International Journal For Advanced Research In Science & Technology
A peer reviewed international journal
ISSN: 2457-0362
www.ijarst.in

Safety and Stability Study of Concrete High-Rise Buildings Under Local Fire

Reza Fereydouni1, Dr. Abdullah Keyvani2

1- MSc in Civil and Structural Engineering, Shahid Madani University of Azerbaijan, Tabriz, Iran
reza.f510@yahoo.com
2- Faculty of Civil and Structural Engineering, Shahid Madani University of Azerbaijan, Tabriz, Iran
abdullahkeyvani@gmail.com

Abstract:

The subject of this project is the reaction and behavior of reinforcements against heat caused by fire. Therefore, as the temperature rises, the concrete surface coating, which has a thickness of between 30 and 50 mm, loses its protective ability and is destroyed due to internal reactions of concrete caused by vapor pressure of internal moisture and loss of cement adhesion at high temperatures. With the occurrence of this phenomenon, the longitudinal surface of the reinforcements is exposed to heat and the effect of this phenomenon on the reinforcements shows its effect by causing local creep and flow of steel and ultimately the destruction of the element. In this research, an 18-story concrete building in operation was selected as a study case and the structural model was analyzed and investigated using SAP 2000 (V 11.0) software based on the behavior of concrete and steel under high heat. In order to determine the depth of heat penetration in the concrete and in the area where the reinforcements are located, a fire in a reinforced concrete column was modeled with ANSYS software, considering an assumed temperature of 500 degrees Celsius. The process of changes in the heat penetration rate from the surface to the depth was such that the temperature in the cover area and the area where the reinforcements are located was determined to be about 458 degrees Celsius, and in the middle 1/3 of the column, the concrete temperature was determined to be about 400 degrees Celsius. In order to understand the performance of the structure, the first and second floors of the building were placed under fire conditions assuming temperatures of 600 and 500 degrees Celsius, and the behavior of the structure was evaluated with respect to the location and type of plastic joints. Considering that the failure of the beams had a local effect on the performance of the first and second floors of the structure, it did not show much effect on the overall instability of the building. For this purpose, a column located in the central area of the building (column C7) that bears more load than other columns and is in the failure phase due to the formation of a plastic hinge was selected and removed from the load-bearing system of the structure and the performance of the building was examined. The removal of the column and the consequent increase in bending moments in the beams leading to the column due to the increase in the length of the beam and the increase in the values of axial forces between 15-20% and shear forces and bending moments between 40-50% in the central columns that were not connected to the shear walls and especially in the upper column aligned with the aforementioned column caused progressive failure in the upper floors of the building.

Keywords: Fire - High-rise buildings - Nonlinear region - Plastic joints - Progressive failure

1-Introduction

One of the major advantages of reinforced concrete structures over steel structures is their greater resistance to fire, which has led to the increased use of concrete structures in the construction of buildings, especially high-rise buildings with commercial or strategic uses. However, this assumption cannot be a sufficient reason for the 100% safety of these materials when exposed to fire. Because any material that is subjected to heat, the physical and chemical properties of the material or materials that make up the material undergo significant changes to reduce or increase quantitative and qualitative values. These changes can be in the form of changes in mechanical strength or modulus of elasticity or changes in chemical properties, such as loss of adhesion. Therefore, examining the behavior of concrete materials and especially structures made of these materials when exposed to fire is of great importance and will provide useful information in modern design methods [1].

The important issue in this project is the reaction and behavior of reinforcements against heat caused by fire. Therefore, as the temperature rises, the concrete surface coating, which has a thickness of between 30 and 50 mm, loses its protective ability and is destroyed due to internal reactions of concrete caused by internal moisture vapor pressure and loss of cement adhesion at high temperatures. With the occurrence of this phenomenon, the longitudinal surface of the reinforcements is exposed to heat and the effect of this phenomenon on the reinforcements shows its effect by causing local creep and flow of steel and ultimately the destruction of the element [2].

When a material ignites, the fire surrounds the combustible material until it completely surrounds it. This process of fire spreading on the surface of the material depends on the fuel composition, orientation, material ratio, prevalent heat and air source. When sufficient air is available, the amount of energy released from a fire is increased by the common heat (heat released due to burning) and the properties of the fuel, especially the heat of combustion (the temperature at which the material ignites when it reaches that temperature) and the latent heat of vaporization (the temperature at which the material loses its surface moisture). Formula (1) shows the relationship between these variables [3].

(1)

: Amount of energy released per unit area of fuel

: Common heat per unit area of fuel (heat flow)

: Latent heat of vaporization

: Heat of combustion

The total burning rate per unit area in a typical building is about 20 to 40 and the heat released per unit area is 320 to 640 [4].

In a fire, if the area of the fire increases, the amount of energy released increases to a maximum value. After a period of time, which depends on the type of fuel, the completeness of the fuel combustion, the amount of oxygen in the area, and the presence of a suppressant, the process of increasing the temperature decreases in proportion to the loss of fuel [5].

During a fire, the combustion of fuel occurs at an early stage, and the release of energy occurs very quickly. Over time, the burning rate decreases until it finally becomes calm [6].

The overall goal of designing the building skeleton and protective plans is to increase the safety factor of the building in proportion to the type of accident. In a system designed to deal with unexpected accidents, the following factors that can help reduce the destruction and collapse of the structure due to fire should be considered [7].

The September 11 incident in the World Trade Towers and the explosions on the upper floors forced structural design engineers to investigate the cause of the collapse of these two buildings. The main cause of the destruction was the occurrence of a fire phenomenon caused by an explosion in the steel core in the middle part of the structure, caused by explosives and flammable materials. Due to the high temperature,

International Journal For Advanced Research In Science & Technology
A peer reviewed international journal
www.ijarst.in
ISSN: 2457-0362

the steel used reached its critical temperature (the temperature at which the physical and chemical properties of the material change qualitatively and quantitatively), and as the steel flowed, local buckling occurred in the middle steel structure, which caused the destruction of the upper floors, the application of impact and overload caused by the debris of the upper floors on the lower floors, and the loss of the load-bearing resistance of the elements of the lower floors of the structure in the two buildings. The spread of fire caused by the explosion is shown in Figure (1) [8].

Figure 1- Destruction of the World Trade Center towers in the United States

Another famous building that was almost completely destroyed by fire was the Federal Building in Oklahoma, USA. The cause of the fire was a bomb explosion in a car parked next to the building, as shown in the figure [9]. Most concrete is porous and contains a certain amount of water, a large part of which is the water from the mixing and making of the concrete paste that is present in it during the time it takes to gain final strength. The other part is the moisture in the air that is absorbed by the concrete surface from the surrounding environment, which is collectively known as the physical (free) water of the concrete. This water begins to evaporate at a temperature of about 100-120. On the other hand, the chemical or molecular water of the aggregate and cement particles, which constitutes the second part of the water in the concrete, evaporates at a temperature of about 400-450. As the physical water in concrete evaporates in the early stages, the pore water pressure in it increases, which increases the strength of the concrete in the initial moments. However, as the temperature continues to increase and the water in the concrete evaporates, the pressure caused by evaporation exerts a force from the inner core to the outer surface of the concrete. This increase in pressure causes mechanical forces to occur inside the aggregates, and as a result of the application of these forces, the concrete bursts. The bursting causes the surface pieces of the concrete to separate and be thrown outwards. As a result, a progressive failure occurs in the concrete structure, and if the spread or scope of this phenomenon is wide, it has very serious effects on the strength of the structure or concrete element [10]. In the last two decades, research has been conducted on the performance of structures, especially concrete structures, against fire. One of these studies was conducted at the University of Sheffield, England, by the Department of Civil Engineering under the supervision of Dr. Hong, Professor Burgess, and Professor Planck. In the last two decades, research has been conducted on the performance of structures, especially concrete structures, against fire. One of these studies was conducted at the University of Sheffield, England, by the Department of Civil Engineering under the supervision of Dr. Hong, Professor Burgess, and Professor Planck. Another study on the behavior of reinforced concrete elements was conducted at Ching Yun University, Taiwan, by Dr. Hsu and Dr. Lin. This project investigated the residual strength, ductility, and cracking capacity of reinforced concrete beams under fire [11].

International Journal For Advanced Research In Science & Technology
A peer reviewed international journal www.ijarst.in
ISSN: 2457-0362

2- Modeling a reinforced concrete building to investigate the performance of a structure against fire

The study example in this project is an eighteen-story reinforced concrete building. This building is in operation. The first and second floors are for parking and the other floors up to the eighteenth floor of the building are for service use. The building skeleton is a torsional frame and shear wall with medium ductility, and the roof load distribution system is a cement block beam. The location of the structural columns and the spacing of the spans are shown in Figure (1). Also, the details of the beams and columns of the structure are shown in Tables (1).

شماره قاب و طبقات استفاده شده	ارتفاع	عرض
FRAME A-G : 1-2-3-4-5-6	0.65	0.55
FRAME A-G : 7-8-9-10-11-12	0.70	0.55
FRAME A-G : 13-14-15	0.60	0.55
FRAME A-G : 16-17-18	0.55	0.50
FRAME 1-5 : 1-2-3-4-5-6	0.65	0.55
FRAME 1-5 : 7-8-9-10-11-12	0.70	0.55
FRAME 1-5 : 13-14-15	0.60	0.55
FRAME 1-5 : 16-17-18	0.55	0.50
تیرهای کنسول	0.40	0.50

Table 1- Details of concrete building beams

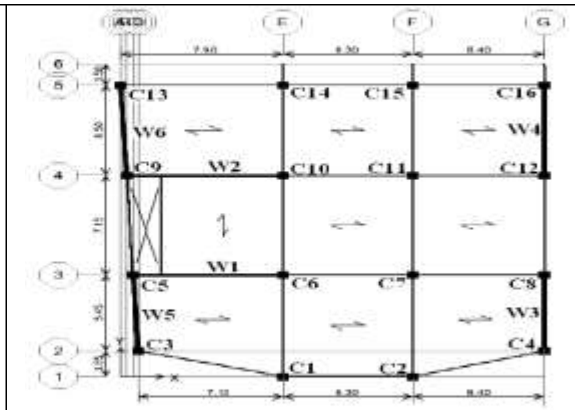


Figure 2 - Building plan and typology of structural columns

- Specifications of materials used in the concrete building under study

The calculated and consumed reinforcements in the structural skeleton are of type A-III and the concrete used in the building skeleton has a characteristic strength of 28 days. The modulus of elasticity of concrete is equal to and the modulus of elasticity of steel is equal to and has been used in the design of the concrete structure according to the ACI318-99 code.

- Modeling of an eighteen-story concrete building

According to the existing structural drawings of the aforementioned building, the concrete skeleton of the building has been modeled in accordance with the plan presented in Figure (2) in SAP software (V 11.0). The cross-section specifications of the beams and columns, as well as the shear walls and consumed materials have been introduced to the desired model with the help of information available in the executive drawings and finally the building model has been formed as shown in Figure (2).

International Journal For Advanced Research In Science & Technology
A peer reviewed international journal
ISSN: 2457-0362
www.ijarst.in

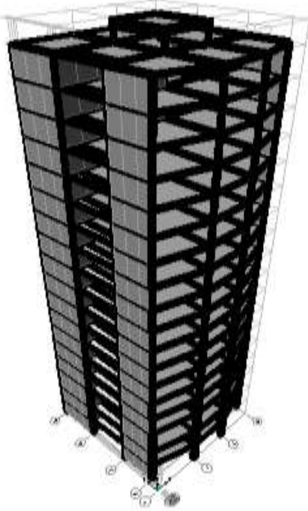


Figure 2- Three-dimensional model of the building in SAP software

تیپ ستونها و طبقات استفاده شده	تعداد میلگرد در هر جهت	شماره میلگرد	طول (m)	عرض (m)
C1---1-2-3	6	280	0.8	0.8
C1---4-5	5	280	0.7	0.7
C1---6	5	280	0.6	0.6
C1---7-8-9-10	5	250	0.6	0.6
C1---11-12-13-14	4	250	0.55	0.55
C1---15-16-17-18	4	250	0.5	0.5
C2---1-2-3	7	280	0.9	0.9
C2--- 4	7	280	0.8	0.8
C2--- 5-6	7	280	0.7	0.7
C2---7-8	5	280	0.7	0.7
C2--- 9-10-11-12	5	280	0.6	0.6
C2---13-14-15	4	250	0.55	0.55
C2---16-17-18	4	250	0.5	0.5
C3---1-2-3	7	280	1	1
C3--- 4	7	280	0.9	0.9
C3--- 5	7	280	0.8	0.8
C3--- 6-7	6	280	0.8	0.8
C3--- 8-9	5	280	0.7	0.7
C3---10	5	280	0.6	0.6
C3---11-12	5	250	0.6	0.6
C3---13-14-15	4	250	0.55	0.55
C3---16-17-18	4	250	0.5	0.5
C4---1-2-3	7	280	0.9	0.9
C4--- 4-5	7	280	0.8	0.8
C4--- 6-7	6	280	0.8	0.8
C4--- 8-9	5	280	0.7	0.7
C4---10	5	280	0.6	0.6
C4---11-12-13	5	250	0.6	0.6
C4---14-15-16	4	250	0.55	0.55
C4---17-18	4	250	0.5	0.5

Table 2- Concrete building columns details

- Applied load combination for analyzing the structure under fire

The study of the behavior of the structure with respect to the fire phenomenon is carried out in the case of operational loads, from this, the dead load applied to the structure must be applied completely and without a coefficient to the structure. In calculating the mass of the desired structure, we consider the live load by applying a coefficient of 0.25 for the loading combination. Finally, the desired load combination according to equation (2) is as follows:

$$DL+0.25LL \qquad (2)$$

- Material specifications

. The material specifications of the desired column are shown in Table (3).

Table 3- Material specifications of C7 concrete column

Compressive strength of concrete (28 days) $f_c'(kg/cm^2)$	Modulus of elasticity Concrete $E_c(kg/cm^2)$	Yield limit of reinforcement $f_y(kg/cm^2)$	Yield limit of stirrups $f_y(kg/cm^2)$	Modulus of elasticity Steel $E_s(kg/cm^2)$
250	2.38×10^5	4000	3000	2.38×10^6

International Journal For Advanced Research In Science & Technology
A peer reviewed international journal
ISSN: 2457-0362
www.ijarst.in

- Column modeling in ANSYS software

To model a concrete column, we first define the concrete specifications in the software. For concrete modeling, we use the SOLID65 element to assign specifications. The SOLID65 element is one of the default elements of the program that allows the user to define reinforced concrete. This 8-node element allows us to define reinforcements in different coordinate directions. In the model in question, we have omitted the use of reinforcement because in this project, we are investigating how the heat spreads and distributes in the column cross-section. Considering the discussion of analyzing the model under the effect of heat, the assumed temperature is defined in terms of 733 degrees Kelvin in the load specification definition section in the structural loads sub-branch in the form of a thermal load on the element surface.

- Thermal analysis of the model in ANSYS software

In order to measure the heat in different layers of the concrete column and especially in the area where the reinforcements are located, whose location is shown in Figure (4), we analyze the model by assuming a temperature of 500 at the column surface and a fire duration of 260 minutes using the ANSYS program.

In order to show the location of the reinforcements, whose modeling has been omitted in the column in question, we use 5 cm mesh samples in the first row of the division to mesh the column. After that, we increase the dimensions of the other meshes to 10 cm and repeat this process up to a depth of 50 cm of the column axis. With this meshing process, the heat that affects the reinforcing reinforcements can be obtained. According to the amount of heat generated in that area, the amount of reduction in the yield strength of the steel can be determined to be introduced in the new material specifications after the fire. The results of the thermal analysis in the target column are shown as a colored contour in a line in Figure (5).

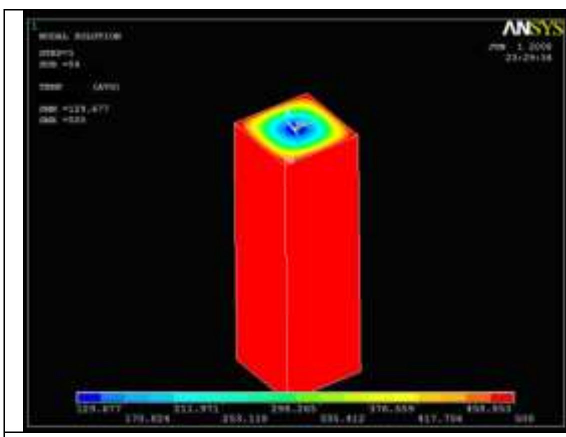


Figure 5- Distribution and spread of heat from the surface to the depth of the target column

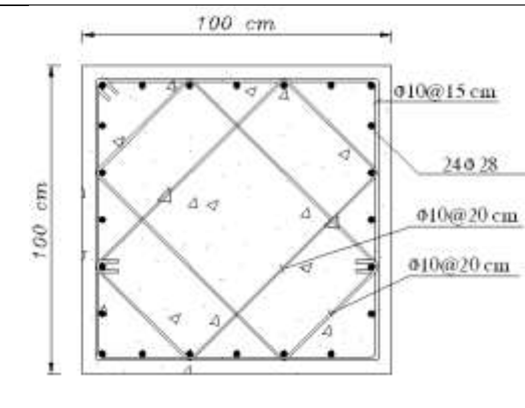


Figure 4-Cross section and position of reinforcement in the target column

By calculating the approximate distance of each area relative to the central axis of the column and the temperature corresponding to that area, it is possible to determine the effective temperature values at the location of the reinforcements and a part of the concrete sample that causes a decrease in the strength of the concrete. According to the color contour of the heat distribution in the cross-section of the sample, the temperature in the center of the sample is approximately 129, within 10 centimeters of the central axis of

International Journal For Advanced Research In Science & Technology
A peer reviewed international journal
www.ijarst.in
ISSN: 2457-0362

the sample, the temperature is approximately 170, within 20 centimeters of the central axis of the sample, the temperature is approximately 211, within 25 centimeters of the central axis of the sample, the temperature is approximately 253, within 30 centimeters of the central axis of the sample, the temperature is approximately 294, within 35 centimeters of the central axis of the sample, the temperature is approximately 335, within 40 centimeters of the central axis of the sample, the temperature is approximately 376, within 45 centimeters From the central axis of the sample, the temperature is about 417 and within 45 centimeters from the central axis of the sample, the temperature is about 458. The distribution of heat in the cross section of the concrete column is shown in Figure (6).

According to the heat distribution process in a concrete column with a surface temperature of 500, the effective temperature on the reinforcement at a depth of 5 centimeters is about 458. With the destruction of the concrete surface due to heat and the reduction of the resistance of the steel of the inner layers, the concrete is affected by heat, and with the loss of the resistance of the steel, the damage occurs progressively in the concrete area. For this purpose, in order to increase the reliability factor in the analysis and design stages of the structure under fire conditions, we assume the effective temperature on the concrete area to be about 417, considering the penetration depth at a distance of 40 centimeters from the central axis, and we extract the mechanical properties of the steel and concrete according to the desired temperatures and the presented graphs. The reduced values of these properties are shown in Tables (4) and (5). Considering that in this project, the first and second floors of the building are examined with the assumption of two different temperatures, that is, the first floor with a temperature of 650 and the second floor with a temperature of 500, for this purpose, considering the process of heat distribution in the concrete column in question, assuming a temperature of 650 on the column surface, the temperature in the area where the reinforcements are located is about 600 and in the concrete area is about 542. So, we determine the reduced values of the mechanical properties of the materials in the first floor by assuming the desired temperatures. The reduced values of these properties are shown in Tables (6) and (7).

Consumable Materials	$E_c (kg/cm^2)$ Modulus of Elasticity of Concrete	$f_c' (kg/cm^2)$ Compressive Strength of Concrete (Cylindrical Specimen)
Concrete	$1.88 \times 10^5 \, kg/cm^2$	$158 \, kg/cm^2$

Table 5- Mechanical properties of concrete at a temperature of 500 based on the experiment

Consumable Materials	$E_s (kg/cm^2)$ Modulus of Elasticity of Steel	$F_y (kg/cm^2)$ Yield Strength of Steel A-III
Steel فولاد	$0.8 \times 10^6 \, kg/cm^2$	$2000 \, kg/cm^2$

Table 4- Temperature distribution in the target column

Consumable Materials	$E_c (kg/cm^2)$ Modulus of Elasticity of Concrete	$f_c' (kg/cm^2)$ Compressive Strength of Concrete (Cylindrical Specimen)
بتن	$1.70 \times 10^5 \, kg/cm^2$	$130 \, kg/cm^2$

Table 7-Mechanical properties of concrete at 650 according to experiment

Consumable Materials	$E_s (kg/cm^2)$ Modulus of Elasticity of Steel	$F_y (kg/cm^2)$ Yield Strength of Steel A-III
فولاد	$1.57 \times 10^6 \, kg/cm^2$	$2600 \, kg/cm^2$

Table 6-Mechanical properties of steel at 500 according to Euro code

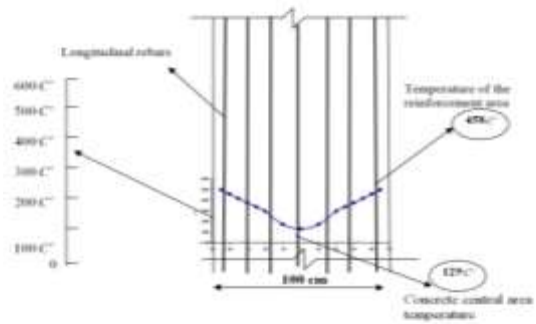


Figure 6-Temperature distribution in the target column

- Introduction of material properties and determination of stress-strain curves of concrete and steel under fire

After determining the values of mechanical properties of materials at temperatures of 500 and 650, the stress-strain curves of materials under fire were drawn by SAP software. These curves were assigned to concrete materials and longitudinal reinforcements and transverse reinforcements (tie-downs) to study the behavior of the structure.

In figures (7) and (8), how to introduce material properties and stress-strain curves under fire is shown.

International Journal For Advanced Research In Science & Technology
A peer reviewed international journal
ISSN: 2457-0362
www.ijarst.in

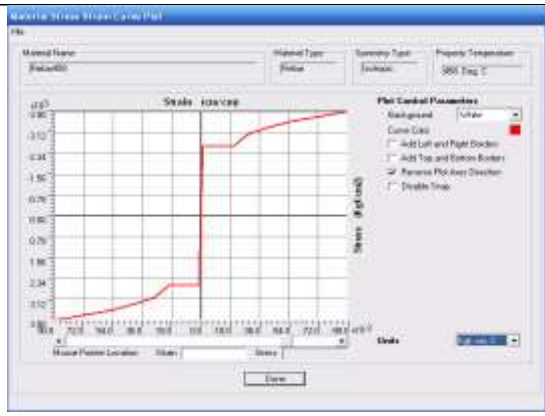
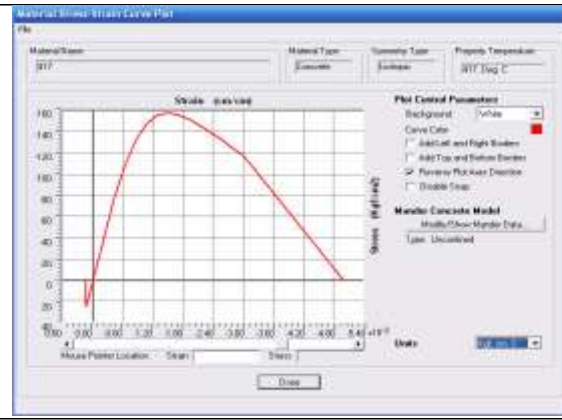
	

Figure 8- Stress-strain curve of steel at a temperature of 500	Figure 7- Stress-strain curve of concrete at a temperature of 500

3- Nonlinear static analysis and analysis of the structure

The building in question is analyzed according to the combination of operational loads (DL+0.25LL) and the application of fire conditions on the first and second floors. Figures (9) to (11) show the location of the structural elements and the stages of formation of plastic joints, as well as the overall deformation of the structure after the fire. As can be seen in Figure (10), after the fire, those points or areas that change their performance relative to the existing conditions appear as plastic joints in the structural elements. Considering the type of joints and their location on the load-deformation diagram, the formed plastic joints are analyzed and examined in four groups.

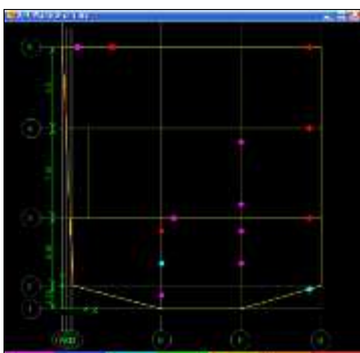
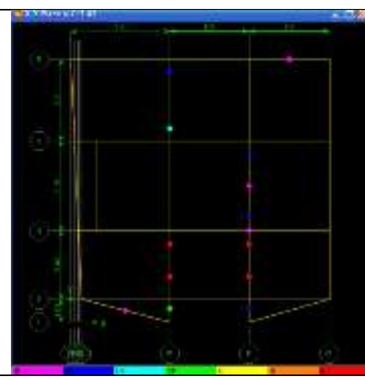
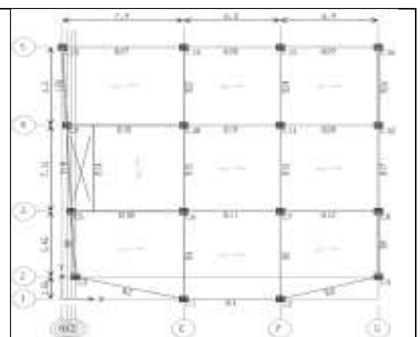
		

Figure 11- Formation of plastic joints on the second floor of the building	Figure 10- Formation of plastic joints on the first floor of the building	Figure 9- Building plan

The formation of these joints in the beams in question confirms the performance of the beam in the failure phase and shows that the possibility of destruction and failure of this group of elements by applying fire conditions is probable. However, considering that the failure in the beams and roof of the building on the first and second floors has a local effect on the performance of the structure and their failure will have a

International Journal For Advanced Research In Science & Technology
A peer reviewed international journal
www.ijarst.in
ISSN: 2457-0362

less effect on the occurrence of progressive damage than the columns. Therefore, examining the performance of the structure due to the failure of the column or columns in terms of the progressive effect and the spread of damage is of great importance.

. In Figures (12) and (13) you can see the direction of the deformation of the structure.

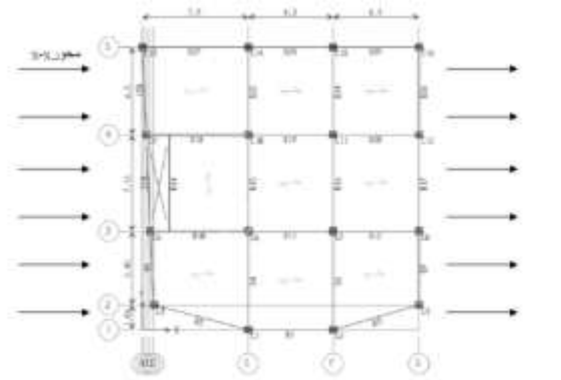
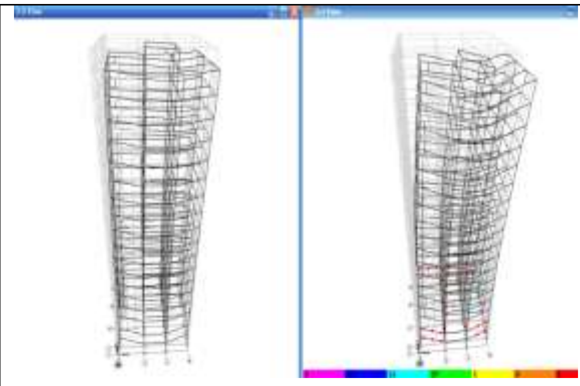
	

Figure 13- Direction of the overall deformation of the structure after fire	Figure 12- Change in the overall shape of the structure before fire (a) and after fire (b)

Due to the area enclosed between axes A and F, as the structure deforms, this area moves towards the central part of the structure, and since it has a greater weight than the area enclosed between F and G in terms of the amount of load, the building deforms along the x-axis.

Considering the overall deformation process of the structure and the tendency of the structure to shift towards the central part, that is, column C7, the performance of the structural elements is examined in terms of axial forces, shear and bending moment of the columns of the first and second floors, as well as the beams connected to column C7.

3-1- Performance of building beams after fire

This movement leads to the deformation of the two beams connected to it, which themselves have plastic joints in the middle and end areas. Considering that column C7, which is located at the intersection of two axes F and 3, is considered a central column and has allocated a large share of the load-bearing surface of the structure.

Due to the large number of beams on the first and second floors of the building, beams connected to column C7, which are located in the central part of the building, were selected to investigate and compare the performance of the beams after the fire, and among them, beam B6 was selected as one of the load-bearing beams, and its performance in terms of force changes was discussed and examined.

- Investigation of the effect of fire on the shear force (V2) and bending moment (M3) of beam B6

As can be seen in Figures (14) and (15), the distribution of shear forces in beam B6 before and after the fire has followed a uniform course. Considering that the beam in question is a load-bearing beam and is

International Journal For Advanced Research In Science & Technology
A peer reviewed international journal
ISSN: 2457-0362
www.ijarst.in

located perpendicular to the direction of deformation of the structure, after analyzing and redistributing the forces, only the amount of forces has increased, and in practice, the lateral deformation of the building does not have much effect on the displacement of the initial and final points of the beam in question. For this reason, the trend of force changes depends on the non-uniform difference of node displacements, and the increase in shear forces and bending moment in the floors has taken a uniform trend.

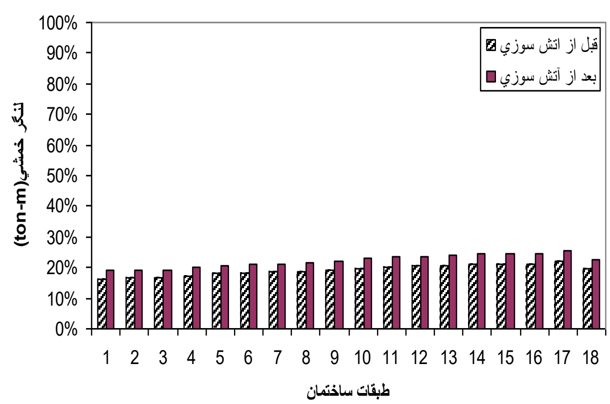


Figure 15 - Bending moment (M3) of beam B6 in the floors of the building before and after the fire

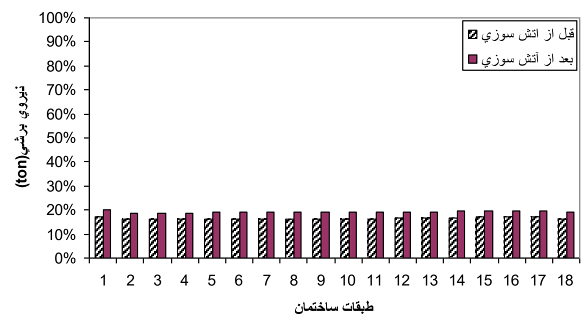


Figure 14 - Shear force (V2) of beam B6 in the floors of the building before and after the fire

3-2 - Performance of building columns after the fire

Considering that columns have an important role in bearing and transferring forces and that changes in their behavior cause irreparable damage to the structure, for this purpose, the performance of building columns in the form of axial, shear and bending moment forces before and after applying fire conditions has been investigated.

The increase in forces has increased the amount of deformation in the columns, and on the other hand, the decrease in stiffness has reduced the column's bearing capacity. This has placed the C7 column, considering the existing conditions and the formation of a plastic joint at the connection point to the foundation, in the elastic deformation zone of the load-deformation curve, depending on the type of joint.

For this purpose, the performance of the columns, especially the C7 column, which is located in the central area of the building and has a higher load than other columns, has been investigated. Here, the columns of the building have been divided into three groups for examination, and the performance of each group has been investigated in terms of changes in axial, shear and bending moment forces.

- The first group of columns around the C7 column: This group includes columns C2 and C1 located next to the building, columns C12, C8 and C4 which are side columns and in connection with the shear wall, and columns C10 and C6 which are the two middle columns and in connection with the shear wall of axes 4 and 3, and columns C11 and C7 which are the two middle columns and not connected to the shear wall.

-The second group of side and corner columns connected to the shear wall: This group includes columns C16, C13, C9, C5, C3, which are connected to the shear wall of axes G and A, B, C, D.

IJARST
International Journal For Advanced Research In Science & Technology
A peer reviewed international journal
www.ijarst.in
ISSN: 2457-0362

-The third group of side columns without connection to the shear wall: This group includes columns C15 and C14, which are located on axes (F-5) and (E-5).

- Effect of fire on the axial force of the first group columns

After analyzing the structure and determining the position of the plastic joints in the beams and column C7, and considering the redistribution of forces in the structural components to control the ultimate capacity of the structural members against the increase in force and the resulting increase in deformation, the values of the axial forces in the first group columns undergo changes. The process of these changes is shown in Figures (16) and (17).

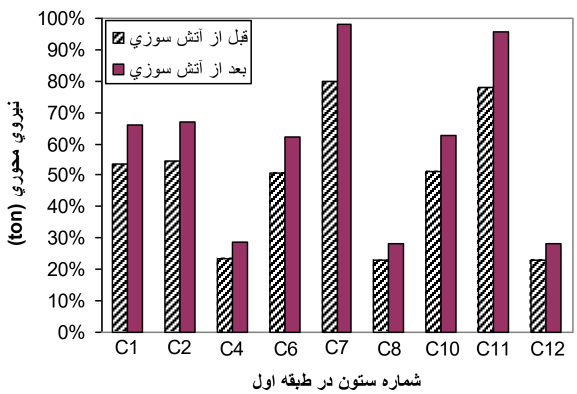


Figure 17 - Axial force of the first group columns on the first floor before and after the fire

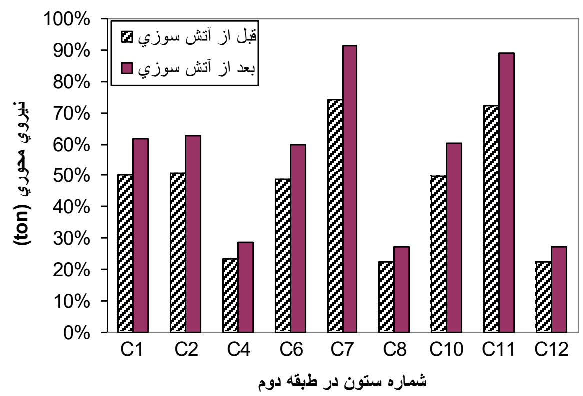


Figure 16 - Axial force of the first group columns on the second floor before and after the fire

As can be seen in Figures (16) and (17), the most changes are attributed to columns C11 and C7, which are the central columns of the building. Columns C2 and C1, which are considered the two side columns, have the second highest increase in values, and the axial force has increased by about 15% compared to before the fire.

Columns C10 and C6, considering that they are considered as central columns, but due to the connection with the shear wall of axes 4 and 3, part of the forces are transferred to the shear walls, and therefore the increase in force in them is less than in the previous four columns. In the case of columns C12, C8 and C4, the situation is the same as columns C8 and C6, because by connecting them to the shear wall of axis F, a large part of the force is borne by the wall and a small percentage of it is allocated to the side columns.

- Effect of fire on the bending moment (M2) of the columns of the first group

By examining figures (18) and (19), it is observed that the largest changes in the moment occur in columns C6, C2 and C1. Considering that columns C2 and C1 are part of the side columns, the column alone bears the largest changes in the moment. However, in columns connected to the shear wall, a large part of the moment values are borne by the shear walls. The increase in the M2 bending moment values on the first floor for columns C2 and C1 has shown its effect in the form of positive bending moments and on the second floor with a higher percentage increase in the form of negative bending moments. The

International Journal For Advanced Research In Science & Technology
A peer reviewed international journal
ISSN: 2457-0362
www.ijarst.in

distribution of M2 bending moments in column C7 does not change much because column C7 undergoes a vertical displacement and the clamping effect that causes the increase in bending moment in the column has decreased. In column C4, almost all the bending moment applied to the column is borne by the shear wall connected to it. In other columns, no significant changes are seen in terms of connection with the shear wall.

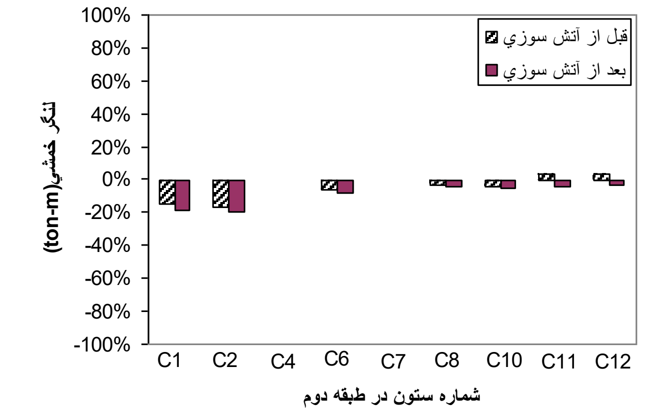


Figure 19-Bending moment (M2) of columns of the first group on the second floor before and after fire

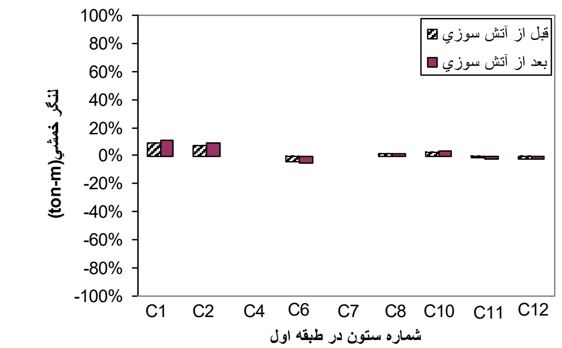


Figure 18-Bending moment (M2) of columns of the first group on the first floor before and after fire

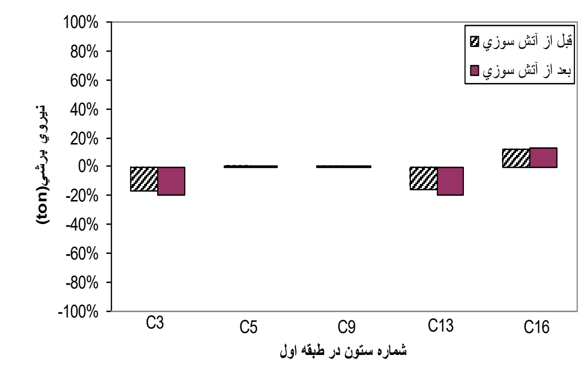
.

3-3- Effect of fire on the performance of columns of the second group on the first and second floors of the building

- Effect of fire on the axial force of columns of the second group

As can be seen in Figures (20) and (21), columns C18, C13, C3 have shown the highest changes in the values of axial force on the first and second floors, and columns C9 and C5 have shown the lowest changes in terms of two-way connection with shear walls of axes 4 and 3. Considering that the walls connected to columns C9, C5 are parallel to the direction of deformation of the structure, they play a significant role in bearing and contributing to the forces arising from asymmetric displacement. However, the walls connected to the three columns C16, C13, C3 have shown a lower contribution in bearing and accepting the applied forces due to their perpendicular direction to the direction of deformation of the structure. The comparison between the values of shear forces in Figures (20) and (21) and the bending moments in Figures (22) and (23) has more clearly explained the distinction between the contributions of aligned and non-aligned shear walls to the deformation of the structure. In the case of shear forces and bending moments, the largest changes are related to columns C16, C13, C3 and the smallest changes are related to columns C9, C5.

International Journal For Advanced Research In Science & Technology
A peer reviewed international journal
www.ijarst.in
ISSN: 2457-0362



21 - Shear force (V2) of columns of the second group on the first floor before and after the fire

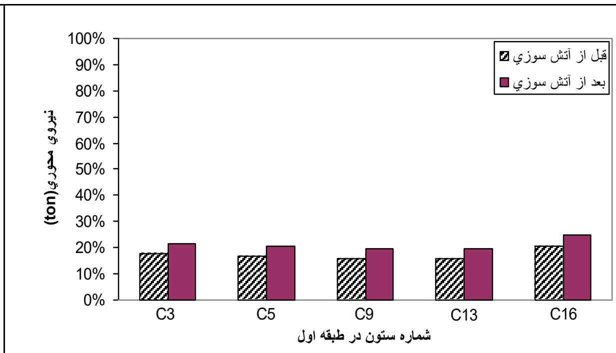


Figure 20 - Axial force of columns of the second group on the first floor before and after the fire Figure

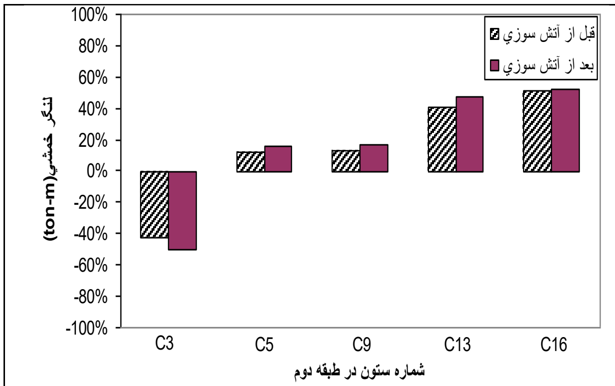


Figure 23 - Flexural moment (M3) of columns of the second group on the second floor before and after the fire

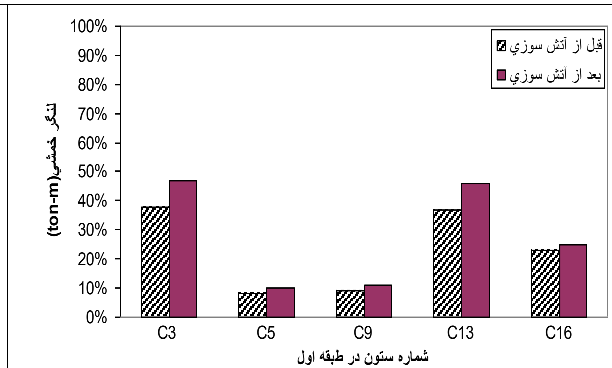


Figure 22 - Flexural moment (M3) of columns of the second group on the first floor before and after the fire

3-4- Effect of fire on the performance of columns of the third group on the first and second floors of the building

3-4-1- Effect of fire on the axial load of columns of the third group

According to figures (8-23) and (8-24), it is observed that the increase in the axial force of the two columns after the effect of fire has increased by about 10%. The columns of this group are part of the side columns of the building.

International Journal For Advanced Research In Science & Technology
A peer reviewed international journal
ISSN: 2457-0362
www.ijarst.in

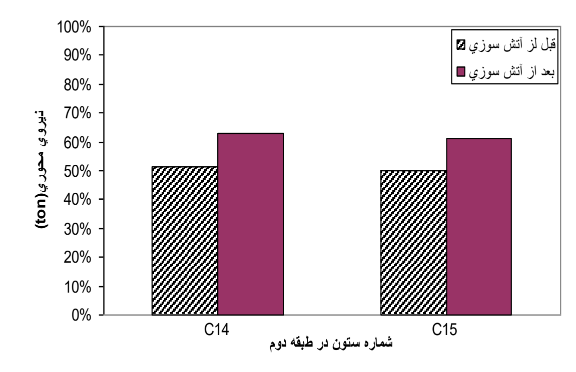


Figure 25- Axial force of columns of the third group on the second floor before and after the fire

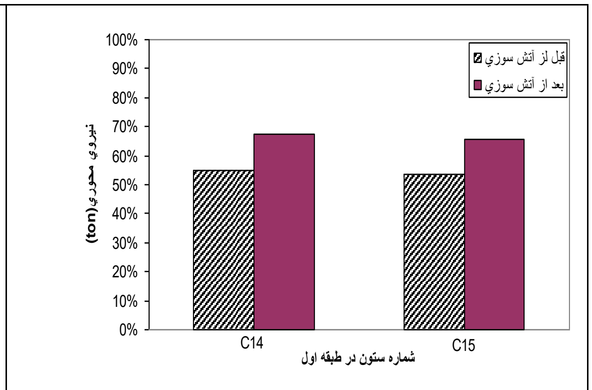


Figure 24- Axial force of columns of the third group on the first floor before and after the fire

- Effect of fire on the shear force (V3) of columns of the third group

According to figures (26) and (27), the increase in the shear force values on the second floor is more than on the first floor. This difference indicates that the effect of further deformation due to the increase in the floor and its effect on the increase in the secondary moment in the columns causes an increase in the moment values in the columns and a corresponding increase in the shear force in the second floor, and the increase in force in both columns is about 10%.

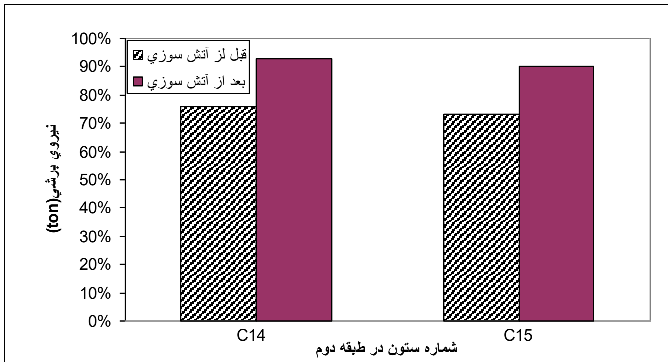


Figure 27 - Shear force (V3) of the third group columns on the second floor before and after the fire

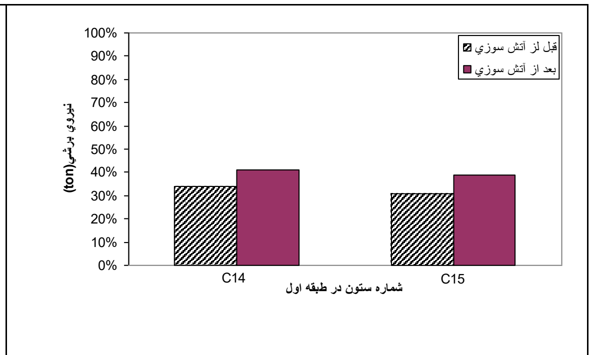


Figure 26 - Shear force (V3) of the third group columns on the first floor before and after the fire

- Effect of fire on the bending moment (M2) of the third group columns

International Journal For Advanced Research In Science & Technology
A peer reviewed international journal
ISSN: 2457-0362
www.ijarst.in
IJARST

As stated in the previous material, the performance of the shear force V3 corresponds to the bending moment M2, and the increase in the shear force V3 in the columns has caused an increase in the bending moment M2 in them. The nature of this process is presented in Figures (28) and (29).

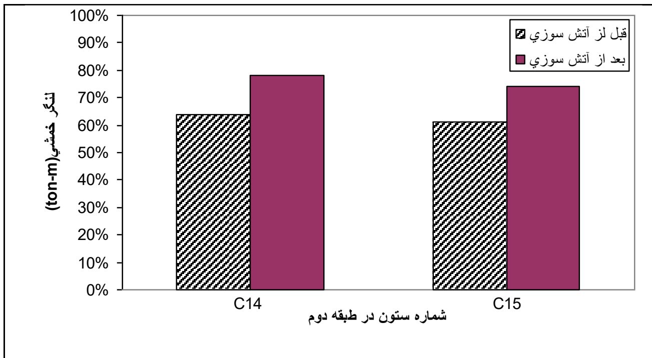


Figure 29- Flexural moment (M2) of columns of the third group on the second floor before and after the fire

Figure 28--Flexural moment (M2) of columns of the third group on the first floor before and after the fire

3-5- Effect of failure of column C7 on displacements

The failure of column C7 caused the structural nodes to shift in different directions, especially to the central areas of the building. Vertical displacements occurred in the nodes of the floors above the failed column and in the four nodes of the columns adjacent to column C7. The largest displacement occurred in the vertical direction at the node of column C7, which, according to Figure (30), in the first floor, this difference in displacement compared to before and after the failure was 7 centimeters, and with the increase in floors, this difference in displacement decreased and reached about 3.5 centimeters on the last floor. In addition to the vertical displacement of the structure, lateral displacements also occurred in two directions. These displacements caused large changes in the internal forces of the members, and their destructive effect is mainly due to the difference in their displacement at the two ends of the member, which does not occur uniformly and symmetrically, and they lead to changes in shear force, bending moment, and severe cracking of the concrete, and ultimately to a reduction in the capacity of the members.

International Journal For Advanced Research In Science & Technology
A peer reviewed international journal
ISSN: 2457-0362
www.ijarst.in
IJARST

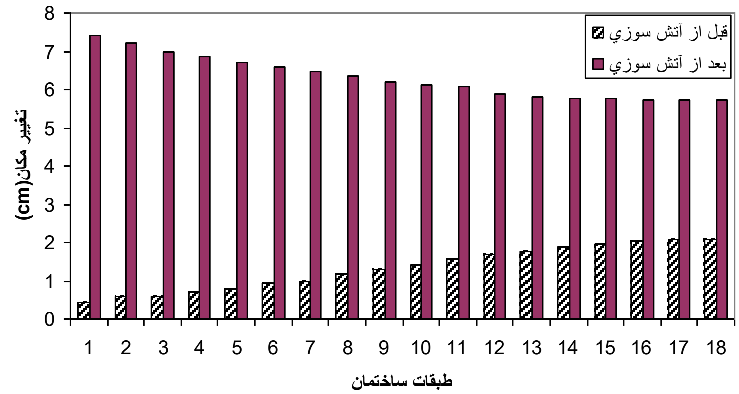


Figure 30 - Vertical displacement of node number 14 due to the failure of column C7 before and after the fire

3-6 - Result of the investigation of the failure scenario of column C7

Columns C7, C10, C11, and C6 are the four middle columns of the structure, which are the strongest columns of the building due to their dimensions and number of reinforcements, and the highest axial loads are applied to these columns. Considering the location of column C7 in the central area of the building, with the failure of the column, the axial load of all vertical load-bearing members, namely columns and shear walls, was affected, and the axial load of this column was mainly redistributed to columns C11, C2, and C6. Meanwhile, the axial load of column C2 increased from 545 to 662, or 117 tons, column C6 increased from 506 to 611, or 105 tons, and column C11 experienced an increase in axial load from 778 to 904, or 126 tons.

By examining the building beams, it was also determined that beams B16 and B6, which are connected to column C7, are the most critical beams in this failure scenario, with the negative bending moment of these beams increasing from 4 to about 67 ton-meters on average, and the maximum shear force in these beams increasing to 23 tons. Therefore, considering that the mentioned beams are connected to columns C11 and C2, and considering the large displacement of node 14 on the first floor that has occurred in the middle of the span of beams B16 and B6, the mentioned beam will shift downward. This causes tensile stresses in the beam and jeopardizes the performance of the columns in question.

By examining the results and performing a nonlinear static analysis of the structure, plastic joints were formed in beams B16 and B6 from the first floor to the second floor. Then these plastic joints are also formed in the direction of column C7 of the second floor. Hence, the formation of plastic joints in the mentioned elements causes that with the rupture of column C7, the beam and column assembly B16, B6 and C7 of the second floor are exposed to the rupture phase, and this causes progressive damage in the central area of the building. The process of damage scenario and formation of plastic joints is shown in Figure (31).

International Journal For Advanced Research In Science & Technology
A peer reviewed international journal
ISSN: 2457-0362
www.ijarst.in

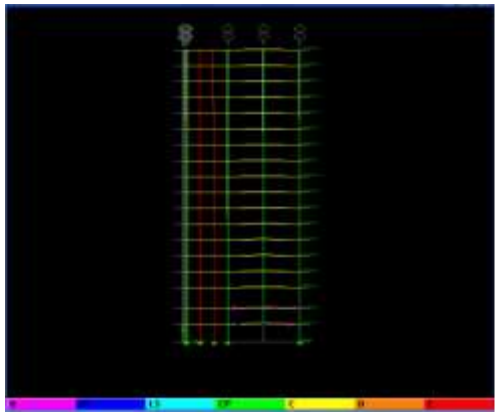


Figure 31- Formation of plastic joints in the second floor C7 concrete slab resulting from the failure of the first floor C7 column

4- Conclusion

According to the presented material, it seems necessary to mention a few points about how concrete buildings perform in the face of fire and the solutions to deal with the failure of members and delay the time of destruction of structural components.

- With increasing temperature, the strength of concrete and steel materials decreases, and the speed of this decrease varies depending on the severity and time of the fire. For this reason, in order to increase the durability (the length of time that a member can maintain its load-bearing capacity during a fire), concrete members need to first increase the durability of the constituent materials as much as possible, and then increase the durability by observing the implementation criteria. Using resistant aggregates such as limestone, using foaming agents in concrete construction, and observing the water-to-cement ratio to limit the internal moisture of the concrete to prevent sudden cracking and compaction of the concrete, and using rebars with standard mechanical specifications (Fy) that have a higher critical temperature can increase the durability of the concrete member in terms of materials. Observing the elimination of sudden changes in the cross-section, observing the anchoring length of the reinforcement in the bending elements that suffer from the one-way membrane phenomenon due to the destruction of the column, distributing longitudinal bars in the tension and compression areas in the bending elements and implementing them without interruption in the cross-section due to the change in the direction of the bending moments and the uneven distribution of heat along the length of the element, repeating and distributing transverse bars in the middle part of the bending and compression elements in addition to the support areas, using double intermediate pins and yokes in the columns for better confinement of the concrete and longitudinal reinforcement, observing the column slenderness limit in the designs of compression members according to valid regulations.

- In the event of a fire, the performance of building members will vary depending on their location in the building and the amount of load applied to them. In examining the fire scenario, it was determined that beams connected to the shear wall and also peripheral beams that have a smaller load-bearing share than the load-bearing intermediate beams are less damaged due to the effect of fire and the reduction in the mechanical properties of the materials, but intermediate beams that have a larger share in the load-bearing and bending moments and shear forces have a more critical effect on their performance experience greater

International Journal For Advanced Research In Science & Technology
A peer reviewed international journal
www.ijarst.in
ISSN: 2457-0362

deformation and behavior changes due to the effect of fire. The formation of plastic joints in intermediate beams according to fire conditions is a proof of this claim.

- The performance of columns under fire conditions is such that the behavior of columns connected to the shear wall is less compromised due to the participation and compatible interaction of the shear wall and the column in bearing the load. The performance of the side columns that are not connected to the shear wall is such that the changes are tolerated by the column alone and the increase in forces and deformations is tolerated by the column. These changes are such that the column anchors increase, which increases their effect on creating local instability of the structure by about 50%. In the case of the middle columns, which have a very large share in the load-bearing capacity of the building and are connected to at least four beams in their location, any change in their performance has a significant impact on the overall behavior of the structure and their effect on creating structural instability is at the highest level.

This indicates that in high-rise buildings, the middle section columns are in a more critical state than other columns, and any change in the state of the structure has a great effect on the performance of these columns, which may cause their destruction and rupture.

If these columns fail, the possibility of damage to the upper floors that are affected by fire and to areas where column failure can jeopardize the performance of intermediate members and components is obvious.

According to the above, the most important factor that can cause instability and reduce safety in high-rise buildings under the effect of fire is the failure and destruction of columns in the central area of the building. Because by removing these columns, the connected beams, whose existence plays an important role in providing their support effect, are destroyed and the beams undergo large deformations. According to the theory of plates and shells, if a beam with a fixed support undergoes large deformations, a one-way membrane function is created in the beam, which indicates an increase in tensile forces and the elimination of compressive forces in the beams. This tensile force has a great effect on increasing the bending moments of the columns connected to them and severely affects the supporting performance of the column, in such a way that its effect can cause the beam-column connection area to rupture or force the column to buckle locally in that area.

Distributing longitudinal reinforcement in the tension and compression areas of the beam and not interrupting the reinforcement at the support location, distributing the transverse reinforcement of the column at the connection location, and using steel couplings instead of overlapping the reinforcement at the connection location in order to comply with the steel percentage criteria can be appropriate solutions to prevent the destruction and rupture of members and, as a result, local or overall instability of the structure.

Refrences

1. Buchanan,A,H.Structural Design for Fire Safety,John Wiley& Sons. (2001).
2. V.K.R Kodur :Fire performance of High-Strength Concrete Structural Members,"Construction Technology Updates"National Research Council of Canada,December 1999.

3. ZHAOHUI HUANG,IAN W. BURGESS and ROGER J . PLANK: BEHAVIOUR OF REINFORCED CONCRETE STRUCTURES IN FIRE,Civil & Structural Engineering,University of Sheffiel,S1 3JD,UK,2006.
4. J . H .Hsu and C . S. Lin :Residual Bearing Capabilities of Fire-Exposed Reinforced concrete Beams,Department of Civil Engineering , Ching Yun University,Taiwan,August29,2006.
5. BSI, Structural Use of Concrete, BS110,British Standards Institution,UK(1985).
6. ISO,(1975)," FireResistance Tests-Elements of Building Construction",ISO 831975, International Organization for Standardization.
7. Eurocode3.Design of concrete structures: general rules,structural fire design, European Committee Standardisation,(1995).
8. Federal Emergency Management Agency. World Trade Center building performance study. Report 403. FEMA, Federal Insurance and Mitigation Administration; 2002.
9. UNIFIED FACILITIES CRITERIA (UFC)Department of defense: DESIGN OF BUILDINGS TO RESIST PROGRESSIVE COLLAPSE(UFC 4-023-03 25) ,USA January 2005