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Study of the resistance and stability of existing educational structures (public school buildings) under the effect of fire and explosion

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Abstract

Explosion and fire are phenomena that every school may experience during its useful life. With the development of modern urbanization, the potential for fire hazards and their probability of occurrence in schools have also expanded. What is important is how to prevent and also deal with fire hazards in these buildings, which should be given great attention. In this article, a 3-story building with educational use (a boys' school in Tehran) with a bending frame system is modeled using software and their behavior is evaluated under the effect of nonlinear static analysis. Then, all models were modeled in the Abaqus finite element software and subjected to explosive loads and the resulting fire at certain distances from the structures according to the criteria of topic 21 and re-evaluated. The results were compared with models related to seismic design. Then, the progressive failure in the building was investigated using the method of instantaneous removal of certain columns in the Abaqus software. The results and the method of redistributing forces in the related elements were presented so that the comparison of the obtained results can be used to achieve a better understanding of the behavior of structures in this type of analysis.

Keywords: Resistance, stability of existing educational structures, fire, explosion

1- Introduction

Today, urban buildings are built with metal and concrete skeletons, and each of the concrete and steel structural systems has its own unique advantages and disadvantages. [1] But what is less considered when choosing the structural system of buildings is their fire resistance. [2, 6]. One of the main disadvantages of metal buildings is that they are very weak against fire and with the slightest fire that occurs next to the columns, the steel immediately melts and loses a large part of its resistance[3]. What happened in the collapse of the Plasco building in Tehran was precisely the failure to prevent the fire from occurring in the building and then, due to the spread of heat to the main members of the building, which were metal, they melted and weakened their resistance, and in an instant the building collapsed with an explosion. Unprotected structural steel has a bending

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temperature of 470-500 degrees Celsius and loses about half of its resistance at a temperature of 550-500 degrees Celsius, therefore it is very vulnerable to fire. According to ISO and BS 476 tables, iron reaches a temperature of 470 degrees in less than 10 minutes after a fire, which is the bending point of iron. Research conducted on fire caused by explosions has focused more on the risk of its occurrence and modeling of how it occurs, spreads, and extinguishes from a firefighting perspective, and less attention has been paid to studying the effect of this fire on buildings from a structural perspective. [8]. From a structural perspective, the behavior of a structure under earthquake load is more important because when a structure enters its plastic performance limit under the influence of large earthquakes, its load-bearing capacity decreases. If such a structure is exposed to fire, the probability of loss of structural stability increases and the durability of the structure decreases. This point is of particular importance in metal skeleton structures because the mechanical properties of steel deteriorate rapidly with increasing temperature. In addition, structural steel members that have entered their functional plastic range due to an earthquake also lose their load-bearing capacity more rapidly. Among the studies that have been conducted on the behavior of steel structures under the effect of fire after an earthquake, we can mention the studies of Delacourt et al. [3]. In this study, they investigated the behavior and resistance of flexural steel frames against fire after an earthquake. Another analytical study was conducted by Yasin et al. on the performance of steel frames under fire after an explosion [4]. The analysis of two-dimensional steel frames in this study showed that the behavior of these frames under fire after an earthquake is strongly affected by the lateral deformation that occurs in these frames due to the earthquake. In another analytical study conducted by Eldridge et al. in 2009[5], an earthquake-resistant building constructed of composite steel-concrete frames was investigated for its structural performance against post-earthquake fire. Although steel elements insulated with fire-resistant coatings perform well against ordinary fires (not post-earthquake fires), and this delays the collapse of structures composed of these elements, this coating may lose its protective role for steel elements that have been exposed to earthquakes. Under post-earthquake fire conditions, steel members enter their plastic performance range to withstand earthquake loads, and plastic hinges form in beams and columns. Therefore, in this research, according to the introduction mentioned above, following the completion of studies conducted with different specifications and conditions, the effect of fire on steel structures will be investigated, taking into account the effects of explosion on it.

2- Models under study

The models under study in this study are 4-span steel frames in plan and height, in a 3-story type. The spans of the frames in all models are 3 to 5 meters and the height of all floors is 3.3 meters. Considering the advanced capabilities of ETABS software in the design of building structures, all models have been modeled, loaded and designed in this software in three dimensions. After determining the most optimal sections, the frames under study were subjected to nonlinear static analysis using the ETABS software features and then the models have been evaluated in Abaqus software to examine progressive damage. The length of the building plan is 16 meters and its width is 14 meters and the height of the building floors is 3 meters. The building in question is located in an area with very high seismicity, the soil is type 3, and the rest of the parameters have also been modeled with logical information. Figure 1 shows the general plan of the building along with its joist design. Box sections and sections available in the market were used optimally for the design. The models were analyzed and designed according to the third edition of the 2800 Code.

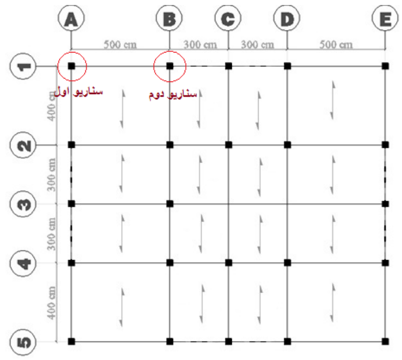


Figure 1 - Floor Plan

After the studied models are determined, loading is required to design the structural members. The gravity loading of the structure is in accordance with the Iranian Loading Code, Section 6 (Edition 92). In Figures 2 and 3, the stress ratio of the members is presented after the models are designed. As can be seen, in order to achieve more accurate results, the design was done in such a way that the best stress ratio occurs for the members.

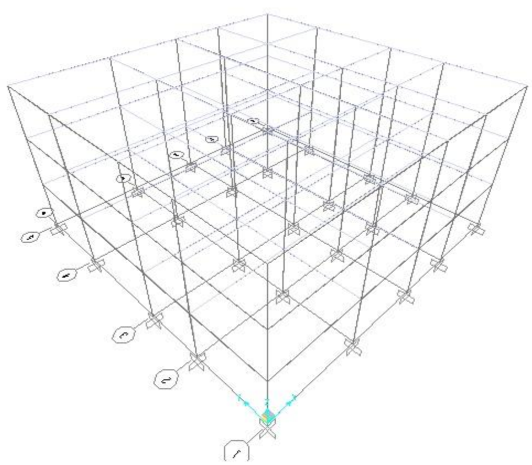


Figure 3- 3D view of a three-story steel structure model with a bending frame system

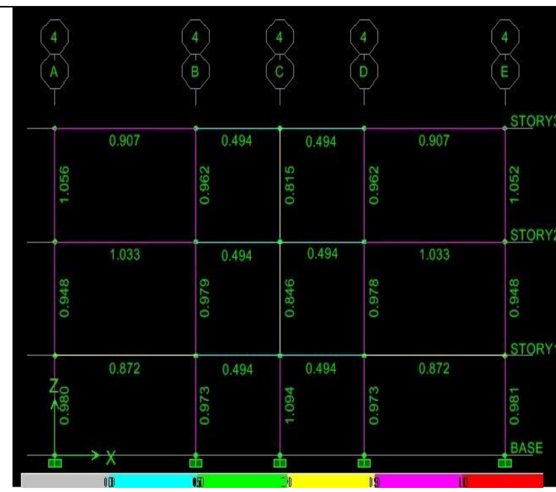


Figure 2- Stress ratio of members after design for a 3-story structure

3- Explosive loading

The equivalent method is used to distribute the load on the structure. In this method, the amount of pressure applied to each point of the structure is obtained according to the distance from the center of the explosion and the amount of explosive material from the following equation[9]:

$$P_w = \frac{14.072}{Z} + \frac{5.54}{Z^2} + \frac{0.357}{Z^3} + \frac{0.00625}{Z^4} \qquad (0.05 < Z < 0.3)$$

$$P_w = \frac{6.194}{Z} + \frac{0.326}{Z^2} + \frac{2.132}{Z^3} \qquad (0.3 < Z < 1.0)$$

$$P_w = \frac{6.662}{Z} + \frac{4.05}{Z^2} + \frac{3.288}{Z^3} \qquad (1.0 < Z < 10.0)$$

(1)

This is an empirical formula for calculating the increase in static pressure caused by the explosion on the structure, which is expressed in terms of the scaled distance parameter Z. This parameter is calculated based on the weight of the explosive material and the distance (Equation 1).

The method used to equate the explosion load is that the increase in pressure applied to each of the beams and columns constituting the structure is calculated using the empirical equations provided, and then it is converted into a force by multiplying this pressure by the element area, and the contribution of each of the beams and columns constituting this pressure is calculated and applied to that node.

To calculate the compressive load of a column at a distance of 6 meters from the center of the explosion for 500 kg of TNT, first calculate the scaled distance and calculate the base pressure, and then calculate the pressure on the column surface with the relevant relations:

$$Z = \frac{6}{500^{\frac{1}{3}}} = 0.7559$$

$$0.3 < 0.7559 < 1$$

$$p_w = \frac{6.194}{0.7559} + \frac{0.326}{0.7559^2} + \frac{2.132}{0.7559^3} = 13.7$$

$$p_r = 2p_w\left[\frac{7p_0 + 4p_w}{7p_0 + p_w}\right] = 2 \times 13.7\left[\frac{7 + 4 \times 13.7}{7 + 13.7}\right] = 81.8\text{kg/cm}^2$$

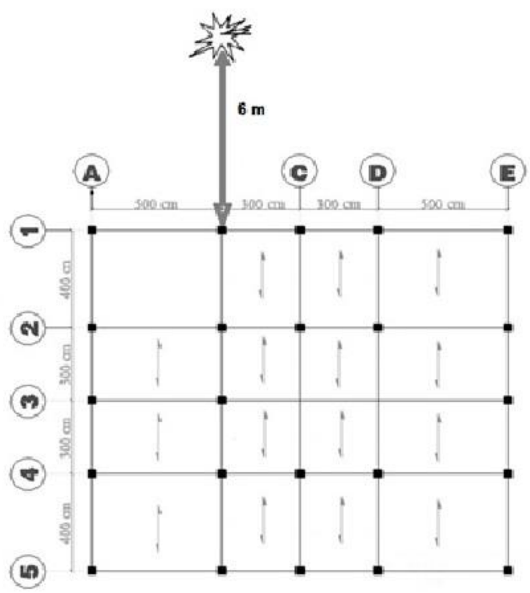


Figure 4- Location of the explosive loading site

For the first part of the analysis of the modeled buildings, the location of the explosion was considered at distances of 6, 4.5 and 3 meters and was assumed to be near the ground surface. Figure 2 shows the location of the explosion at a distance of 6 meters. The weight of the explosive material is considered to be 500 kg of TNT. And in the second part of the analysis of the effect of the explosion load, the amount of 750 and 1000 kg of TNT was considered at a distance of 6 meters from the structure. Having the distance and consequently the scaled distance, the increase in static pressure due to the explosion is obtained based on the relations presented. By multiplying this pressure by the load-bearing surface of each beam and column, the force exerted on it is calculated.

The model geometry consists of a frame that is made up of concrete beams and columns with rebars embedded in them. The concrete frame is considered as 4 spans of 3 to 5 meters and 3.3 meters high with beam-column connections as clamps. The column base connection is also considered as clamps.

4- Model construction

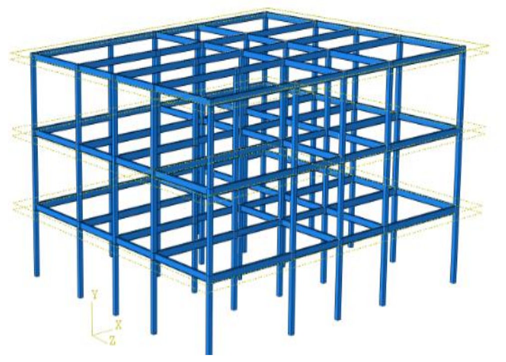
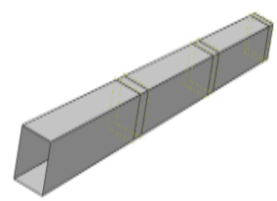
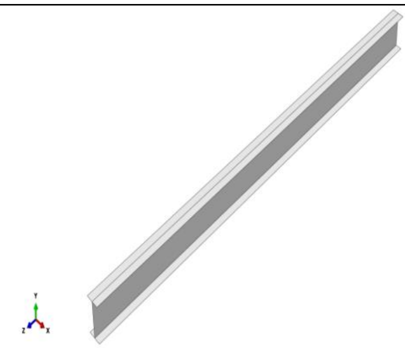
		

Figure 6- 3-story structure model	Figure 5- Geometric shape of the box column modeled in Abaqus	Figure 4- Geometric shape of the beam modeled in Abaqus

5- Specifications

- **General principles of the plastic failure model**

The failure criterion is expressed in the plastic limit of the material under combined stresses. This criterion is divided into two main categories based on the material's response to hydrostatic pressure. In most materials, ductile behavior is known as hydrostatic pressure dependent, and non-metallic materials such as soil, rocks, and concrete belong to this category and are pressure dependent. The following explains some of the parameters required in the software to define the model properties.

- **Dilation angle**

The dilation angle defines the plastic strain, due to shear, in the post-elastic phase, and when ψ = 0, the material has no change in strain volume. In fact, the expansion angle expresses the relationship between volume and shear strain according to the following equation

$$\psi = -(\delta\varepsilon_v)/(\delta_\gamma)$$

According to the findings of Vermeer de Borst, materials subject to hydrostatic pressure such as soil, rock and concrete have an expansion angle of 20 degrees, which is greater than their internal friction angle (normal concrete 12 degrees) when they are under multiaxial stresses. If the confinement phenomenon occurs, the value of the expansion angle can change. According to experiments, it has been determined that for reinforced concrete, the expansion angle will be between 20 degrees and 40 degrees.

The smaller the value of this angle, the brittle behavior of the material and the larger the angle, the material itself shows a behavior similar to that of ductile materials. It has been shown that the angle ψ = 31 defines the best behavior in compression and tension in concrete.

- **Parameter (f)**

This parameter expresses the characteristics of the failure function and is defined by f=fbo/fc. In laboratory studies conducted for concrete fc=30.3 Mpa and fc=39 Mpa, the value of fbo is considered to be 16.4%. However, in some scientific articles, the ratio fbo/fc is considered to be 1.16. **Parameter (ϒ)**

This parameter defines the shape of the deflection of the stress plane and also the angle of confinement of the concrete. When the deflection plane is close to a triangular shape, the degree of confinement is considered to be less, and when it is circular, the degree of confinement is considered to be greater. This parameter is calculated from the above relations, where J2 is the second invariant of the deflection stress tensor for meridional tension and compression. In this formula, the parameter ϒ is defined between 2.3 and 1, but some researchers introduce its maximum value as 0.8.

$$\gamma = \frac{3(1-\rho)}{2\rho+3}$$

$$\rho = \frac{(\sqrt{J_2})_{TM}}{(\sqrt{J_2})_{CM}}$$

- **Viscoplastic Modifier Parameter (η)**

In advanced material models, it is difficult to achieve convergence for tensile and stiffness decay. The usual way to overcome this problem is to use equations in the viscoplastic modifier. This value calculates the stiffness tangent of the material using the softening behavior.

ABAQUS software uses the Duvant-Lions modifier, according to which the relative strain tensors are used for the viscosity-hardening decay using the following relationship. Small values of the viscoplastic modifier parameter are compared with time intervals to obtain the results of the softening regime of the model to improve its convergence. In this relationship, it is recommended that μ/t→∞.

$$d^{\bullet}{}_{v} = \frac{1}{\mu}\left(d - d_v\right) \qquad \overset{\bullet}{\varepsilon}{}_{v}^{Pl} \; (\varepsilon_{pl} - \varepsilon_v)$$

The characteristics of the three grades of concrete and steel, as well as the FRP fibers used, are given in Tables (1) to (4), respectively:

Table 1- Properties of concrete used in the model

f_c (Mpa)	υ	(Mpa) E	Concrete grade
30	0.18	25742	Type 1 concrete

Table 2- Specifications of the plastic range of concrete

Dilation Angle	Eccentricity	Fbo/fco	K	Viscosity Parameter
30.5	0.1	1.16	0.666	0.001

Table 3- Specifications of RCC concrete in compression and tension

stress	plastic strain	tensile stress	cracking strain
5.7	1.17E-05	3.286335	0
10.8	4.66E-05	0.03	0.001277
15.3	0.000105		
19.2	0.000186		
22.5	0.000291		
25.2	0.00042		
27.3	0.000571		
28.8	0.000746		
29.7	0.000944		
30	0.001165		
25.5	0.002809		

Table 4- Steel specifications table

جنس	$\Upsilon (kg/m^3)$	*E(Gpa)*	*v*	$F_y(Mpa)$	$F_u(Mpa)$
Steel	7850	203	0.3	448	530

- Element Type

Solid elements are used to model beams and columns. Solid elements in Abaqus can be used for complex linear or nonlinear analyses with contact, plasticity, or large deformations in problems such as stress analysis, heat transfer, acoustics, thermodynamics, fluid flow in a medium, piezoelectric, electromagnetic, and thermoelectric. This type of element is used to observe a three-dimensional stress. These elements are available in three-dimensional and two-dimensional spaces. In three-dimensional space, the stress analysis problem is complete and the shape of the elements can be 4-, 5-, and 6-sided. In two-dimensional space, the problem is in the form of plane stress or plane strain, and the shape of the element can be 3- and 4-sided. It is worth noting that the degrees of freedom of the nodes of solid elements are only of the translational degree of freedom type. If the shape function is selected from the first degree, the stress is obtained uniformly throughout the element, and if the shape function is selected from the second degree, the stress will change linearly in the element.

A truss-type wire element is used to model steel wires. Truss elements are one of the common structural elements that can be used in Abaqus. A truss element is a two-force member in which forces are applied only at its nodes, and these nodes only have translational degrees of freedom. These elements only have axial resistance. In these elements, only the nodes transmit the force, and therefore the truss elements do not show bending resistance. These elements can be used in two-dimensional and three-dimensional modes. Two-dimensional truss elements are used in axisymmetric models to represent bolts and connections and also in modeling trusses on the plane. Three-dimensional truss elements are recommended for modeling spatial trusses or cables, such as prestressed cables in reinforced concrete or oil and gas pipelines, where fluid-structure interaction is not considered.

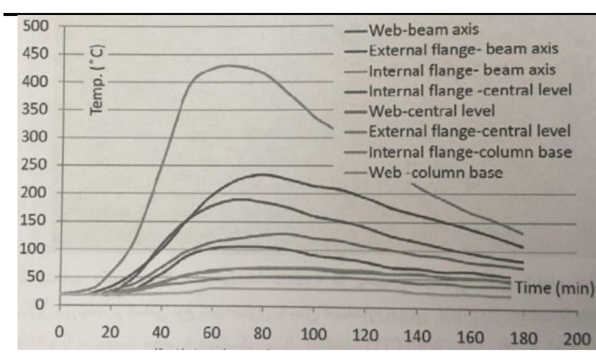
An eight-node cubic element (C3D8R) was used in the meshing of the beam and column mass and walls, and a four-node element (S4R) was used for the steel beam and columns. In sensitive areas, the mesh dimensions are smaller. The meshed image of the model parts is shown in the figure.

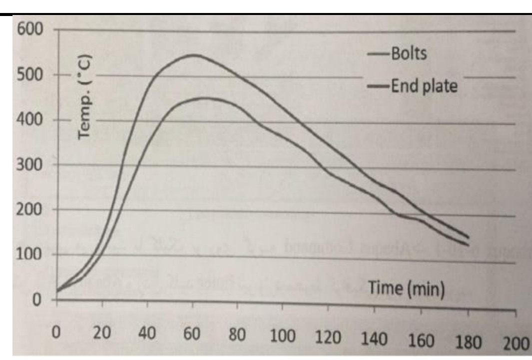
Figure 7 - Structure Meshing 3

- Type of Analysis and How to Apply Fire to Models

In this study, the standard solver was used to analyze the models. The standard solver uses Newton's method to solve nonlinear equilibrium equations. Many problems involve time-dependent responses; therefore, the solution usually consists of a series of increments with trial and error (iterations) to reach equilibrium at each increment.

One of the problems that affects structures such as educational buildings is fire. This is more important and sensitive in the case of steel structures, because in steel members, due to their high thermal conductivity and thin thickness, fire causes a decrease in strength and stiffness. In this research, based on the tests carried out on the steel frame and the stress-strain diagrams obtained from it, it has been modeled and studied in the Abaqus software. The thermal loading applied to the structural components is in the following forms:





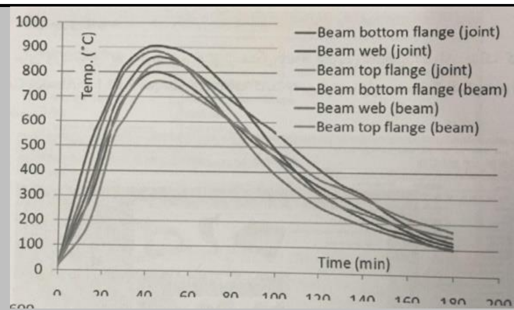


Figure 8- Temperature curves applied to different parts of the structure

6- Modeling Results

In this section, the results of the explosion analysis on a three-story building with a bending frame system with the plan specified in Figure 2 are examined. To analyze the results, first the explosive distance in terms of the TNT of the building is examined, then the subject is examined based on the amount of explosives.

- Response of a three-story structure to explosive loading based on the explosive distance

In Figure 1, the base shear of a 3-story building for an explosive loading of 500 kg TNT for distances of 6, 4.5 and 3 meters is shown. As can be seen, the base shear for the first two cases does not show a significant difference from each other. The maximum base shear in the three explosive loading cases is compared with the seismic design base shear of the building. As can be seen, the base shear for static seismic analysis is about 14.4% of the maximum explosive shear with 500 kg TNT at a distance of 6 meters. This ratio for the other two cases, i.e. distances of 4.5 and 3 meters with 500 kg of TNT, is about 11.3% and 9.9%, respectively. In all three cases, a major difference is observed between the base shear caused by the explosion and that on which the building was designed.

Another parameter that was compared in this study is the displacement of the roof of the building. In Figure 2, the displacements on the roof (third floor) in three explosive loading cases are compared with each other. As can be seen, with the decrease in the distance of the explosive, the displacement of the roof increases, which is not proportional to the decrease in the distance of the explosive. In other words, with the decrease in the distance of the explosive from 6 to 3 meters, the maximum displacement has increased by about 160%.

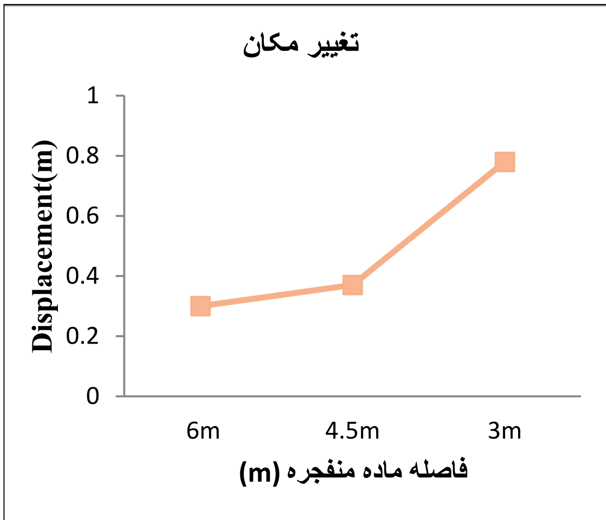


Figure 2- Comparison of the lateral displacement of the roof of a three-story structure in three different explosive loading conditions

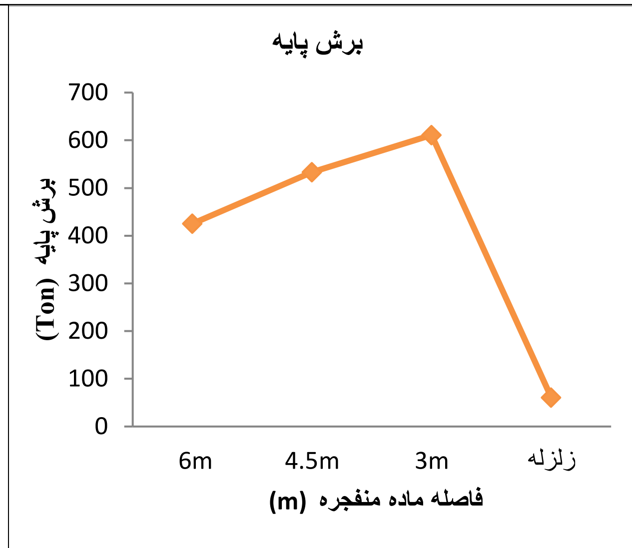
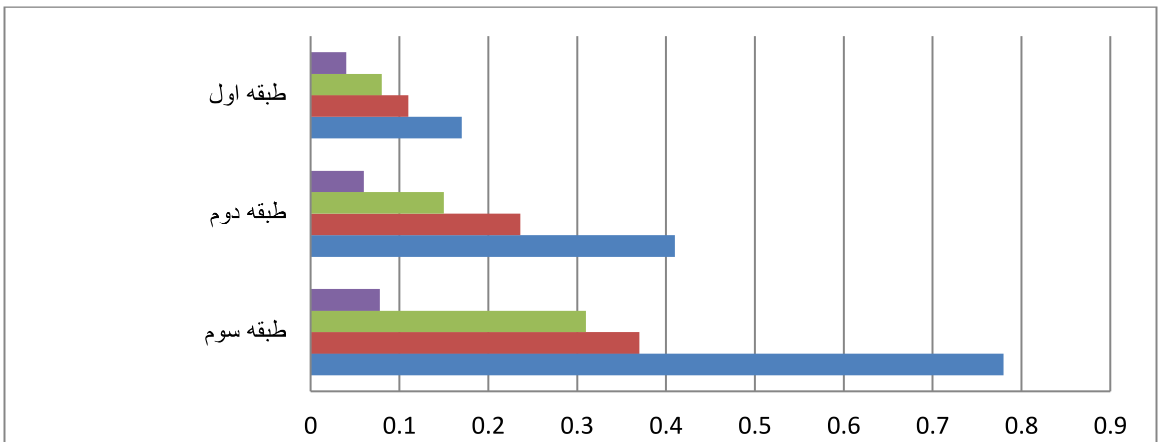


Figure 1- Comparison of the base shear of a three-story structure in three different explosive loading conditions

For further comparison, the maximum lateral displacement at the level of all three floors due to an explosion with explosives with different distances of explosives was compared with the lateral displacement due to the design seismic load and is displayed in Figure 3 as a graph. As can be seen, the lateral displacement due to earthquakes is much less than the lateral displacement due to explosive lateral loading.



	طبقه سوم	طبقه دوم	طبقه اول
بار زلزله	0.078	0.06	0.04
انفجار در فاصله 6 متری	0.31	0.15	0.08
انفجار در فاصله 4.5 متری	0.37	0.236	0.11
انفجار در فاصله 3 متری	0.78	0.41	0.17

Figure 3- Comparison of the lateral displacement of the floors of a three-story structure in three different explosive loading conditions

- Response of a three-story structure to explosive loading based on the amount of explosive

Considering that the response of the structural elements to the removal of columns on the ground floor is much greater and more critical than the upper floors, and also the sensitivity of the first floor elements to the stability of the structure is greater, the results of this floor have been evaluated. In Figure 4, the base shear of a 3-story building for different explosive loading conditions including 500, 750 and 1000 kg of TNT at a distance of 6 meters from the structure is shown. The maximum base shear in the three explosive loading conditions is compared with the seismic design base shear of the building. As can be seen, the base shear for seismic analysis is about 13.2% of the maximum explosive analysis with 500 kg of TNT. This ratio is about 10.8% and 8.7% for the other two explosive loading conditions, respectively. In all three cases, a major difference is observed between the base shear caused by the explosion and what the building was designed for.

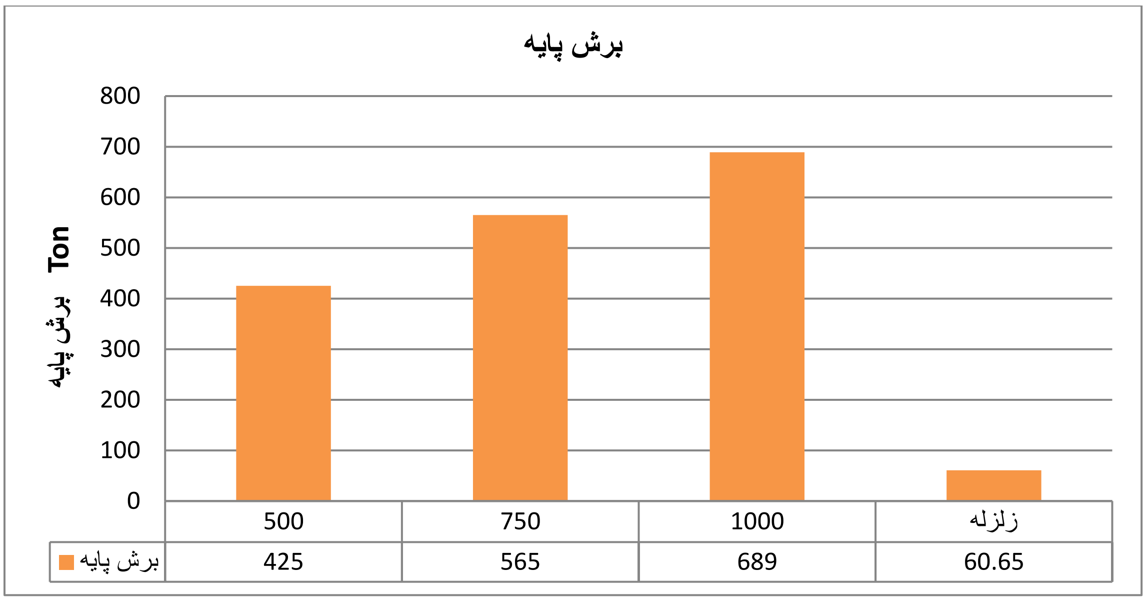


Figure 4 - Comparison of the base shear of a three-story structure in three different explosive loading conditions

In Figure 5, the displacements in the three explosive loading conditions on the roof (third floor) are compared with each other. As can be seen, with an increase in the amount of explosive, the roof displacement increases, which is not proportional to the increase in the amount of explosive. In other words, by doubling the amount of explosive from 500 to 1000 kg, the maximum displacement has increased by about 186%.

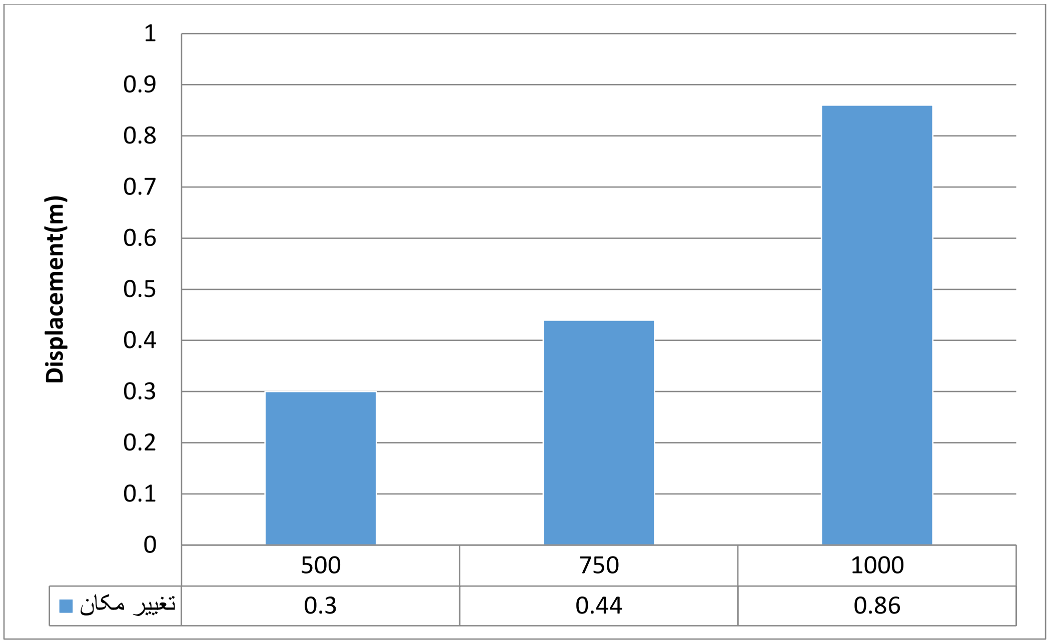


Figure 5 - Comparison of lateral displacement of a three-story structure in three explosive loading conditions according to the amount of explosive

For further comparison, the maximum lateral displacement at the level of each of the three floors due to explosions with different amounts of explosives was compared with the lateral displacement due to the design seismic load and is displayed in Figure 6 as a graph. As can be seen, the lateral displacement due to earthquakes is much less than the lateral displacement due to lateral loading.

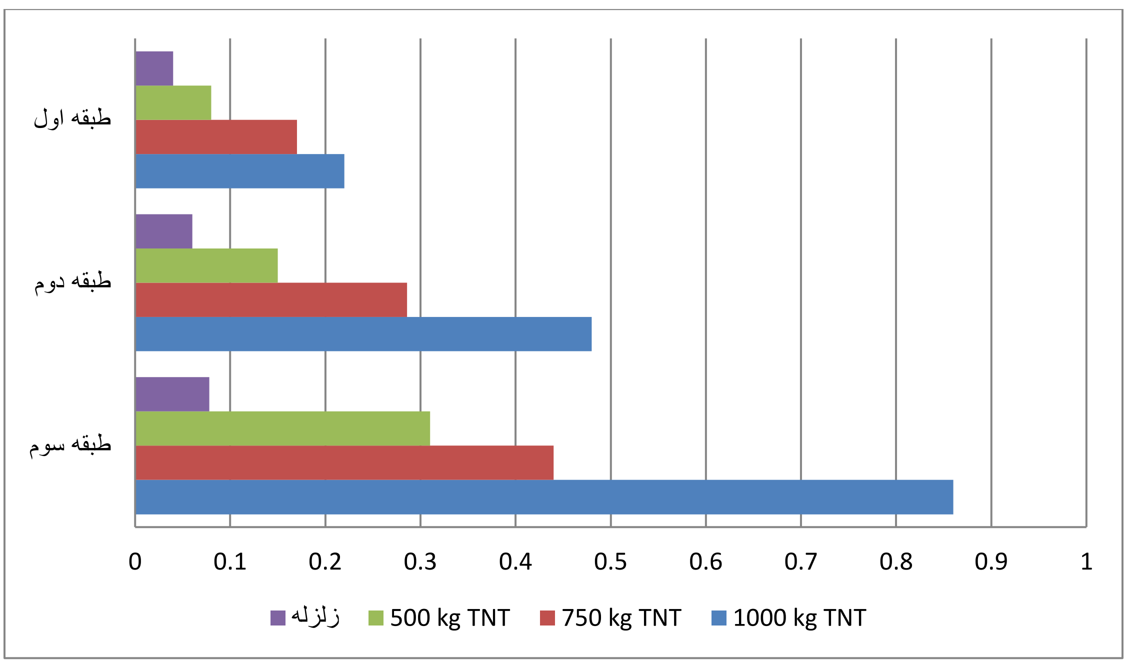


Figure 6 - Comparison of lateral displacement of three-story structures in three different explosive loading conditions

- **DCR Criterion Study**

One of the criteria for examining progressive failure is the DCR criterion, which is the demand-to-capacity ratio, also known as the demand coefficient[3].

$$DCR = \frac{Q_{ud}}{Q_{uc}} \qquad (2)$$

QUD is the force resulting from the analysis in the member or connection and Q is the expected capacity in the member or connection. If the DCR ratio is greater than 2, the member is severely damaged and is likely to collapse, and by removing them from the model, the failure range is compared with the allowable values[3]. In this study, the DCR criterion was calculated for bending and shear in members adjacent to the blast load location, and the results were presented in shape diagrams.

In this part of the study, the DCR criterion, which is one of the criteria for examining progressive failure, which is the demand-to-capacity ratio, also known as the demand coefficient, is examined. The higher the value of this criterion, the more severe the damage is and the higher the probability of collapse. In Figures 7 and 8, the results of the DCR criterion are displayed in the form of a graph. As can be seen, the results for the axial force of columns B1 and A2 are higher than those of other members and the probability of collapse in them is higher. Also, for the bending moment of the beams, it is observed that the probability of collapse of beams AB-1 and A1-2 is very high. For example, the calculation of the DCR ratio for column B1 and beam AB-1 is as follows.

ton109.082= Axial force created in column B1 as a result of dynamic analysis in Abaqus software

Ton 160.4= Capacity of column B1 section for axial force (calculated using ETABS section designer software)

$$DCR_{B1} = \frac{Q_{ud}}{Q_{uc}} = \frac{109.082}{160.4} = 0.68$$

Ton 8.563= Axial force created in beam AB-1 as a result of dynamic analysis (peak point from Figure 4-11)

Ton 16.2= Capacity of column AB-1 section for axial force

$$DCR_{AB-1} = \frac{Q_{ud}}{Q_{uc}} = \frac{8.563}{16.2} = 0.54$$

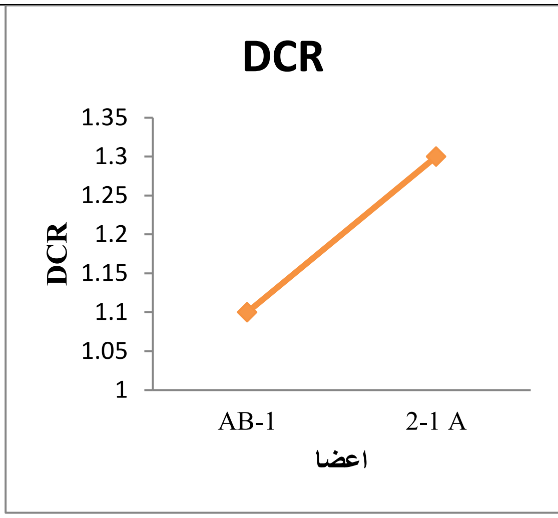


Figure 8- DCR criterion for buckling in case Column A1 Removal for Three-Story Structure

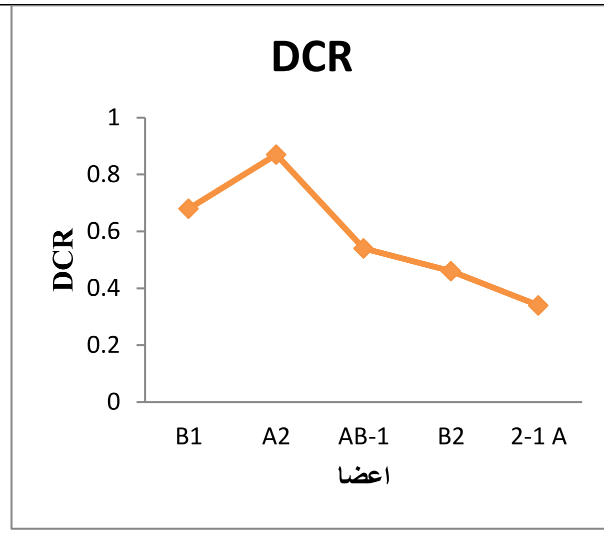


Figure 7- DCR criterion for axial force in case of removing column A1 for three-story structure

- **Structure Response to Column Removal**

In this part of the study, the DCR criterion for the changes in the forces under study for the second case of column removal has been examined. In Figures 9 and 10, the results of the DCR criterion are displayed in the form of a graph. As can be seen, the results for the axial force of columns A1, A2, and C1 are higher than those of other members, and the probability of collapse in them will be higher. Also, for the bending moment of the beams, it is observed that the probability of collapse of the beams under study in the vicinity of the removed member is lower than in the first case of column removal, i.e., removal of column A1.

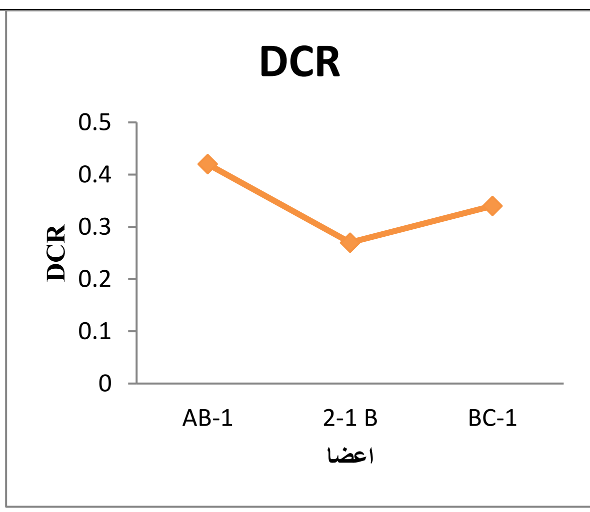


Figure 10- DCR criterion for torsional buckling in the case of removing column A1 for a three-story

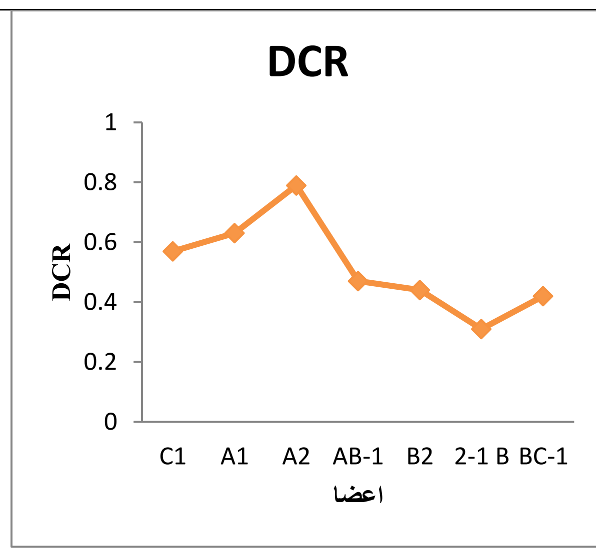


Figure 9- DCR criterion for axial force in the case of removing column A1 for a three-story structure

structure	

7- Conclusion

At the end of the analyses used in this paper, results were obtained to understand the use of progressive failure analysis in the evaluation of steel flexural frames exposed to explosive loads and fire applications, and some of these results are mentioned below as a case study.

Reducing the explosive detonation distance increases the displacement of the floors, so that for a 3-story structure, for 4.5 and 3 meters of explosive distance, the amount of displacement of the roof of the structure for 500 kg of TNT at a distance of 6 meters has increased by 23% and 160%, respectively.

By three-dimensional analysis of 3-story steel buildings under the effect of explosive load and comparing it with the results of static analysis of earthquake load, it was found that the base shear caused by the explosion load presented in this study is much greater than the base shear in the seismic design, so that for 500 kg of TNT at a distance of 6 meters from the 3-story structure, the roof base shear is about 7 times the base shear in the seismic design.

By reducing the distance of the center of the explosive explosion from the structure, the base shear of the structures increases, so that for a 3-story structure, for 4.5 and 3 meters of explosive, the amount of base shear has increased by 26% and 44% compared to the explosion of 500 kg of TNT, respectively.

The results show that reducing the amount of explosive reduces the base shear and reduces the displacement of the floors.

According to the results, removing the corner column in the studied bending frame building is more critical than removing the middle columns. So that for the members that have the highest probability of collapse based on the DCR criterion, for a 3-story structure in the case of removing the corner column, it is 10% higher than the case of removing the middle column.

The probability of collapse for column removal cases has increased with increasing floors of the structure, so that for a 3-story structure for the most critical member, the DCR criterion is equal to 1.3.

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