Numerical study of the progressive failure of high-rise steel construction with composite roof system under blast load effect

Faroughi A.[1]

Abstract:

Progressive failure means, failure or total destruction or part of the structure due to the lack of the ability of a part of the structure which to be damaged and not able to distribute overload for the stability and continuity of the structure. Different improvement methods are accessible to reach buildings where are vulnerable to events in relation to progressive failure. These procedures are designed to provide the property and health of human life. By adopting the optimum improvement manner, taking into account the economic and technical considerations, it can be reduced the value of financial and life loss caused by the progressive failure. In this study, explosion modeling was performed by the CONWEP program (Conventional Weapons Effects Program) in ABAQUS / CAE software. CONWEP is a loading model based on experimental results, in the ABAQUS software environment. The which is very important, is the involvement of the side openings of the ninth column which is done under the conditions of the removal of the eighth floor column and in the post-blast operating conditions reduces the efforts of the ninth-grade deck girders with tensile performance. Considering that the brackets are in design condition for the maximum operating and ultimate traction modes there is always some overcapacity to the design controller mode, which is mainly pressure, therefore, these added capacities are used under abnormally explosive conditions and eliminate the short- and long-term effects of column removal.

Keywords: Blast Load, Progressive Crack, Steel Construction, Tall Structure, Composite Roof, Finite Element Method.

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*Corresponding author Email: faroughi@gmail.com
1 *Assistant Professor, Department of Civil Engineering, East Tehran Branch, Islamic Azad University, Tehran, Iran

1. Introduction

In the last few decades, the occurrence of terrorist attacks, especially in the Twin Towers of the World Trade Center, has widely raised the issue of evaluating and investigating the potential of progressive damage in existing important structures and structures that are in the design phase among researchers around the world. Progressive failure is a situation in which the occurrence of a local failure in a structural member leads to the failure of adjacent members and additional collapses in the building [1]. Various factors can cause local failure and finally the beginning of progressive failure in the structure. Among the most important of these factors is the occurrence of an explosion in the structure or a strong collision with the surrounding columns of the structure, in the event of such an event, one or more key load elements in the structure may be damaged and the structure may suffer progressive failure, progressive failure is often caused by causing damage is not proportionate and the structure may be subject to progressive collapse due to a small incident; In other words, in the course of the progressive failure mechanism, the amount of destruction is far beyond the effect of the factor causing it [2]. The current standards that are used for the design of structures against normal loads, generally take advantage of degrees of resistance and ductility in a structural system to resist heavy loads and prevent progressive destruction. which generally used frames with small openings, inherently had the necessary endurance and resistance against progressive deterioration, but the changes in architectural styles combined with the evolution of computer-aided structural design and the use of high-performance materials led to Advanced building systems have long spans, are relatively light and malleable, and therefore are more vulnerable to loading conditions that are beyond the design prediction [3]. Generally, when one of the main load-bearing members of the building, such as columns or load-bearing walls, which are considered to be key members of the building, is destroyed due to explosions or unforeseen accidents, this destruction of all structural members, which are somehow called that key member, is destroyed. relied on, it affects, for example, by destroying a column, a part of the roof of the upper floor that is placed on the column is also destroyed. This destruction, in turn, leads to the damage of other parts of the structure, and this sequence may continue until it leads to the destruction of the entire structure or a large part of it. The phenomenon of progressive failure can be investigated with various analytical methods that include from very simple methods to very complex analyses, which are generally carried out by using finite element software that has full capability to consider Dynamic and non-linear properties of structures can be done. It is clear that the progressive failure phenomenon is a dynamic and non-linear phenomenon due to its occurrence in a very short period of time and the imposition of non-linear deformations on the components before rupture. The methods of reducing the risk of progressive collapse are divided into three main categories of incident control: indirect design method and direct design method. In the incident control method, efforts are made to control and prevent abnormal loading. That is, eliminating an accident, reducing the effects of an accident, and protecting against an accident, the method of incident control does not increase the structure's resistance and is not under the control of the structural engineer and is outside his scope of work. Placing the building in areas far from dense areas and creating a fence around the column to prevent the impact of vehicles are examples of this method. In the indirect design method, minimum strength, ductility and uncertainty are provided in the structure to reduce the potential of progressive collapse. Improving the nodal connections of uncertain production and plasticity are among the techniques in this method. The direct design method is divided into two categories. Special local retrofitting method and alternative path method in special local retrofitting method, which is also known as key member design method, a tool is provided to reduce the risk of local failure. In this method, critical components must be able to withstand hypothetical abnormal loading such as explosion pressure. Another method in this branch is the alternative path method. In this design method, the design is done in such a way that after removing the assumed component, the plasticity of the components adjacent to the removed component is sufficient so that they can allow residents to evacuate the place by creating a new path. Several regulations have been developed in the field of progressive failure, which have evaluated the potential of progressive failure of structures under static and dynamic analysis. The US Department of Defense guidelines require the design of all buildings with three stories or more to consider progressive deterioration [4-6]. The regulations of the US Public Service Administration have also provided methods for examining buildings in terms of the potential for progressive damage [7]. This regulation also deals with the combination of suitable loads to check progressive failure in static and dynamic analysis. The European standard, by classifying buildings into four categories, considers the progressive deterioration for each category differently, and if the scope of damage is too wide by removing the vertical load-bearing component, this component is considered a key component and considering the load. The equivalent of 34 kilonewtons per square meter has been considered for the design of the key component. For the first time, Gross and McGuier in 1983 by creating a computer program with graphic capabilities to analyze and design structures against progressive failure, evaluated both direct design methods including specific local resistance and alternative load path method. They gave Kaewkulchai and Williamson in 2003, using a two-dimensional model, compared two static and dynamic analyzes in the discussion of progressive failure and reached the conclusion that in static analysis, since the dynamic effects caused by column removal are not seen, the answers are low [8]. Ruth and his colleagues in 2006 investigated the equivalent static analysis for progressive failure [9]. Considering that in the regulations of the American Public Service Administration, a coefficient of 2 is used in static analysis to consider dynamic effects, in this study it was found that this coefficient is very conservative and considering a dynamic coefficient of 1.5 in the analysis Static is better for this effect and leads to a more economical design. In 2009, Laskar et al. [10] presented a two-step approach to analyze the progressive failure of buildings under the effect of blast loading. They used two-dimensional reinforced concrete frame models to identify their vulnerability during progressive failure caused by Local damages of key components under blast loading were developed and analyzed. However, in this study, a complete 3D frame was not investigated and only one damaged area was evaluated for a blast loading mode. Silva et al. [11] in 2009 specified a method to estimate damage caused by some blast loading conditions through a series of tests on reinforced concrete slabs. This study was only limited to reinforced concrete slabs, therefore, other structural components such as the key columns of the building needed a similar investigation to identify the response of the structural components against the explosion and the amount of damage caused to them.

Bao et al. [12] in 2010, conducted a numerical study to investigate the dynamic response and residual axial strength of reinforced concrete columns. In this study, the influence of reinforced concrete column parameters, such as the amount of reinforcement used, the amount of axial load and the dimensions of the column, on the response of the structure under the effect of explosion was investigated. They found that the behavior of reinforced concrete under damage conditions may change due to the loss of concrete confinement due to the breakage of reinforcements. Therefore, estimating the residual capacity based on the displacement criterion is not a suitable method for reinforced concrete structures, and a new criterion based on the yielding of materials and concrete confinement is needed. In 2015, Hongha et al. [13] investigated the reliability analysis of conventional reinforced concrete columns and reinforced with FRP under explosive loads. In their study, the dimensions of the column, the amount of reinforcement and the strength of the materials with normal distribution and with the design parameters It was considered as average values, also for the average value and standard deviation of maximum pressure and duration of explosive load in different scaled distances, existing experimental formulas were used. had estimated different scaled values, but they had not addressed the overall response and progressive failure of reinforced concrete buildings under the effect of explosive loads. In 2016, Hadian Far et al. [14] investigated the effect of blast load on steel columns with different cross-sections and their nonlinear behavior. Their results showed that wide wing sections (IPB) have the best performance against blast. Also, by using structural analysis software, they calculated and evaluated the non-linear response of various steel building frames against the shock loads caused by the explosion. Also, by comparing the non-linear behavior of building frames with the number of openings and different floors, they identified the factors affecting the response of the structure. In 2016, Faroughi and his colleagues investigated the effect of progressive failure on double bending frames plus diverging brace [15]. In this study, it was found that for a 5-story structure, the appropriate amount of bracing is 10-20% of the frame opening, and for an 8-story frame, this ratio is between 20-30%. It was also found that the common bracing system has a more appropriate behavior during progressive failure. In 2017, Ling Li and his colleagues investigated the progressive failure of steel bending frames [16]. In this study, the effect of the span-to-depth ratio of the beams on the response of the structure was investigated. In this study, it was found that instead of the length of the span, the ratio of the span to the depth is effective in bearing the beams in the span where the column is removed. In another study in 2017, Tavakoli and his colleagues investigated the effect of earthquake characteristics on the failure potential of flexural steel frames under progressive failure [17]. In this study, 5- and 15-story steel frames were subjected to non-linear static and dynamic analyzes considering the load pattern of the American General Administration. The results of progressive failure analysis showed that the potential of progressive failure is largely dependent on the location of the removed column in the frame as well as the number of floors. Also, the results showed that the dynamic response of the system is largely dependent on the characteristics of the earthquake, including the intensity of the areas, the maximum acceleration of the ground and the frequency content. The results showed that with the increase in the intensity of the arcs due to the increase in the energy input to the system, the final vertical displacement increases at the location of the removed column. Also, removing the side column creates more critical conditions than removing the middle column. After removing the column during the earthquake, the earthquake causes an increase in the amount of displacement at the place where the column is removed. In 2019, Abdelwahed made a general review about the progressive deterioration of buildings. In this study, once again, direct and indirect methods were introduced in the design of buildings resistant to progressive deterioration. The indirect method in which the minimum resistance limit and uncertainty of structures are considered and the direct method in which the chain occurrence of tensile and bending efforts after removing the element is considered [17]. Most of the studies that have been carried out in the field of progressive failure assessment in steel buildings have been two-dimensional models of steel frames, in which the distribution of roof systems is not considered, and this can cause a decrease in the accuracy of the model. while the consideration and calculation of three-dimensional effects as well as the existence of composite roofs can have an influential role in the response of the structure, for this reason, in this research, the three-dimensional finite element model of a 12-story steel building with soft use ABAQUS/CAE software has been simulated and the potential of progressive failure has been evaluated.

2. METHODOLOGY

2-1- Investigation of seismic analysis methods of structures

Due to the capability of Abaqus software in blast modeling, it can be used in blast design to observe and evaluate the results of composite roof thickness reduction and differences in side beam distances and in Abaqus software; you can simulate progressive destruction by stepping up the explosion and removal phase. The main purpose of simulating an explosion in Abaqus software is to simulate the progressive vibration and degradation effects caused by the initial pressure of the explosion wave and its secondary suction simultaneously in the corresponding software.

There are various methods for seismic analysis of structures. The difference between these methods is in assuming linear behavior for the structural elements and how the force is applied. In linear models, it is assumed that the structural elements during the analysis have unlimited resistance and constant stiffness, while in the nonlinear models the structural toughness and stiffness are considered during the analysis.

Accordingly, the methods of analysis are:

Static and Dynamic Linear

Nonlinear static and dynamic

In linear analysis (linear material behavior) only the hardness and resistance of the core members are modeled, in nonlinear analysis of hardness and resistance of both main and non-core members as well as changes in the strength and hardness of these members due to their reduction, must be incorporated into the structural model [8].

2-2- Selection Criteria for Analysis (Linear and Nonlinear) of structures

To choose the method of analysis the engineer can choose the nonlinear analysis method from the very beginning by judging the engineering and consulting the employer, But if it is necessary to prove this in accordance with the rules of procedure or to select the solution by the controller, a preliminary linear analysis of the conditions must first be performed and then by checking the above mentioned cases and controlling the conditions of continuation of the linear path and by changing the analysis method to the nonlinear method [9].

2-2-1 Linear Dynamic Method (LDP)

This is done in two ways: spectral (quasi-dynamic) or temporal (full dynamic).
The basis of these methods is based on modal analysis, that is, the structural response is obtained at each vibration mode and by combining the modal responses, and the overall structural response is obtained. The static linear method is described in seismic codes as the equivalent static method. In this method it is assumed that the main mode of structure is the mode that governs the behavior of the structure and ignores the vibrational modes of the structure. Also, assuming linearity of the main mode shape, the seismic forces are distributed as a series of static lateral loads equivalent to the height of the building [10].

2-2-2 Linear Static Method (LSP)

In this method, the total lateral force is calculated as a coefficient of structural mass. This coefficient is the reaction spectral acceleration. If the lateral force obtained in this way is applied to the structure and the behavior of the linear elastic structure is assumed, the resulting deformation will be equal to what would be expected in a design earthquake. But in formable structures, the behavior of structures during earthquakes goes beyond the linear elastic range. For this reason, to estimate the deformation more accurately, the lateral force is increased by applying the coefficients C, , So that if the force values of this method are applied to the model with linear elastic behavior, structural deformations with non-elastic behavior are estimated [11].

2-2-3 Nonlinear analysis methods

As already mentioned, this method of analysis is divided into two categories: static and dynamic, whose accuracy depends on many parameters depending on the type of analysis. Although elastic analysis and linear estimation provide a good view of the structural capacity and provide the position of the first yielding point, however, it is not able to predict the mechanism of the structural failure and how the forces are redistributed during successive surrenders, and does not provide reliable results on the extent of plastic deformation and consequently the extent of structural damage.
Therefore, the analysis and design of new and old structures cannot be justified by the results of linear analyzes. On the other hand, nonlinear behavior of structures is necessary for damaged buildings that have undergone significant changes after the earthquake, as well as for buildings that are to be seismically reinforced with new techniques [11].
The purpose of nonlinear analysis with each of these methods is to determine the maximum plastic change [12].

2-2-4 Nonlinear Dynamic Method (NDP)

The most accurate nonlinear analysis method is now the nonlinear time history dynamic method. The use of non-recursive time history analysis because the dynamic response is highly sensitive to conditional modeling. Figure1 shows a schematic representation of the nonlinear dynamic analysis process.

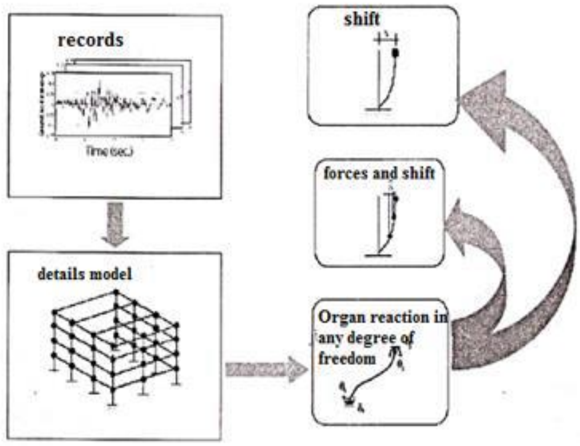


Fig. 1. Schematic representation of the nonlinear dynamic analysis process [11].

3. Method of gathering information

The process of gathering information is the beginning of a process by which the researcher collects library findings and inductively classifies and then analyzes them and evaluates its formulated hypotheses and ultimately issues a ruling and finds the answer to their problem. In other words, relying on the information gathered reveals reality and truth as it is, therefore, the validity of information is important because unreliable information prevents the discovery of truth and fact and the researcher's problem is not properly understood.

An important requirement of any study is the availability of reliable information and the speed and ease of access. Having this information provides the researcher with an opportunity to follow the flow of study and data analysis to evaluate research goals and hypotheses. The researcher also has the opportunity to achieve the desired goals with minimal cost and time [10]. In this study, data collection was done using library method and software modeling.

The library method starts with studying the building rules and then modeling. The tools are both ABAQUS software and ETABS software which is designed and optimized in ETABS software. The research method is computer modeling.

The type of modeling and analysis is dynamically explicit (EXPLICIT DYNAMIC). And three models are used. And 2800 Fourth Edition 2015 standards are used for seismic loading of structures and national building regulations for design and finally for the impact of explosions on the progressive demolition of regulations (UFC, 2016). The 12-story structure is modeled on ABAQUS in full detail with AISC-LRFD. The girder are of cross section I with dimensions of 400mm × 200mm × 20mm × 10mm and beams of cross section I with 0.9m intervals and 250mm × 125mm × 10mm × 10mm. The cross section of the column is a box section with dimensions of $450mm \times 450mm \times 30mm$. The elastic and plastic characteristics of sections are according to table (1) to (3).

Table 1. Elastic and Plastic Capacity of Floor Column 8.

$450mm \times 450mm \times 30mm$	Thickness, width, height	Column cross dimensions
$26333\ mm^2$	Shear cross section	
$6619200\ mm^3$	The basis of the elastic cross section	
$7951500\ mm^3$	The basis of plastic cross section	
$632.0\ tonf$	Plastic shear capacity	Cross section capacity
$158.9\ tonf.m$	Elastic flexural capacity	
$190.8\ tonf.m$	Elastic flexural capacity	

Table 2. Elastic and Plastic Capacity of the Cross Section.

$250mm \times 125mm \times 10mm \times 10mm$	Thickness, width, height	Dimensions of beam cross section
$2463\ mm^2$	Shear cross section	
$489531\ mm^3$	The basis of the elastic cross section	
$565998\ mm^3$	The basis of plastic cross section	
$59.1\ tonf$	Plastic shear capacity	Cross section capacity
$11.7\ tonf.m$	Elastic flexural capacity	
$13.6\ tonf.m$	Flexural capacity of plastic	

Table 3. Elastic and Plastic Capacity of the girder Cross Section.

Thickness, width, height	$400mm \times 200mm \times 20mm \times 10mm$
Shear cross section	$4010\ mm^2$
The basis of the elastic cross section	$1648947\ mm^3$
$1854344\ mm^3$	$1854344\ mm^3$
Plastic shear capacity	$96.2\ tonf$
Elastic flexural capacity	$39.6\ tonf.m$
Elastic flexural capacity	$44.5\ tonf.m$

4. Detailed analysis of the explosion process in ABAQUS

The concrete behavior model is Drucker-Prager plasticity with the definition of progressive degradation behavior under compression. The behavioral model of steel is plasticity and hardness of Johnson-Cook. In the Johnson-Cook model, the main parameters used are hardness strain, strain rate and thermal softness. The relationship of stress and plastic strain yield to this behavioral model is as follows:

$$\sigma_Y = [A + B(\varepsilon_P)^n](1 + C \times ln\dot{\varepsilon}_p{}^*)[1 - (T)^m] \qquad (1)$$

ε_P Equivalent plastic strain, $\dot{\varepsilon}_p$plastic strain rate, , A, B, C, n, m are Johnson-Cook constants. The normalized strain-rate and temperature in the Johnson-Cook behavior equation are defined as follows:

$$\dot{\varepsilon}_p^* = \frac{\dot{\varepsilon}_p}{\dot{\varepsilon}_{po}} \qquad (2)$$

$$T^* = \frac{T - T_o}{T_m - T_o} \qquad (3)$$

$\dot{\varepsilon}_{po}$ Is the user-defined plastic strain rate, $T_o$the standard temperature, and T_m the melting temperature, nonlinear behavior of steel are:

Table 4. Nonlinear behavior of steel.

A	2400 MPa
B	1000 MPa
C	0.045
N	0.4
m	1.2
$\dot{\varepsilon}_p$	0.001

In addition to plastic deformation, an important part of the deformation occurs at high loading rates in the explosion problem area of the batch environment, after rupture and cracking and the expansion of unstable cracks; Therefore, Material Damage can also be decisive in solving explosion problems. This process can be modeled by adding the progressive Johnson-Cook crash model. The progressive failure model states the criterion of initiation and progression of degradation in steel at high strain rates. A general expression for equivalent plastic strain at the onset of degradation in the Johnson-Cook degradation model is given by the following equation:

$$\bar{\varepsilon}_D^{pl} = [d_1 + d_2 \exp(-d_3\eta)][1 + d_4 \ln(\dot{\bar{\varepsilon}}^{pl}/\dot{\varepsilon}_o)](1 + d_5 T^*) \quad (4)$$

Where $\eta = -p/\bar{\sigma}$ Trixie parameter of stress and pressure ratio (p) to von Mises stresses ($\bar{\sigma}$) and d_1 to d_5 is the constants of the failure equation. The parameters of the steel breakdown equation are presented in Table 5:

Table 5. Behavioral Parameters of Steel Degradation.

d_1	0.04
d_2	1.03
d_3	1.39
d_4	0.002
d_5	0.46
$\dot{\varepsilon}_o$	1

Other conventional parameters for linear behavior of steel are presented in Table 6:

Table 6. Parameters of Linear Behavior of Steel.

E	2.05×10^5 kg/m^3
ν	0.29
ρ	7850 kg/m^3

The composite deck slab also includes a reinforced concrete surface restrained by girder steel or concrete slabs and bolts. The properties of this concrete are described by the Drucker-Prager behavioral equation. Definition of batch behavior has been done by modeling the destruction of materials. Table 7 presents the behavioral parameters of C200 Concreate. The stress-strain characteristics of plastic concrete are presented in Tables 8 to 10.

Table 7. Concrete Behavior Parameters.

ρ	2400 kg/m^3
E	16522.9 MPa
ν	0.2
Viscosity ratio	0.00
k	0.66
Dilation angle	10°
f_{b_o}/f_{c_o}	1.66

Table 8. Parameters of Nonlinear Behavior of Concreate under Pressure.

Stress (Mpa)	Nonlinear strain
20.00	0.0001021
19.98	0.0001021
19.38	0.0002042
19.86	0.0003062
77.19	0.0005104
19.65	0.0006125
19.52	0.0007146
19.37	0.0008167
19.22	0.0009188
19.05	0.0009188
18.87	0.0010208
18.69	0.0011229

Table 9. Nonlinear behavior of tensile concrete.

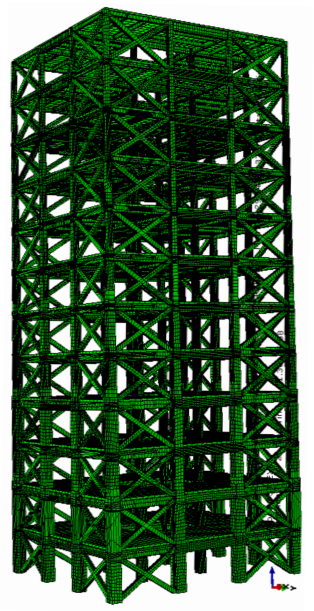
Stress Yield (Mpa)	Cracking Strain
1.49	0.00
0.00	0.09

Table 10. Parameters of Nonlinear Behavior of Concreate under Pressure.

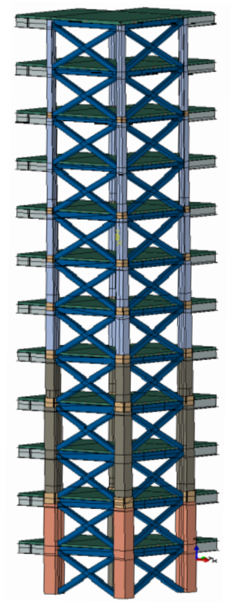
Tensile hardness ratio	0.8
Nonlinear strain	*Breakdown parameter*
0.000	0.000
0.002	0.04

5. Blast Loading and Analysis Results

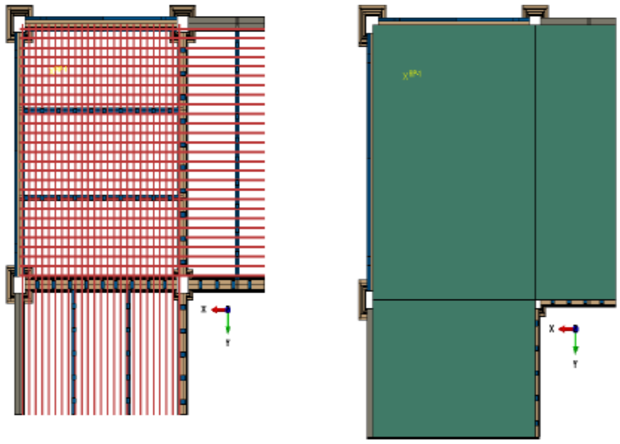
The explosion modeling was performed by the CONWEP program (Conventional Weapons Effects Program) in ABAQUS / CAE software. CONWEP is a loading model based on experimental results, in the ABAQUS software environment. And for this purpose the explosion charge characteristics are defined using the Incident Wave Interaction feature in the Software Interaction Module. The parameters of the CONWEP detonation, including the position of the explosives (TNT), the weight of the explosives and the surfaces exposed to the load are determined in this module. Output Analysis of 200kg TNT in the eighth floor of a 12-story structure, The exact geometry of the 12-story structures in Abaqus is shown in Figure 2. also The compressive stress caused by the explosion on the eighth and ninth floor (IWCONWEP) and all explosive surfaces specified in the interaction module is shown in Fig 3.



(a): Structural geometry at ABAQUS software



(b) The geometry of the 12-story structure in the first quarter of the axis of symmetry

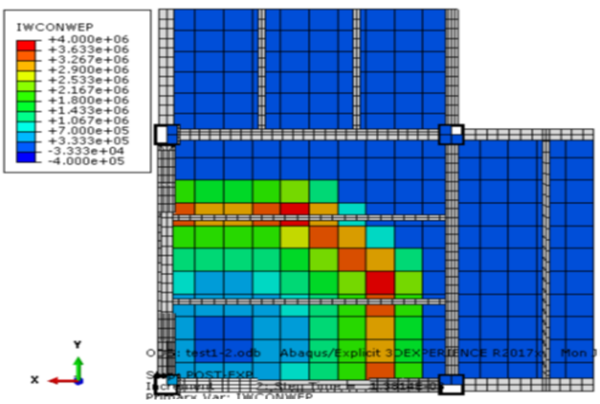


(c): Concrete model and deck bars

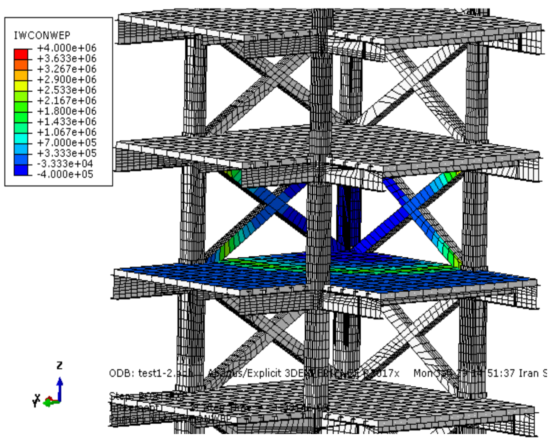
(d): Cutter stuck in the composite deck

Fig. 2. Detailed geometry of 12-story structures in ABAQUS.

According to Figure 3 the compressive stress of the explosion at the level of structural members in the 8th floor (T = 0.0015 sec) The pressure is 40,000,000 Pascal, due to the explosion causing high tension and causing high pressure. Fig. 4 Shows the shear and flexural effort curve at the boundary points of the column (beginning and end of the column). The horizontal chart shows the time in seconds. The vertical diagram shows the force applied to the Newton unit. When the structure is subjected to blast loads, the structure is designed to have sufficient shear strength such that the flexural modes of the failure are controlled. In flexural, the components of reinforced concrete instruments, which are properly steeled, have good formability. While in the shearing, rapture occurs in crisp form. Therefore, it is advisable that the flexural modes of the control be broken. Analyzing the plastic joint on a component under blast load considers all potential situations of plastic joint formation to ensure the maximum shear required. The two-sided interior displacement of the corner pillar on two vertical paths is estimated in terms of the scaled distance from the upper point. The values of these shifts are shown in Fig 5. The horizontal diagram shows the unit scaled distance of that meter. The vertical graph shows the displacement with the unit of meter.



(a): IWCONWEP contour on the floor



(b): Steel members affected by explosion

Fig. 3. Explosive compressive stress on surfaces of structural members in floor 8 (T = 0.0015 sec).

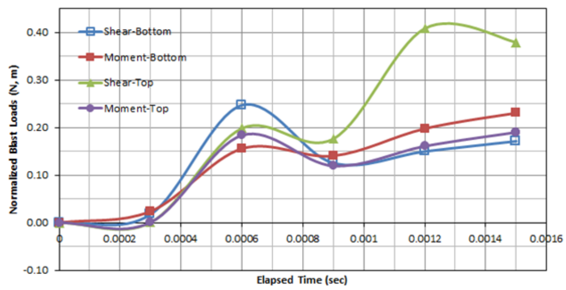


Fig. 4. Shear and flexural effort curve at the boundary points of the column (beginning and end of column).

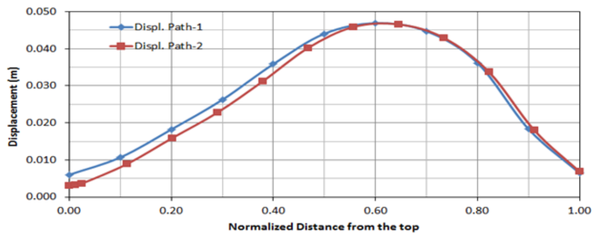


Fig. 5. Displacement of two sides of the column on two intermediate paths in terms of scaled distance.

Changing the location of deck beams on the floor and ceiling of the blast roof is shown in diagram 6. Two paths for the girder and two paths for the composite deck beams have been defined to represent the curve of the beam at the moment of blast loading completion. The horizontal diagram shows the scaled distance of that unit of meter, the vertical diagram shows the displacement with the unit of meter. Figure 7 shows the shear and bending moment at the junction of the beams to the main beams and the column of its horizontal graph is in terms of time in seconds, and the vertical graph shows the force in units of newtons.

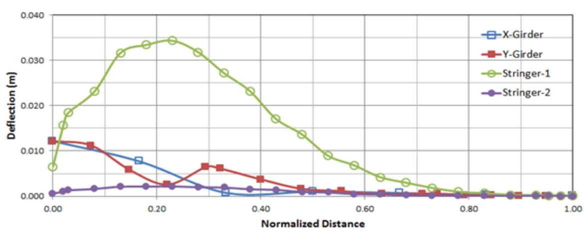


Fig. 6. Girder and beam rise at end of blast time T = 0.0015.

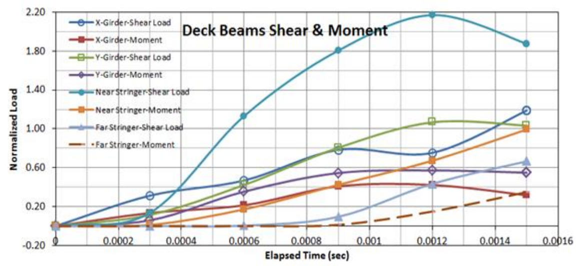


Fig. 7. shear and flexural anchor at beam joint with girder and column.

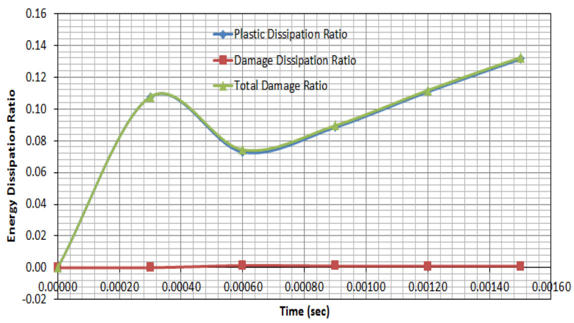


Fig. 10. Structural energy curves under blast loading.

The horizontal diagram shows the unit time of those seconds, the vertical diagram shows the force with the Newton unit. Deformation contours are observed in the concrete floor slab and floor ceiling under blast load in Fig. 8. that the amount of deformation is 10 cm. Due to the explosion, a large number of explosions have entered the structure, causing stress and strain, and the structure has been displaced and there is a camber.

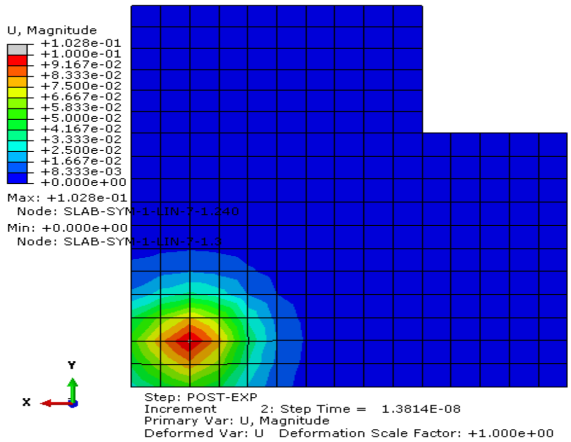


Fig. 8. Slab reinforced concrete floor camber under blast load.

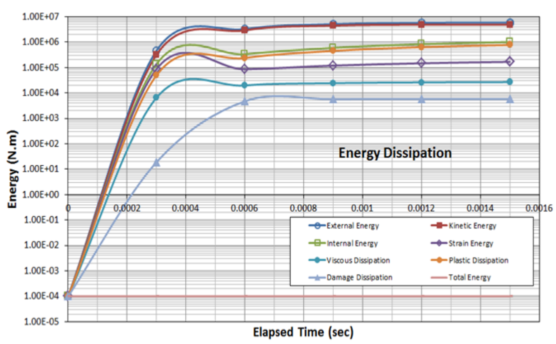


Fig. 9. shows the ratio of energy to detonation energy.

But one of the most important parameters expressing the extent of irreversible deformations, including plastic deformation and degradation in structures on a global scale, the values of damping energy are plastic strain and damping energy 9. These parameters can also be estimated separately for each member. According to Figure 9, the concrete reinforcement slab of the floor deck is 4 cm deformed under the blast load.

According to Figure 10, the energy curves of the structure are under horizontal load explosion loading in terms of time and seconds, its vertical diagram is also its unit energy, Joule. The energy diagrams of the degradation in 0.0006 seconds are 20,000,000 joules. The ratio of the damped energy due to deformation of the plastic and the degradation to the energy injected into the structure by an explosion, which is an indicator of the rate of structural failure and shows the extent of structural failure, i.e. localization or overall failure rate, is shown in Figure 10. As shown in Figure 10, the energy-to-explosion-to-energy ratio curves. The horizontal chart is in seconds and in units of seconds. Its vertical diagram is also Joule's unit energy failure. The energy diagrams of the degradation in 0.0006 seconds are 0.08 joules.

6. Conclusions

In sum, the information obtained from the behavior of the composite deck members indicates that:

1- The explosion effects are mostly dependent on the quantity of explosives and the distance of the explosion point from the structural members, and due to the very short time period of the end of the loading process i.e. the transient nature of the loading, the effects of the explosion are not dependent on the dynamic properties of the structure.

2- The explosion effects are unique to the local deformation around the blast point and the cumulative effects are related to the post-blast period, especially in the wide slab floor deck, which continues to deform and degrade after the blast due to high inertia.

3- Deformation in the structural elements is also more local in nature, and the contribution of the deformation and local effort of the cross-section members is greater than that of the entire cross-section of the blasting point.

4- At the cross-section scale, the column cross-section is better than the cross-section beam, and among the internal reactions, the shear effort is particularly worse at the upper column and cross-section near the blast point. The shear position at column cross section is better than that of deck beams, Therefore, with increasing blast load intensity, the beams are expected to be initially cut and the reinforced concrete slab with considerable delay in structural response after the end of the blast after cutting the main members of the load transfer from slab to shaft has a complete failure between girder.

In the final step, by assuming removal of the column and brackets attached to the eighth floor, transient reactions immediately after column removal and steady state after passing through the dynamic effects of column removal have been studied. The alternate route is the eighth floor roof girders and the ninth floor brackets. The alternating path shall transfer the axial load bearing effect of 815.3 kN and flexural anchors of 21.5 kN.m and 29.7 kN.m respectively in two major directions at the junction point of the removed column to the upper deck in transient and static dynamic mode to the base plane. The bulk of the transfer this time seems to be the flexural and shear behavior of the ninth-grade girders. For this purpose, the pre-blast, post-blast and break-down columns and transient start and finally after transient oscillations and steady state structures have been investigated in ceiling floor girders.

Examination of the girders efforts shows that the operation state attempts for non-coefficient loads after column removal than before by applying a coefficient of 1.33 for the transient mode immediately after the failure and column removal increased by a maximum of 10% in shear force, 14% in the negative anchor of the distal support, and 6% in the flexural anchor of the midline's girder. Due to the increased efforts of the final limit state of the structure, it is expected that the capacities provided in the final limit state design will meet the additional requirements of the burst limit state. The final point, which is very important, is the involvement of the side openings of the ninth column which is done under the conditions of the removal of the eighth floor column and in the post-blast operating conditions reduces the efforts of the ninth-grade deck girders with tensile performance. Considering that the brackets are in design condition for the maximum operating and ultimate traction modes there is always some overcapacity to the design controller mode, which is mainly pressure, therefore, these added capacities are used under abnormally explosive conditions and eliminate the short- and long-term effects of column removal.

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