimens were 91%, 81% and 71% respectively.
- Increasing the ratio of longitudinal reinforcement will enhance the residual load capacity.
- Increasing the reinforcement ratio by 44% lead to increase the ultimate load capacity by 10%, 10%, 24% and 19% at the exposure temperature of $35^{\circ}C$, $300^{\circ}C$, $500^{\circ}C$ and $700^{\circ}C$ respectively for the gradually cooled specimens, while it was 17% and 14% at $500^{\circ}C$ and $700^{\circ}C$ respectively for the sudden cooled specimens.
- for the same longitudinal reinforcement ratio, the ultimate load capacity for the sudden cooling specimens were less than that of gradually cooled specimens, by about 5% for C_4 with respect to C_3 and 10% for C_{10} with respect to C_9. While at $700^{\circ}C$ the sudden cooling specimens of both the two longitudinal reinforcement ratio were less than the gradually cooled specimens, by about 32%.

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Table 1: Chemical cement test results.

Chemical composion	
Composion	Quantity%
SO_3	2.33
MgO	2.88
C_3A	9.49
SIO_2	21.86
Al_2O_3	5.58
L.O.I	1.29
C_3S	35.1
CaO	62.60
Fe_2O_3	3.14

*Chemical analysis was conducted by National Center for Construction Laboratories and Researches

Table 2: physical cement test results

Physical properties	
Compressive strength, MPa (3 days) (7 days)	 23.46 28.80
Setting time (Vicate apparatus), Initial setting, h:min Final setting, h:min	 2:50 4:15
Specific surface area (Blaine method), m^2/kg	394
Soundness (Auto Clave) method, %	0.10

*Physical tests was conducted by National Center for Construction Laboratories and Researches

Table 3: Chemical and physical gravel test results.

Properties	Test results
Absorption %	0.64
Specific gravity	2.62
Dry loose-unit weight kg/m^3	1562
Sulfate content as SO_3 %	0.05
Materials finer than 75μm%	1.36

* Tests was conducted by National Center for Construction Laboratories and Researches

Table 4: Chemical and physical sand test results.

Properties	Test results
Absorption %	0.82
Specific gravity	2.49
Sulfate content	0.043

* Tests was conducted by National Center for Construction Laboratories and Researches

Table 5: Concrete mix proportions

	SCC
Water Kg/m^3	200
Superplasticizer lit./100Kg (powder)	3
Cement Kg/m^3	392
Silica Fume Kg/m^3	8
Total Powder Kg/m3	400
Gravel Kg/m3	640
Sand Kg/m3	600

Table 6: Properties of steel bars

Bar diameter (mm)	Yield stress (MPa)	Strain at yield stress (microstrain)	Ultimate stress (MPa)
3	542	2710	632
10	512	2497	622
12	504	2571	618

Table 7: Details of the columns specimens.

Column designation	Longitudinal reinforcement	Longitudinal Bar diameter (mm)	Temperature C	Type of cooling
C1	4 - ϕ 10 mm.	10	-	-
C2	4 - ϕ 10 mm.	10	300	gradual
C3	4 - ϕ 10 mm.	10	500	gradual
C4	4 - ϕ 10 mm.	10	500	sudden
C5	4 - ϕ 10 mm.	10	700	gradual
C6	4 - ϕ 10 mm.	10	700	sudden
C7	4 - ϕ 12 mm.	12	-	-
C8	4 - ϕ 12 mm.	12	300	gradual
C9	4 - ϕ 12 mm.	12	500	gradual
C10	4 - ϕ 12 mm.	12	500	sudden
C11	4 - ϕ 12 mm.	12	700	gradual
C12	4 - ϕ 12 mm.	12	700	sudden

* All specimens are made of SCC: self compacted concrete.
** Average concrete strength before burning was 35MPa for the cubes 100 x 100 x 100mm

Table 8 : Columns test results

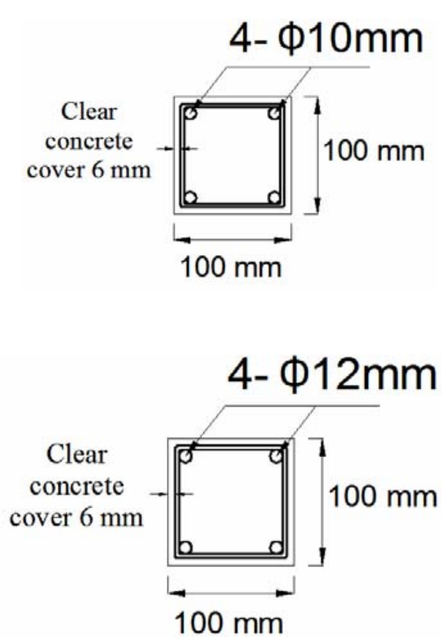
Column designation	Longitudinal Bar diameter (mm)	Temperature $^{\circ}C$	Type of cooling	Ultimate load capacity	Load capacity /reference column %
C1	10	-	-	305	100
C2	10	300	gradual	290	95
C3	10	500	gradual	232	76
C4	10	500	sudden	220	72
C5	10	700	gradual	207	68
C6	10	700	sudden	142	46
C7	12	-	-	335	100
C8	12	300	gradual	320	96
C9	12	500	gradual	287	86
C10	12	500	sudden	258	77
C11	12	700	gradual	247	74
C12	12	700	sudden	162	48

- Steel reinforcement ratio $\rho = 0.0314$ for specimens with 4 - ϕ 10 mm longitudinal bars

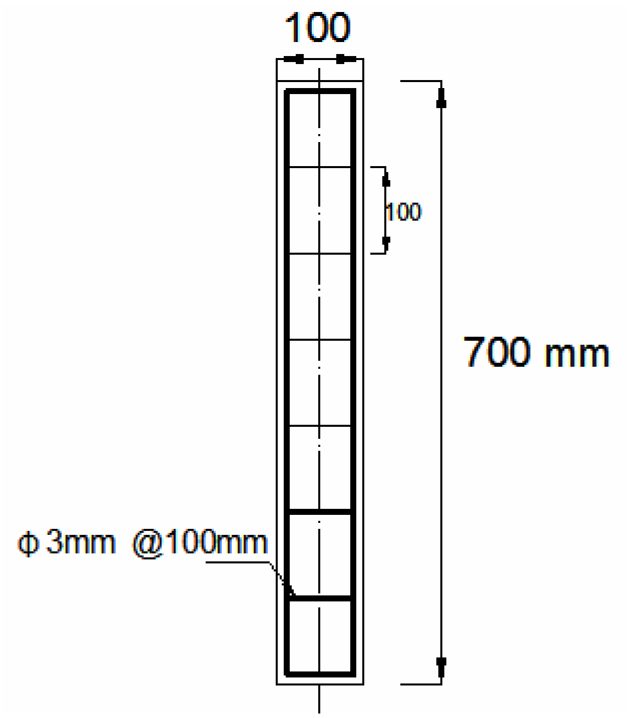
- Steel reinforcement ratio $\rho = 0.0452$ for specimens with 4 - ϕ 10 mm longitudinal bars

Table 9: The effect of reinforcement on the percentage of the ultimate load capacity of specimens had the same burning and cooling conditions

Ratio of ultimate load capacity of	C_7/C_1 Gradually cool	C_8/C_2 Gradually cool	C_9/C_3 Gradually cool	C_{10}/C_4 Sudden cool	C_{11}/C_5 Gradually cool	C_{12}/C_6 Sudden cool
Temperature $^{\circ}C$	0	300	500	500	700	700
Percentage of increase	10%	10%	24%	17%	19%	14%



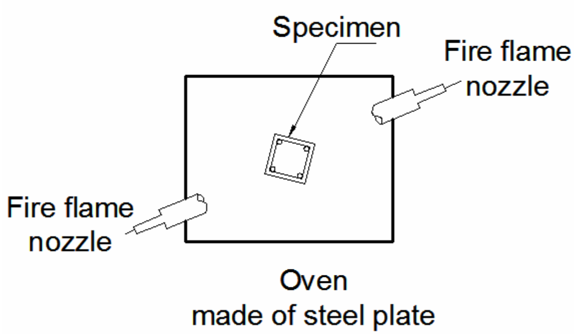
A- Column cross-section



B-Column dimensions

Figure 1 : Details of dimensions and reinforcement of concrete column specimens

A- furnace with nozzles



B- Specimen positioning in the furnace

Figure 2 : Details of furnace and the distribution of the nozzles and specimen position in the furnace during burning

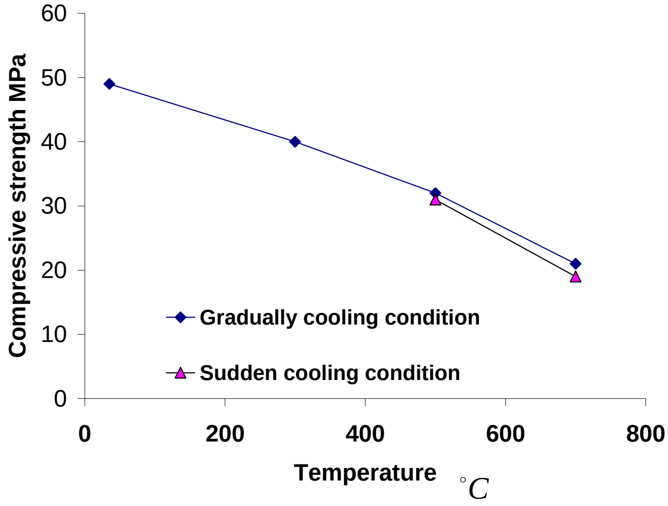


Figure 3: Effect of fire flame on compressive strength

A- Column specimen C_4 after exposure to 500 $^{\circ}C$ and cooled suddenly

B- specimen C_6 after exposure to 700 $^{\circ}C$ and cooled suddenly

C- specimen C_5 after exposed to 700 $^{\circ}C$ and cooled gradually

Figure 4 : Cracks formation at different conditions of cooling and exposure temperature before the loading test

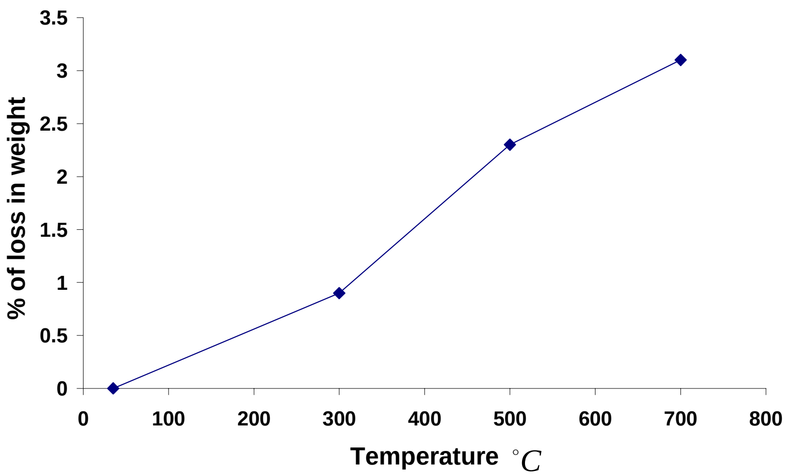
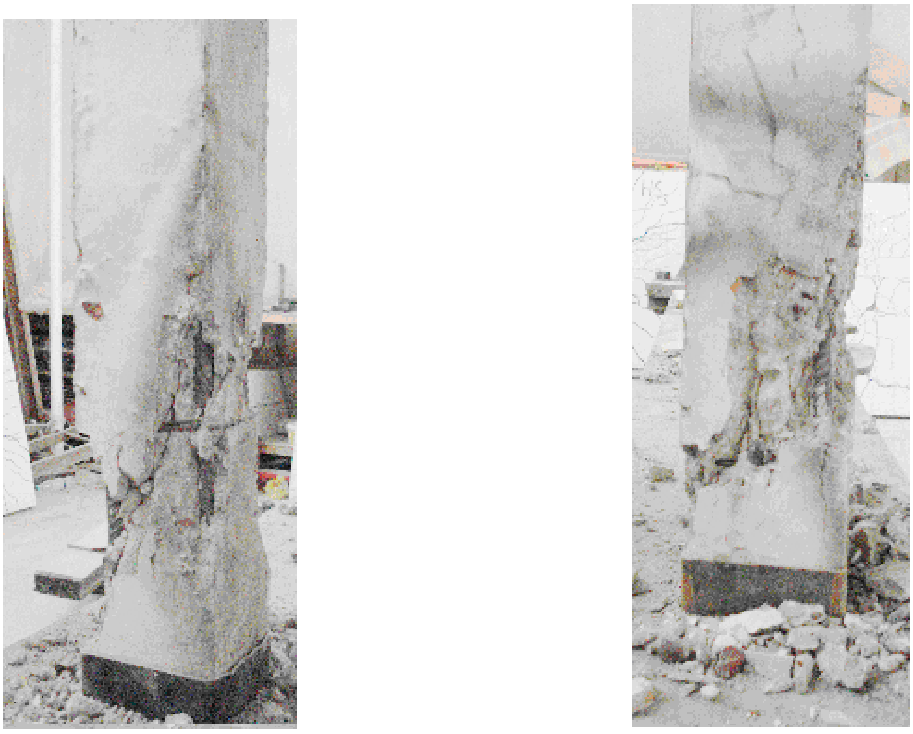


Figure 5: Effect of fire flame on the column specimens loss of weight

A- Column specimen C_5 **B-** Column specimen C_6

Figure 6: Failure mode of column specimens C_5 and C_6 after the loading test.



A- Column specimen C_{11} **B-** Column specimen C_{12}

Figure 7: Failure mode of column specimens C_{11} and C_{12}

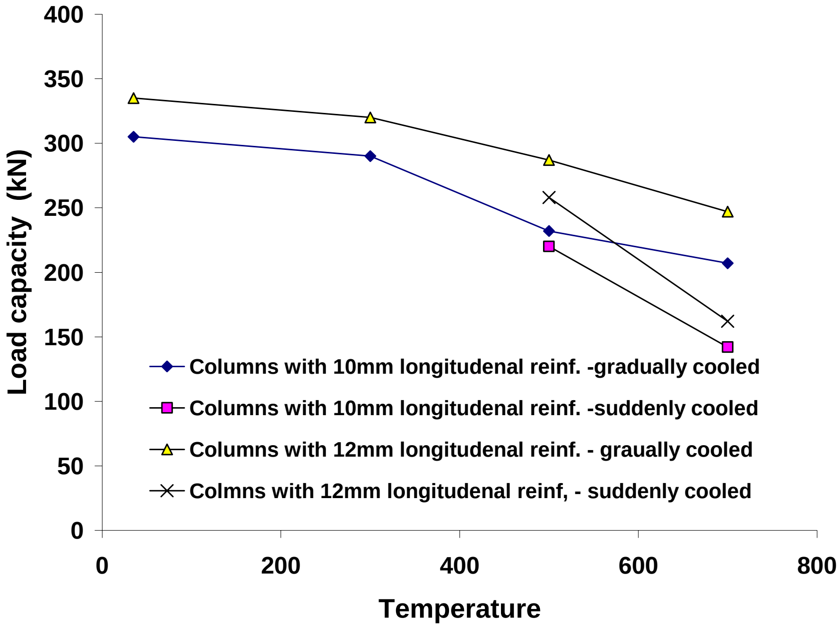


Figure 8 : Load capacity versus fire flame temperature

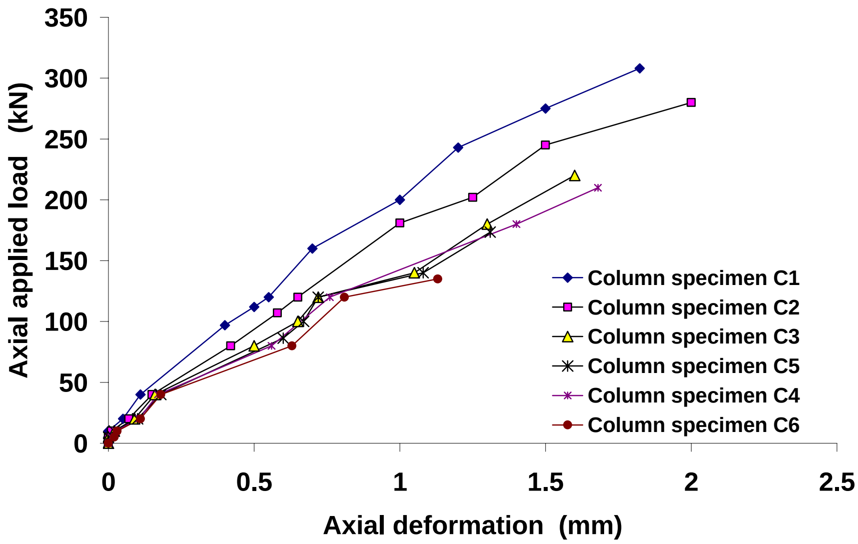


Figure 9 : Applied axial load versus Axial deformation

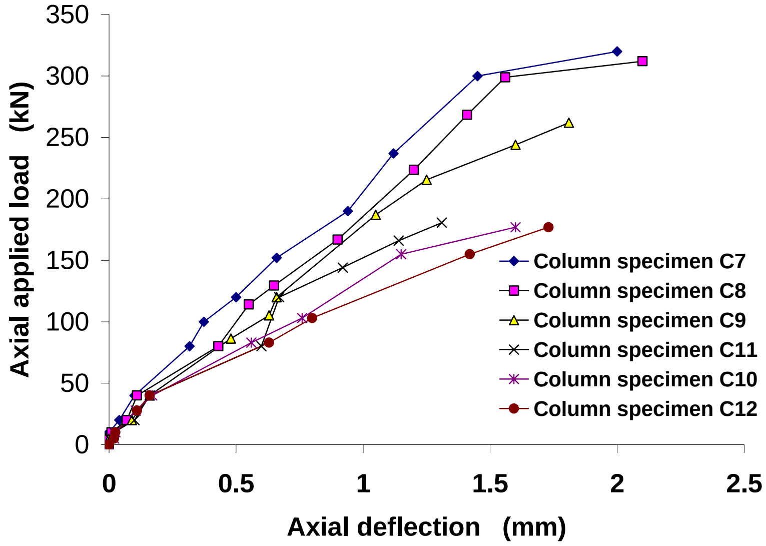


Figure 10 : Applied axial load versus Axial deformation