ACTA TEHNICA CORVINIENSIS – Bulletin of Engineering
Tome VII [2014] Fascicule 2 [April – June]
ISSN: 2067 – 3809

[1.] Mohamed A. A. El-SHAER

STRUCTURAL ANALYSIS FOR CONCRETE COLUMNS SUBJECTED TO TEMPERATURE

[1.] Civil and Construction Engineering Department, higher Technological Institute, 10th of Ramadan City, EGYPT

Abstract: This research is initiated with the objective of investigating the behavior of light weight reinforced concrete columns under elevated temperature. Light weight concrete is achieved by using light weight expanded clay aggregate (LECA) as partial replacement (by volume) to normal weight aggregate. Four specimens were tested experimentally where they were subjected to elevated temperature, and under axial load. Experimentally tested specimens are used to verify a numerical model established by a commercial finite element modeling package ANSYS 13.0. Experimental measurements and numerical results showed a good agreement. Numerical model is then used to cover a wider range of concrete characteristic strengths and with different heating scenarios. Results showed that a slight reduction in the load carrying capacity, stiffness and toughness in unheated light weight columns when compared to the normal-weight concrete columns. Contrarily, an enhancement in the load carrying capacity after subjecting to elevated temperature is obtained.

Keywords: Reinforced concrete columns, Elevated temperatures, Light-weight concrete (LWC), Finite element analysis, Failure modde, Ultimate load

INTRODUCTION

Collapsing of structural elements subjected to fire is worldwide documented. Likewise, it is the case in Egypt. Structural elements when subjected to fire usually fail to resist it and the whole structure collapses. This is considered a big investment economical loss. This occurs persistently in developing countries (i.e. where many illiterate people live) and in hot zones (i.e. where the weather warmth makes the ignition point of materials to be reached easily). The influence of elevated temperatures on the concrete strength was studied by many researchers. Generally, concrete loses most of its strength (i.e. 70% to 80% of its original strength at room temperature) if exposed to 500 – 600℃ for a long time [1]. This sharp drop in the concrete strength occurs due to the complete decomposition of the cement hydrates with appearance of several microcracks [2]. Sait and Turan [3] found that at 900℃ concrete lost almost all of its strength. Also, Abd El-Razek et al [4] studied experimentally the reduction in the ultimate capacities of axially loaded reinforced concrete rectangular columns after the exposure to elevated temperatures while others [5] investigated the effect of the exposure to direct fire on the behavior of high strength concrete columns.

Each of them concluded that the exposure to either elevated temperature or fire causes severe reduction in the ultimate capacity of the tested columns. Khafaga [6] reported that exposure to 550℃ elevated temperatures adversely affected the structural behavior of the tested columns under uni-axial bending moment in terms of residual capacity, serviceability performance, stiffness and toughness.

On the other hand, in concrete structures, the concrete imposes a huge amount of the total load of the structure. Lighter concrete offers design flexibility and substantial cost saving by providing less dead load, improved seismic structural response, low heat conductivity and lower foundation cost when applied to structures. In recent years, due to these advantages, there is an interest in production and investigation of the light or reduced-weight concrete. Demirbog, [7], studied the mechanical properties, durability and thermal conductivity of the lightweight concrete. Kayali, [8], used fly ash light weight aggregate to produce light-weight high performance concrete. He *reported that, concrete produced using these aggregates is around 22% lighter and at the same time 20% stronger than normal weight aggregate concrete. Also, drying shrinkage is around 33% less than that of normal weight concrete. On the other hand, Choi et al, [9], reported that the range of elastic modulus has come out as 24 – 33 GPa, for light-weight concrete (LWC) with compressive strength more than 40 MPa, comparably lower than the normal concrete which possessed the same compressive strength. In addition, for LWC, different researchers, have proposed different relationships to estimate modulus of elasticity value from compressive strength and unit weight. However, these relationships depend on the type and source of the light-weight aggregate, since the light-weight aggregates are porous and have modulus of elasticity values lower than that of natural aggregate. On the other hand, Haque et al [10], carried out an experimental study and found that replacement of Light weight fine aggregate with normal weight sand produces a concrete that is some now more durable as indicated by their water penetrability and depth of carbonation when concretes are of equal strength.*

However, although it was found that light-weight concrete (LWC) has good insulation and mechanical properties; it still needs further investigations of its structural behavior for use as structural members. Also Khafaga, [11], observed enhancement in ultimate carrying capacity of the reduced weight-concrete beams due to the increase in the concrete grade was lower than that of the normal-weight concrete beams and also reported that increasing the shear span to depth ratio promoted the beam action, decreased the cracking and ultimate loads and stiffness and increased the ductility of the reduced-weight concrete beams.

Nevertheless; there is a lack in knowledge about the structural behavior of the light-weight concrete when used in structural members. Previous researches indicated also that the properties of light-weight concrete depend on the type of its lightweight aggregates. Therefore, the structural behavior of light-weight concrete members may vary according to the type of the used light-weight aggregates.

The current research aims to investigate the effect of elevated temperature on the behavior of reinforced light weight concrete columns made of light-weight expanded clay aggregate (LECA) as a partial replacement (by volume) to the normal-weight aggregates. This is one of the widespread light-weight aggregates. Four reinforced concrete columns were fabricated and tested under axial load in compression machine of 5000 kN capacity. The effects of several variables such as type of concrete according to its weight, concrete grade and the effect of exposure duration were numerically investigated. The behavior of the tested columns was analyzed in terms of mode of failure, load-strains response, ultimate carrying capacity, stiffness and toughness. The test results are analyzed to demonstrate the effects of these considered variables on the tested light weight concrete columns as well as the normal-weight concrete columns.

However, due to the financial reasons, was obtaining the results from experiments, was not possible. Finite element method supplied a new way to study the behavior of light weight concrete columns subjected elevated temperature by computer, which can help the researcher to analyze and complete the experimental results and have a better understanding of it.

In recent years, using ANSYS as a finite element modeling software in many research works have been done successfully to simulate the numerical model for axial loaded concrete elements [12]. This software has plentiful element types and offers some default parameters, which makes it easy to develop a finite element model (FEM) to simulate the interactive behavior between concrete and other materials. In this study ANSYS 13 was used to implement the numerical study on the behavior of lightweight reinforced concrete columns subjected to elevated temperature.

Current research results are expected to assist engineers in design of fire resisting structures. Moreover, recommendations are going be added to the Egyptian Code of Practice (ECP).

EXPERIMENTAL PROGRAM

The experimental program consisted of fabricating and testing four reinforced concrete columns. Two of these reinforced concrete columns contained *light-weight expanded clay aggregates (LECA) as a partial replacement (by volume) to the normal weight coarse and fine aggregates with a percentage equals 50%. The unit weight of this type of concrete ranged between 1830 and 1890 kg/m³. The other two columns were cast with normal-weight concrete which contained normal-weight coarse and fine natural aggregates to be used as control specimens.*

Materials and Concrete Mixes

Two concrete mixes were used in the current research. Mix No. I possessed normal unit weights (control mixes) and mix No. II possessed reduced unit weights. The intended compressive strengths is 30 MPa. Table (1) shows the details of these two mixes.

Table (1) Proportions of Concrete mixes {for Concrete Strength 30 (N/mm²)}

Mix No.	Type of Concrete	Cement (Kg/m³)	Silica Fume (Kg/m³)	Coarse Agg. (Kg/m³) Dolomite	Coarse Agg. (Kg/m³) LECA
I	Normal weight	350	---	1224	---
II	Light weight	315	35	612	204
Mix No.	**Type of Concrete**	**Fine Agg. (Kg/m³) Sand**	**Fine Agg. (Kg/m³) LECA**	**Water (Lit/m³)**	**Admix. (Kg/m³)**
I	Normal weight	612	---	195	---
II	Light weight	306	184	185	7.0

Details of the Test Columns

The four tested columns have the same concrete dimensions, (i.e. 200 x 200 mm² in cross sectional area and length 1500 mm) Figure (1-a).

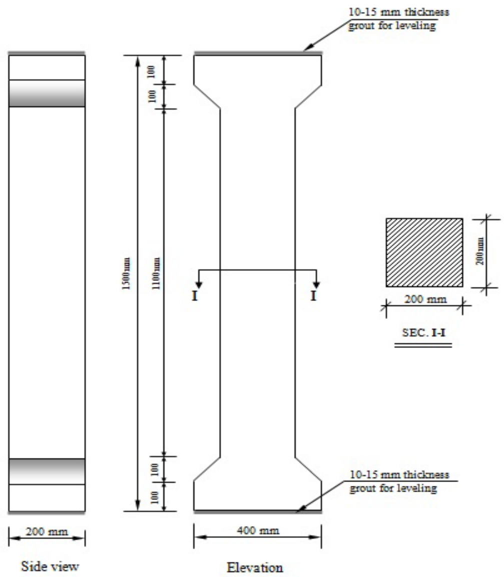

Figure (1-a) Concrete Dimensions of Typical Test Columns (200x200 mm²)

Table (2) presents the group number, column identification number, main characteristic values and the duration of elevated temperature.

Table (2) Column test results

Group	Column ident.	Type of Concrete	Exposure Duration (hrs)	Notes
I	$C1_{exp}$	Normal Weight	-------	Control specimen
	$C2_{exp}$		1.5	-------
II	$C4_{exp}$	Light weight	-------	Control specimen
	$C5_{exp}$		1.5	-------

The percentage of reinforcement (μ=1.13%) was used for all columns in the current study. Columns C1 and C4 were not exposed to elevated temperature.Therefore they were used as control specimens while the remaining two columns were exposed to a target temperature of 550℃ for 90 minutes. Allheated columns were cooled gradually in air for 24 hours before testing.

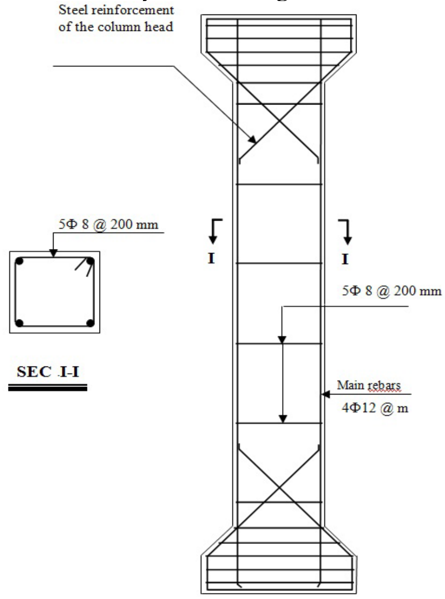

Figure (1-b) Typical Reinforcement Detailing Test Columns

Figure (1-b) shows the general reinforcement details for the tested columns. The main reinforcing bars were 4 Ø12 (high tensile steel 400/600) for columns of all groups. Stirrups with diameter 8 mm (mild steel 280/420) were detailed to keep the thickness of the concrete cover of the tested columns to be 25 mm which is the minimum thickness that achieve the requirements of the fire resistance of reinforced concrete columns according to the Egyptian Code ofPractice (ECP – 203) [13]. The main properties of the used steel bars are listed in table (3). Also, the column heads were designed to avoid failureduring loading by adding additional reinforcement for strengthening the column heads.

Table (3) Properties of Steel Reinforcement

Type Size	Mild Steel	High Tensile Steel
Diameter (mm)	8	12
Actual Cross Sectional Area (mm^2)	50.80	112.4
Weight / Unit Length (kg/m')	0. 399	0.882
Yield Strength (N/mm^2)	307.7	443.6
Ultimate Strength (N/mm^2)	437.7	676.4
Elongation (%)	28.9	13.2

Elevated Temperature Setup

Two of the tested columns were subjected to an elevated temperature of 550℃ from four sides for 90 minutes. The tested columns were heated under the application of concentric constant vertical load equals 1/3 of the ultimate loads determined from testing the comparative control (unheated) columns, $C1_{exp}$ and $C4_{exp}$, respectively. The tested columns and the furnace were mounted, during the exposure periods, in the loading testing machine which applies the concentricvertical load, figure (2).

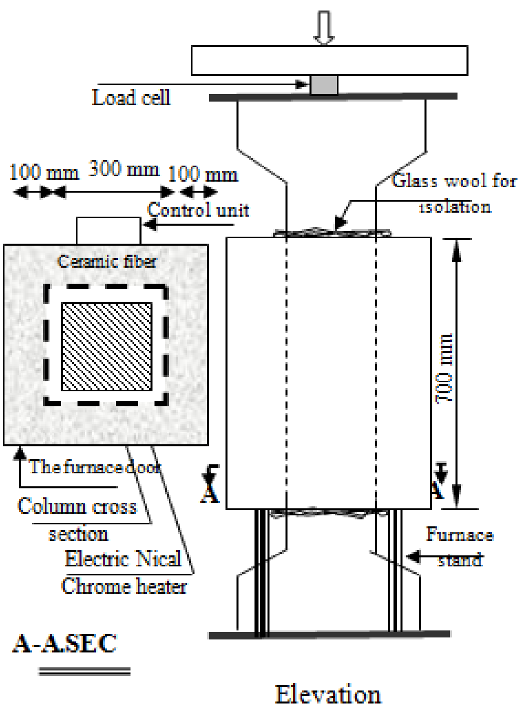

Figure (2) Setup of the exposure of elevated temperature from the four faces of the tested columns

Instrumentation and Test Setup

All columns are loaded up to failure. The control sample and the heated columns are loaded by a 5000 kN hydraulic compression machine. A load cell 2000 kN was used to measure the applied load and the readings were recorded automatically by means of a data acquisition system. Each of the tested columns was acted upon by a concentric vertical load.

Strains were measured at two sides of each tested column by attached linear variable displacement transducers (LVDT) which were connected to the data acquisition system. The LVDT measurements were determined on pegs mounted on the column sides at 700 mm spacing. Figure (3) illustrates a schematic of the loading setup and instrumentation of the tested columns.

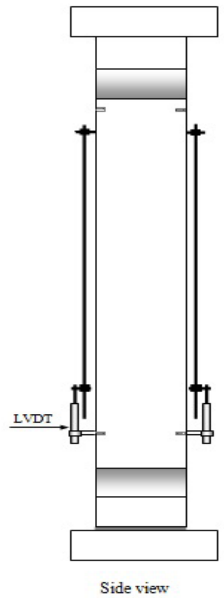

Figure (3) Instrumentation layout of the Test Columns.

Modes of Failure

For columns of group I ($C1_{exp}$, and $C2_{exp}$) of normal weight concrete, when the load increased beyond the point at which the peak load, the concrete crushed and the longitudinal reinforcement bars buckled. Then, a failure plane or a crushed zone was developed where the cover had pulled off. The failure of $C1_{exp}$ (unheated normal concrete column), occurred at sections at a certain height from the upper end. This is due to the effect of stress concentration at the column ends and the large sections chosen for the column heads. For column (heated normal concrete column), $C2_{exp}$ after the exposure to the 550℃ elevated temperature, the crushing of concrete took place at the mid height of the simulated columns, After the yielding point, cracks which were experienced at all sides, widen more. Upon further loading, concrete crushing occurred and larger deformations took place.

On the other hand, the failure modes of the light weight tested columns of the group II ($C4_{exp}$, and $C5_{exp}$), it was observed that light-weight aggregate concretes experience greater spalling when compared to normal-weight concretes. Thus, the lightweight aggregate concretes structures will only have potentially higher fire resistance than normal concrete structures if no spalling occurs.

The failure shape of the analysis column after loading is presented in figure (4).

Figure(4) Failure mode of column (Experimental)

FINITE ELEMENT ANALYSIS BY ANSYS

Finite element modeling in Ansys is considered perfectly performed if four main preprocessors activity is done. Finite element modeling is performed when element types selection, material modeling, geometrical modeling, and definition of loading schemes are completely done.

a. Element type selection

In Ansys, reinforced concrete behavior is achieved by combining two types of elements, namely, Solid65 and Link 8. SOLID 65 is an eight-node solid element, which is used to model the concrete cracking and crushing criterion. On the other hand, Link8 is a 3D spar element which is used to model reinforcing steel bars.

b. Material modeling

Two types of materials are modeled, concrete and steel materials. Concrete material is required to be capable of cracking in tension and crushing in compression. Despite the great advances achieved in the fields of plasticity, damage theory and fracture mechanics, unique and complete constitutive model for reinforced concrete is still lacking. Recently, researchers agreed to model concrete material in Ansys in both elastic and plastic loading stages. Concrete elastic behavior is well defined by both young's modulus (E_c) and Poisson's ratio (ν_c). Numerically E_c is dependent on concrete characteristic strength as will be discussed later and ν_c is assumed to be 0.2. On the other hand, concrete plastic behavior needs to be defined by multiple failure surfaces to capture concrete cracking, crushing, and large deformations during loading scenario.

By nature of material homogeneity, modeling of steel material is much simpler than concrete material. To satisfy the assumption that strain in steel and concrete at the same level is equal, steel material behavior should be defined in both elastic and plastic stress ranges. Steel elastic behavior is defined similar to concrete elastic behavior by both young's modulus (E_s) and Poisson's ratio (ν_s). Numerically, E_s and ν_s are assumed to be 210 GPa, and 0.2, respectively. 10% strain hardening is taken into considerations in steel plastic range rather than plateau behavior.

b.1. Concrete material non-linearity

Two failure surfaces are used to model the concrete plastic behavior, namely, concrete (CONC) and multi-linear isotropic (MISO) material models. Concrete material model predicts the mechanical failure of brittle materials, applied to a three dimensional solid element.

Consequently, CONC material model is capable of cracking in tension and crushing in compression. It can also undergo plastic deformation and creep. Mainly, two types of mechanical behavior data are defined to fill in CONC data table, uniaxial failure data (either in compression (f_c) or in tension (f_t)) and shear cracking parameters (β_t and β_c). The two input strength parameters, ultimate uniaxial tensile (f_t) and compressive strength (f_c), were defined to be 3MPa and 38 MPa, respectively. In this study, shear transfer coefficient of open crack β_t=0.5 and shear transfer coefficient of closed crackβ_c =0.8.

MISO yield surface is considered well defined by the definition of discrete points representing the numerical relation between applied stresses and corresponding strain values.

As per literature, numerical expression (1) is used to construct the uniaxial compressive stress-strain *curve for lightweight concrete in this study. Figure (5), shows the graphical representation of stresses and corresponding strains obtained from equation (1).*

$$f_c = (2\beta-3)\left\{\frac{\varepsilon_c}{\varepsilon_o}\right\}^4 + (4-3\beta)\left\{\frac{\varepsilon_c}{\varepsilon_o}\right\}^3 + \beta\left\{\frac{\varepsilon_c}{\varepsilon_o}\right\} \quad (1)$$

where, f_c : is the concrete stress, ε_C: is the concrete strain

$$\beta = E_{itm}\frac{\varepsilon_o}{f_c'}$$

For lightweight concrete, Wang et al [14] proposed eqn. (2) to estimate E_{itm}

$$E_{itm} = 2.1684\ f_c'^{0.535} \quad (2)$$

where, f'_c: is the concrete compressive strength, ε_0: is the concrete strain at peak stress, in case of lightweight aggregate concrete, and it is proposed by Almusallam and Alsayed [15] to be calculated by eqn. (3),

$$\varepsilon_o = (65.57 f_c'^{0.44} - 6.748)\times 10^{-5} \quad (3)$$

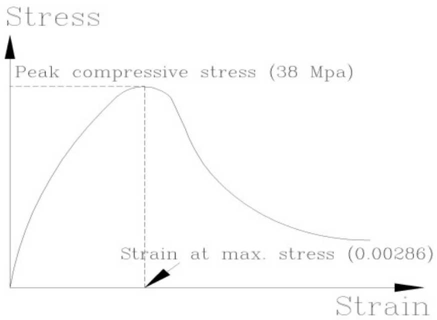

Figure (5) Compressive stress-strain curve for lightweight concrete used in ANSYS model

b.2. Reinforcement material non-linearity

The bilinear kinematic hardening model (BKIN) Constitutive model is sufficiently used to define steel bars material non-linearity (Werasak Raongjant, and Meng Jing [12]).

Bond between concrete and reinforcement is assumed to be perfect and modeling of bond itself is undertaken in this study through shared joints between both Solid65 and Link8 elements.

The bilinear kinematic hardening model (BKIN) was used. Constitutive law for steel behaviour is calculated by Werasak Raongjant, and Meng Jing models [16] shown in eqn. (4).

$$\delta_s = \begin{cases} E_s\varepsilon_s, & \varepsilon_s \le \varepsilon_y \\ f_y + E_s'\varepsilon_s, & \varepsilon_s \succ \varepsilon_y \end{cases} \quad (4)$$

In which δ_s is the steel stress; ε_s is the steel strain; E_s is the elastic modulus of steel; E'_s is the tangent modulus of steel after yielding, E'_s = 0.01 Es; f_y and ε_y is the yielding stress and strain of steel, respectively.

c. Geometrical modeling and finite element meshing

*Numerically modeled columns are typical to those experimentally tested in lab to verify the FEM accuracy. Numerically studied samples are 200 * 200 mm² in cross sectional area and 1500 mm high.Concrete solid continuums are meshed with cubic solid elements having and element size of 20 mm. Similarly, reinforcing bars are 20 mm long link elements, as shown in Figure (6).*

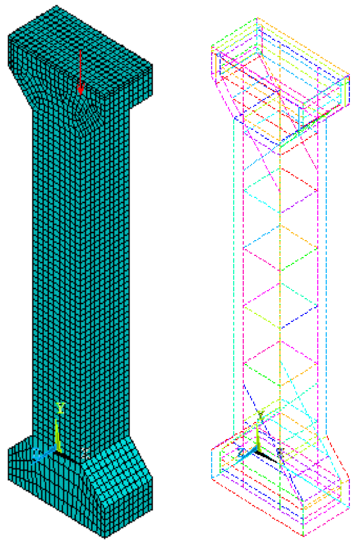

Figure (6.a) Concrete Model Figure (6.b) ReinforcedModel

d. Loading schemes

Two types of loads are considered during analysis. Control specimen is loaded using ramped axial load till failure with a uniform distribution over column cross section. Other specimens are investigated under both ramped axial load till failure simultaneously with an increasing external temperature over time.

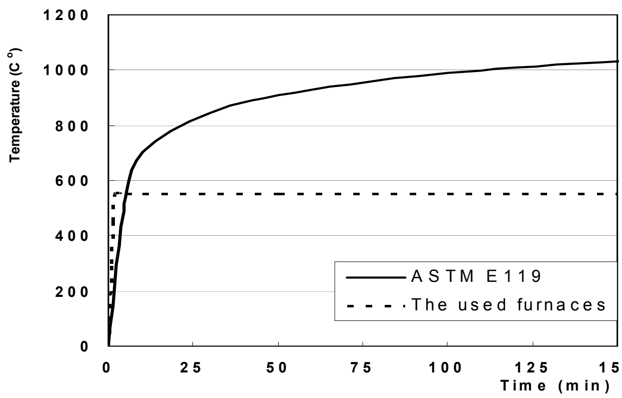

Figure 7. Effective Temperature – time curve

Temperature loading overtime is shown in Figure (7) which is in agreement with ASTM E119 [17]. In both types of specimens, axial loads are applied slowly to the numerical model in order to avoid hardening effect due to rapid loading. In addition, automatic time stepping is enabled to change rate of load application near failure load.

ANSYS supports two types of thermal analysis:

1. *A steady-state thermal analysis determines the temperature distribution and other thermal quantities under steady-state loading conditions. A steady-state loading condition is a situation where heat storage effects varying over a period of time can be ignored.*
2. *A transient thermal analysis determines the temperature distribution and other thermal quantities under conditions that vary over a period of time.*

Thermal analysis in Ansys calculates the temperature distribution and related thermal quantities. Typical thermal quantities of interest are: the temperature distributions, the amount of heat lost or gained, thermal gradients, and thermal fluxes.

Ansys Multiphysics module is used to perform such FE analysis during thermal loading. The basis of thermal analysis in Ansys Multiphysics is the heat balance equation obtained from the principle of conservation of energy.

VERIFICATION OF THE NUMERICAL MODEL

The tested and the ANSYS results of load – concrete strain curves of the four specimens are presented in Figure (8 to 11). Finite element analysis results showed similar trends to the tested results with a deviation nearly about 9% which recommended the proposed numerical model.

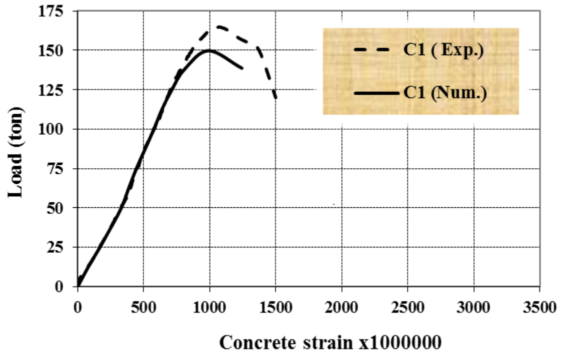

Figure (8) Load – Strain Relationship For Column (C1)

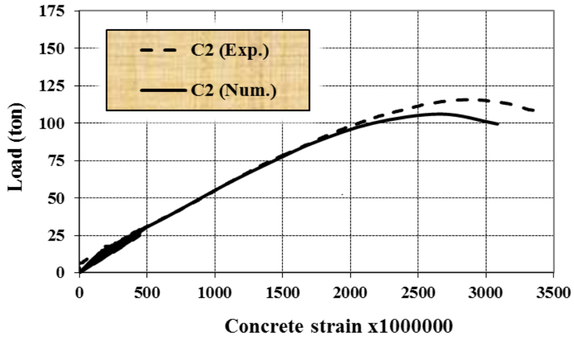

Figure (9) Load – Strain Relationship For Column (C2)

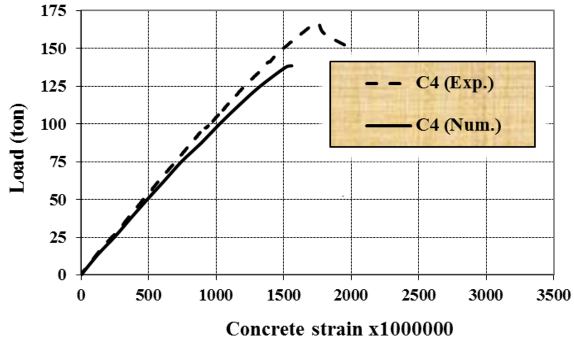

Figure (10) Load – Strain Relationship For Column (C4)

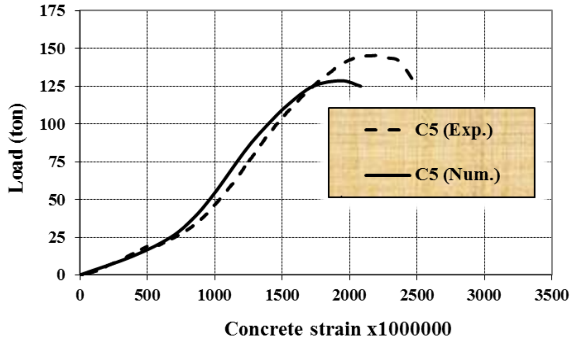

Figure (11) Load – Strain Relationship For Column (C5)

ANALYSIS PROCEDURE (numerical examples)

*A total number of twelve reinforced columns were fabricated and simulated by ANSYS in the current study. Groups A and B consist of columns $C1_{num}$ to $C6_{num}$ with intended concrete compressive strength 30 MPa, while groups B and D consist of columns $C7_{num}$ to $C12_{num}$ with intended concrete compressive strength 40 MPa. All analysis columns in this study were of the same concrete dimensions; 200 * 200 mm² in cross sectional area and length 1500 mmas shown in Figure (1).*

The percentage of reinforcement (μ=1.13%) was used for all columns in the current study. Columns $C1_{num}$, $C4_{num}$, $C7_{num}$ and $C10_{num}$were not exposed to elevated temperature, therefore they were used as control specimens while the remaining eight *columns were exposed to a target temperature of 550℃ for either 90 or 180 minutes. Figure (1) shows the general reinforcement details for the simulated columns.*

The main reinforcing bars were 4 Ø12 (high tensile steel 400/600) for columns of all groups. Stirrups with diameter 8 mm (mild steel 280/420) were detailed to keep the thickness of the concrete cover of the tested columns to be 25 mm which is the minimum thickness that achieve the requirements of the fire resistance of reinforced concrete columns according to the Egyptian code for design and construction of concrete structures (ECP – 203) [13]. The main properties of the used steel bars were listed in Table (2). Table (4) presents the group number, column identification number, main characteristic values, type of concrete and the duration of elevated temperature.

Table (4) Major analysis results

Group	Column ident.	Ultimate load, P_u, (KN)	Stiffness (KN/m)	Toughness (KN mm/mm)	Exposure Duration (hrs)
A	$C1_{num}$	1490	1.059	0.450	----
	$C2_{num}$	1040	0.284	0.120	1.5
	$C3_{num}$	960	0.252	0.107	3.0
B	$C4_{num}$	1380	0.619	0.263	----
	$C5_{num}$	1280	0.493	0.209	1.5
	$C6_{num}$	1410	0.39	0.165	3.0
C	$C7_{num}$	1730	0.80	0.340	----
	$C8_{num}$	980	0.289	0.123	1.5
	$C9_{num}$	890	0.221	0.094	3.0
D	$C10_{num}$	1480	0.684	0.290	----
	$C11_{num}$	1170	0.278	0.118	1.5
	$C12_{num}$	960	0.489	0.207	3.0

ANALYTICAL RESULTS and DISCUSSION

Results of the simulated columns are presented, analyzed and discussed in this section. Topics to be covered include the mode of failure, the load-strain relationships, the ultimate load, the stiffness and toughness of the analysis columns. Table (4) lists the ultimate loads, stiffness and toughness of the simulatedcolumns. The analysis columns showed different structural behavior according to the studied key variables.

□ *Stiffness and Toughness*

The stiffness of the analysis columns can be calculated as the average slope of the ascending part of the load – Longitudinal strain curves. It can be noted from figures (12) to (25) that the slope of the curves is not constant and it decreases 40% to 70% of the ultimate loads of the simulated columns. This stiffness degradation occurs due to the micro-cracking of concrete. Table (4) presented the initial stiffness values of the whole analysis columns. It can be observed that the initial stiffness of the columns of group A ranged between 0.252 kN/m for $C3_{num}$ and 1.059 kN/m for the unheated column, $C1_{num}$. This means that the occurring reduction in the initial stiffness in this group reached 73.2% due to the exposure to elevated temperature for 3hrs of normal strength concrete 30 N/mm^2. On the other hand, the initial stiffness of the light weight columns of the same strength (group B) ranged between 0.39 kN/m for $C6_{num}$ and 0.619 kN/m for the unheated column, $C4_{num}$, i.e. the reduction percentage of the initial stiffness induced by the exposure to 550℃ elevated temperature reached 36.9% in this group and due to the high fire resistance of light weight concrete Moreover, referring to Table (4).

Also, it can be observed that the initial stiffness of the columns of group C ranged between 0.221 kN/m for $C9_{num}$ and 0.80 kN/m for the control column, C7num. This means that the occurring reduction in the initial stiffness in this group reached 72.4% due to the exposure to elevated temperature for 3hrs at normal strength 40 N/mm^2. On the other hand, the initial stiffness of the light weight columns with the strength of group D ranged from 0.490 kN/m for $C12_{num}$ to 0.684 kN/m for the control column, $C10_{num}$, i.e. the reduction percentage of the initial stiffness induced by the exposure to 550 ℃ elevated temperature for 3 hrs reached 28.3% in this group. And this can be explained by the high fire resistance of light weight concrete. Moreover, it is noticed that as the concrete type changed from normal to light weight concrete at the same concrete strength, the rate of loss in stiffness decreased.

Toughness of the analysis columns, which is the ability to absorb the energy through their deformations, is one of the main important characteristics of the structural behavior of the concrete elements. The toughness values can be represented by the total area under the load – compressive strain curves that were calculated numerically. Table (4) presents the toughness values of the analysis columns. Comparing the values given in Table (4), it can be noted that the occurring reduction ranged from 63.9-76.2% in the toughness after analysis of the heated columns as the time of exposure to elevated temperature increased from 1.5 hrs to 3 hrs for groups A and C (normal weight concrete, at strength 30 N/mm² and 40 N/mm²), while the reduction in toughness decreased ranged between 20.5% to 59% in light weight concrete as the time of exposure to elevated temperature increased from 1.5 hrs to 3 hrs for groups B and D at strength 30 N/mm² and 40 N/mm².

□ **Load – Strain Records**

√ **Effect of Concrete Grade**

Figures (12) and (13) illustrate the load- strain relationships of the analysis unheated columns ($C1_{num}$, $C7_{num}$) and ($C4_{num}$, $C10_{num}$) for normal-weight and light weight concrete respectively. In general, for unheated tested columns, it was noticed that the ultimate failure load increased by 13.4% for normal strength concrete ($C1_{num}$ and $C7_{num}$) and 6.75% for light weight concrete ($C4_{num}$ and $C10_{num}$) as the concrete strength increased from 30 N/mm² to 40 N/mm², respectively. This means that normal-weight concrete columns were more sensitive to concrete strength than the light weight concrete columns.

For heated columns at 550℃ for 1.5 hrs, figure (14), it is noticed that the ultimate failure load decreased by 30.2% for normal strength concrete column ($C2_{num}$) comparing with unheated control specimen ($C1_{num}$) at concrete strength 30 N/mm², while it decreased by 43.3% for normal strength concrete column ($C8_{num}$) comparing with unheated control specimen ($C7_{num}$) at concrete strength 40 N/mm². While for heated analysis columns at 550℃ for 1.5 hrs, figure (15). It was noticed that the ultimate failure load decreased by 7.2% for light weight concrete column ($C5_{num}$) compared to unheated control specimen ($C4_{num}$) at concrete strength 30 N/mm², while it decreased by 20.9% for normal strength concrete column ($C11_{num}$) comparing with unheated control specimen ($C10_{num}$) at concrete strength 40 N/mm².

While at 3 hrs exposure duration at 550℃, Figure (16), the ultimate failure load decreased by 35.5% at concrete strength 30 N/mm² to for normal weight concrete comparing with (the unheated control column). While it decreased by 48.5% at concrete strength 40 N/mm² for the same type of concrete.

So it can be noted that, in general the rate of reduction of ultimate failure load of specimens subjected to elevated temperature increased as concrete grade increased from 30 N/mm² to 40 N/mm².

Moreover, from the figures, the rate of loss in stiffness was increased as the exposure duration increased from 1.5 hrs to 3 hrs as the concrete strength increased from 30 N/mm² to 40 N/mm² for normal strength concrete.

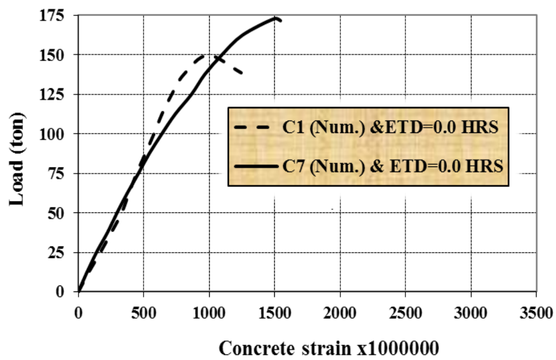

Figure (12) Effect of Concrete Grade on Load –Strain Relationship for Unheated Columns (NWC)

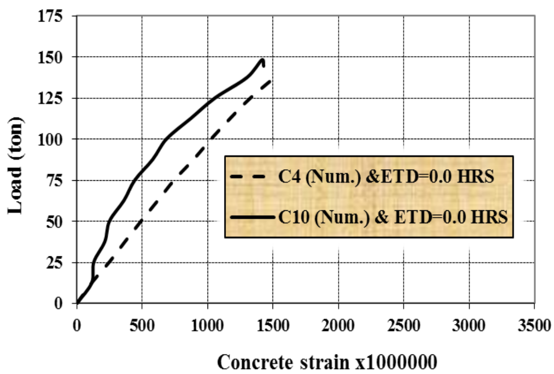

Figure (13) Effect of Concrete Grade on Load –Strain Relationship for Unheated Columns (NWC)

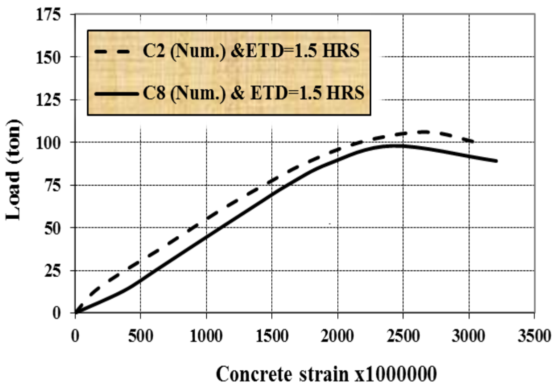

Figure (14) Effect of Concrete Grade on Load –Strain Relationship for Heated Columns (NWC) at 1.5 hrs

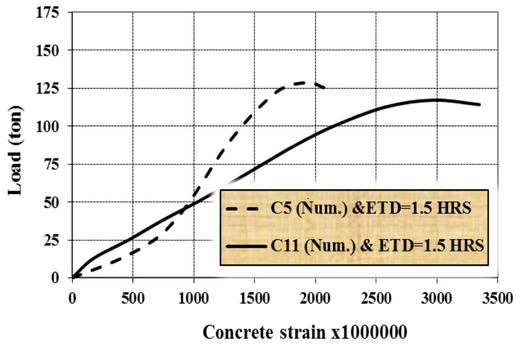

Figure (15) Effect of Concrete Grade on Load – Strain Relationship for Heated Columns (LWC) at 1.5 hrs

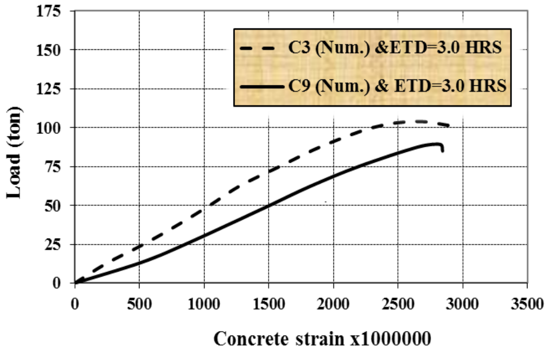

Figure (16) Effect of Concrete Grade on Load - Strain Relationship for Heated Columns (NWC) at 3 hrs

√ ***Effect of Concrete Type***

Figures (17 to 18) illustrate the load- strain relationships of ($C1_{num}$, $C4_{num}$), and ($C7_{num}$, $C10_{num}$) unheated tested columns. It can be noticed that the ultimate load decreased by 7.4% as the concrete type changed from normal weight to light weight concrete at compressive strength 30 N/mm² while it decreased by 14.4% as the concrete type changed from normal weight to light weight concrete at compressive strength 40 N/mm². It can be noted also that, in general, the recorded values of compressive strains for normal weight concrete and low density concrete increased as compressive strength 30 N/mm² to 40 N/mm², respectively.

Figures (19 to 20) illustrate the load- strain relationships of analysis heated columns ($C2_{num}$ and $C5_{num}$) and ($C8_{num}$ and $C11_{num}$) at 550°C for 1.5 hrs, respectively. It can be noticed that the ultimate load decreased from 30.2% to 7.2% compared to unheated analysis columns as concrete type changed from normal weight to light weight concrete respectively, at compressive strength 30 N/mm², while the ultimate load decreased from 43.3% to 20.9% as the concrete type changed from normal weight to low density concrete at compressive strength 40 N/mm².

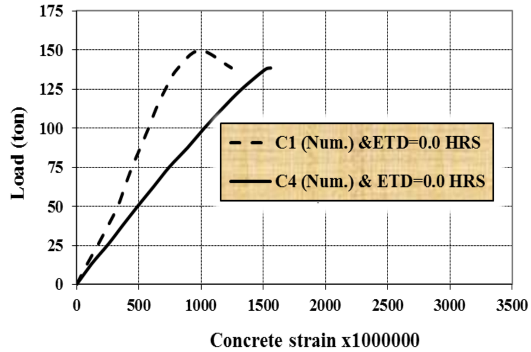

Figure (17) Effect of Concrete Type on Load – Strain Relationship for Unheated Columns at Concrete Strength 30N/mm²

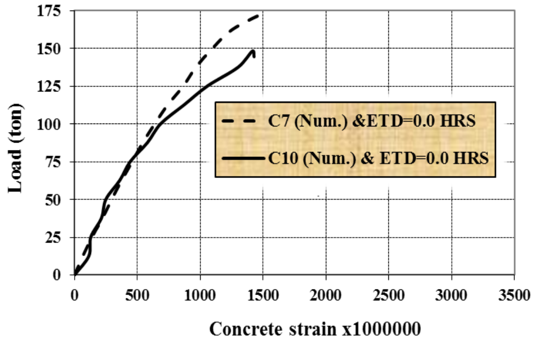

Figure (18) Effect of Concrete Type on Load – Strain Relationship for Unheated Columns at Concrete Strength 40N/mm²

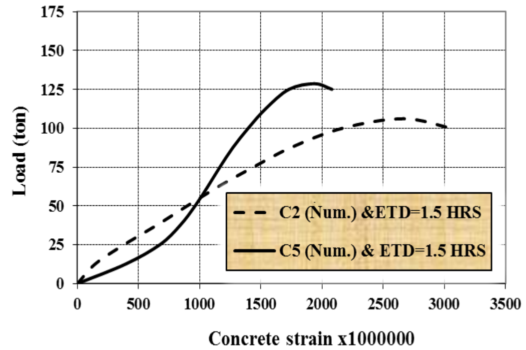

Figure (19) Effect of Concrete Type on Load – Strain Relationship for Heated Columns at 1.5 hrs at Concrete Strength 30 N/mm²

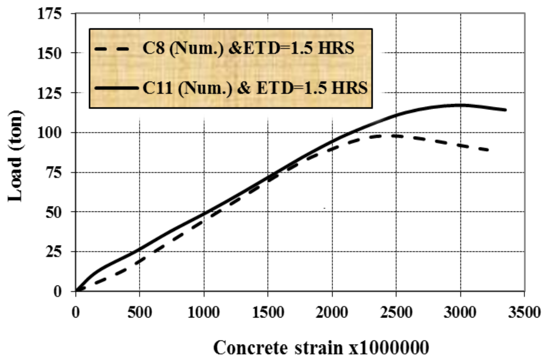

Figure (20) Effect of Concrete Type on Load – Strain Relationship for Heated Columns at 1.5 hrs at Concrete Strength 40 N/mm²

Figure (21) illustratethe load- strain relationships of analysis heated columns ($C9_{num}$ and $C12_{num}$) at 550℃ for 3 hrs. It can be noticed that the ultimate load decreased from 48.5% to 35% compared to unheated analysis columns as the concrete type changed from normal weight to light weight concrete respectively, at compressive strength 40 N/mm².

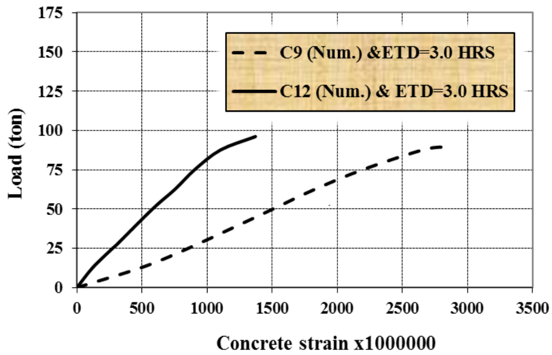

Figure (21) Effect of Concrete Type on Load – Strain Relationship for Heated Columns at 3 hrs at Concrete Strength 40N/mm²

Based on the results, it is noted that the unheated normal weight concrete columns showed larger ultimate loads when compared to the light weight concrete columns thatpossessed the same compressive strength.

On the contrary, the heated low density concrete columns showed larger ultimate loads when compared to the normal weight concrete columns possessed the compressive strength.

Also comparisons between the results of columns of normal-weight concrete and columns of light weight concrete were executed. It can be seen that the percentage of reduction in the ultimate capacity of analysis (unheated) columns increased as the concrete type changed from normal to light weight concrete as the concrete strength increased from 30 N/mm² to 40 N/mm², while the percentage of reduction in the ultimate capacity of analysis (heated) columns was decreased as the concrete type changed from normal to light weight concrete as the concrete strength increased from 30 N/mm² to 40 N/mm².

Also it was observed that the stiffness (the slope of the ascending part of the load–longitudinal strain curve) of the light weight concrete columns decreased compared to the normal weight concrete columns forconcrete strength 30 kN/mm².

√ ***Effect of Exposure Period***

Figures (22) to (25) illustrate the load- strain relationships of analysis columns of groups A, B, C and D, respectively. From Figure (22), the ultimate failure load decreased by (30% to 35.5%) as the exposure duration increased from 1.5 hours to 3 hours at 550ºC, at concrete strength 30 N/mm². Also it was observed that the rate of reduction of stiffness increased as the exposure duration increased. Also, Figure (23) indicated that the ultimate failure load decreased by (7.2%) compared to unheated analysis column $C4_{num}$ after exposed to elevated temperature for 1.5 hrs at light weight concrete at compressive strength 30 N/mm². While from Figure (24) and (25), it can be noticed that the ultimate failure load decreased by (43.3% to 48.5%) and (21% to 35.2%) as the exposure duration increased from 1.5 hours to 3 hours at 550ºC, as the concrete type changed from normal weight concrete to light weight concrete at the same compressive strength 40 N/mm², respectively.

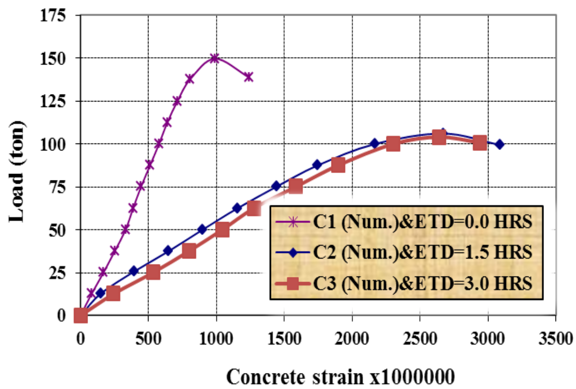

Figure (22) Effect of Exposure Duration on Load – Strain Relationship for (NWC) Columns using Concrete Strength 30 N/mm²

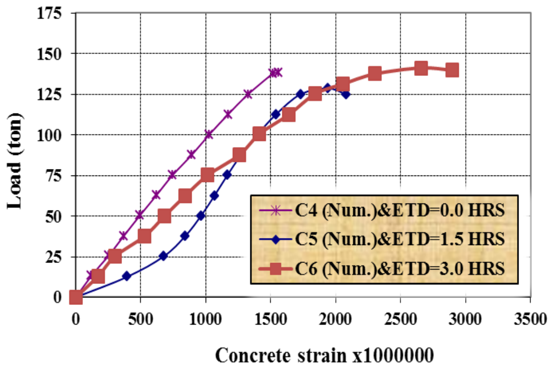

Figure (23) Effect of Exposure Duration on Load – Strain Relationship for (LWC) Columns using Concrete Strength 30 N/mm²

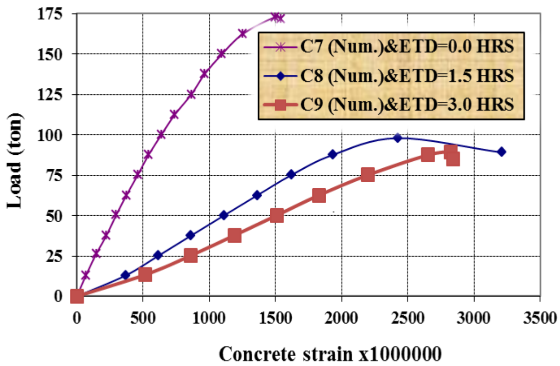

Figure (24) Effect of Exposure Duration on Load – Strain Relationship for (NWC) Columns using Concrete Strength 40 N/mm²

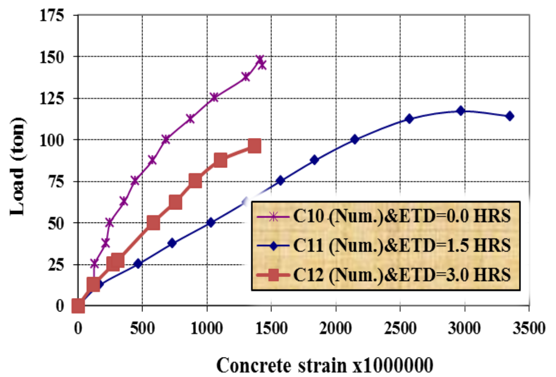

Figure (25) Effect of Exposure Duration on Load – strain Relationship for (LWC) Columns using Concrete Strength 40 N/mm²

CONCLUSIONS

Based on the results the following conclusions could be drawn:

1. *The unheated light weight columns showed a slight reduction in the load carrying capacity, stiffness and toughness when compared to the normal-weight concrete columns.*
2. *The percentage of reduction in the ultimate capacity of analysis (unheated) columns increased from 13.4% in normal weight concrete to 6.75% in light weight concrete as the concrete strength increased from 30 N/mm² to 40 N/ mm².*
3. *The percentage of reduction in the ultimate capacity of analysis (heated) columns at 1.5 hrs decreased as the concrete type changed from normal to light weight concrete.*
4. *For the same grade of strength, the rate of reduction in ultimate capacity of the heated columns was decreased as the concrete type changed from normal weight concrete to light weight concrete.*
5. *The rate of loss in stiffness increased as the exposure duration increased from 1.5 hrs to 3 hrs as the concrete strength increased from 30 N/mm² to 40 N/mm² for normal strength concrete, while in light weight concrete the rate of loss in stiffness decreased.*

REFERENCES

[1.] *Abu Al-magd, S., and Hosny, H., „Fires in Concrete Structures", Egypt, No. 1514, 1994.*

[2.] *Saad, M.; Abo El-Enien, S. A.; Hanna, G. B. and Kotkata, M. F., „Effect of Silica Fume on the Phase Composition and Microstructure of Thermally Treated Concrete", Cement and Concrete Research, Vol. 26, 1996.*

[3.] *Sait, M. and Turan, O., „Effect of Elevated Temperature on the Residual Mechanical Properties of High Performance Mortar", Cement and Concrete Research, Vol. 32, 2002.*

[4.] *Abd El-Razek, M. M.; Hodhod, H. A.; Ragab, A. M. and Rashad, A. M., „Size and Concrete Cover Effect on the Residual Capacity of Loaded Rectangular Short RC Column Models Exposed to Elevated Temperature", Proceeding of International Conference on Future Vision and Challenges for Urban Development, Egypt, December, 2004.*

[5.] *Issa, M. S. and Anwar, H., „Structural Behavior of High Strength Reinforced Concrete Columns Exposed to Direct Fire", Proceeding of International Conference on Future Vision and Challenges for Urban Development, Egypt, December, 2004.*

[6.] *Mohamed A. Khafaga, „ A Novel Approach to Evaluate the Effect of Elevated Temperatures on the Behavior of Reinforced Concrete Columns Subjected to Uni-axial Bending". Engineering Research, Faculty of Engineering, Helwan University, Vol (109), Febrary 2007.*

[7.] *Demirbog, R. and Gu, R., (2003), „The effects of expanded perlite aggregate, silica fume and fly ash on the thermal conductivity of lightweight concrete", Cement and Concrete Research, 33, pp.723 –727.*

[8.] *Kayali, O., (2008), „Fly ash lightweight aggregates in high performance concrete", Construction and Building Materials, 22, pp. 2393 – 2399.*

[9.] *Choi, Y. W., Kilm, Y., J., Shin, H. C. and Moon, H. Y., (2006), „An experimental research on the fluidity and mechanical properties of high-strength lightweight self-compacting concrete", Cement and Concrete Research, 36, pp. 1595 – 1602.*

[10.] Haque, M. N., Al- Khaiat H. and Kayali, O., (2004), „Strength and durability of lightweight concrete", Cement and Concrete Composites, 26, pp. 307 – 314.

[11.] Mohamed A. Khafaga, „Shear Behavior of Reduced Weight Reinforced Concrete Beams". Journal of Engineering Science, JES, Faculty of Engineering, Assuit University, Vol (40), No. (1), January 2012.

[12.] Werasak Raongjant, Meng Jing "Analysis Modelling of Seismic Behaviour of Lightweight Concrete Shear Walls" Proceedings of the International Multi Conference of Engineers and Computer Scientists 2009 vol II IMECS 2009, March 18-20, 2009, Hong Kong.

[13.] Egyptian Code for Design and Construction of Concrete Structures (ECP 203), Housing and Building Research Center, Egypt, 2007.

[14.] Wang P.T. Shah S.P. Naaman A.E., "Stress-strain curve for normal and lightweight concrete in compression". American Concrete Ins. J. Proceeding, 75(`11), 1978. Pp. 603-614

[15.] Almusallam T.H., Alsayed H., "Stress-strain relationship of normal, high-strength and lightweight concrete". Magazine of concrete research, 47 (169), 1995, pp. 39-43.

[16.] Werasak Raongjant, and Meng Jing, "Analysis Modelling of Seismic Behaviors of Lightweight Concrete Shear Walls" International Multi Conference of Engineers and computer Scientists 2009 Vol II IMECS 2009, March 18-20, 2009, Hong Kong.

[17.] ASTM E119, „Standard Method for Fire Tests of Building Construction and Materials", American Society for Testing and Materials, Philadelphia, 2000.

ACTA Technica CORVINIENSIS
BULLETIN OF ENGINEERING
ISSN:2067-3809