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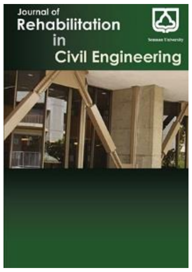

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Comparison between Alternative Load Path Method and a Direct Applying Blast Loading Method in Assessment of the Progressive Collapse

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ABSTRACT

Extensive research has been focused on the progressive collapse analysis of buildings and most of them are based on the alternative path method (APM) with sudden removal of one or several columns. However, in this method the damage of adjacent elements of removed columns under blast conditions was ignored and this issue can lead to an incorrect prediction of progressive collapse. Therefore, in this study to evaluate the alternative load path method in predicting the progressive collapse due to blast loading, a 3-D finite element model of a 7 storey steel building simulated and the behavior of structure was studied using the direct applying of blast load method and alternative load path method. For simulating and applying the blast loading and assessment of their direct effects on structures, a blast load equivalent to 1 ton TNT was considered at a distance of 4 meters from the corner of the structure. The pressures of this blast in 4 loading cases are applied to the adjacent structural members and the structural response has been examined. Finally, the exciting forces in adjacent structural members of blast site in each case have been compared. The results show that in assessment of the potential of progressive collapse occurrence by considering the blast loading as the initial reason of failure, the structure response will be different compared with the alternate load method that in which the initial reason of progressive collapse was ignored.

1. Introduction

After several disastrous building collapses, concepts such as progressive collapse and robustness of structures have been reflected in many research papers and resulted in new codes and guidelines available in Europe [1] and in the United States: [2, 3]. The collapse of an entire structure or an essential part of it that is disproportionately large compared to the initiating local damage is considered a progressive collapse. In addition to the design guidelines, the mentioned standards provide provisions for the progressive collapse analysis of newly designed and existing structures. The main objective of such analysis is the assessment of the potential for progressive collapse. The behavior of the structure is analyzed in terms of the alternate load paths, tie forces, connection redundancy and resilience, and catenary or compressive arching actions of the structural members [3]. One of the main causes of progressive collapse is the explosions occurring near the construction sites. Explosions can be categorized on the basis of their nature. It can be a bomb, a gas-chemical explosion or an airplane attack etc [4]. An explosion can cause damage on the building's structural frames, which may even cause structural collapse. More and more researchers have started to refocus on the causes of progressive collapse in building structures, seeking rational methods for the assessment and enhancement of structural robustness under extreme accidental events. In the United States, the Department of Defense [2] and the General Services Administration [3] provide detailed information and guidelines regarding methodologies to resist progressive collapse of building structures. Both employ the alternate path method (APM). The methodology is generally applied in the context of a “missing column” scenario to assess the potential of progressive collapse by directly removing a column. Most of the published progressive collapse analyses for entire buildings or their components are based on the alternate load path method with column removal. Marjanishvili, presented, in a general manner, four successively more sophisticated analysis procedures for estimating the progressive collapse hazard: linear-elastic static, nonlinear static, linear-elastic dynamic and nonlinear dynamic [5]. Izzuddin et al. presented a design-oriented methodology for progressive collapse assessment of multistory buildings. The proposed assessment framework consists of three stages: nonlinear static response of the damaged structure under gravity loading, a simplified dynamic assessment to establish pseudo static curves, and ductility assessment of the approach to progressive collapse assessment of real steel-framed composite multistory buildings [6]. Hartmann et al. and Moller et al. presented an approach for a simulation of the inverse problem optimization of the structural collapse initiated using controlled explosives. The multilevel methodology of authors, oriented toward uncertainty analysis, is based on multi-body models accompanied by a priori finite element analyses (FEAP) and by transient finite element calculations (LS-DYNA) performed on the computational cluster. The simplified multi-body simulations are then implemented for fuzzy analysis [7, 8]. Song et al. investigated the progressive collapse performance of an existing steel frame building in situ by physically removing four first story columns from one of the perimeter frames. Design methodologies and simplified analysis procedures recommended in design guidelines were evaluated using the experimental data. It was also indicated that 3-D computer models are more accurate in simulating the response of buildings to removal of columns because 3-D models can account for 3-D effects including the contribution of transverse members resulting in solutions that are more conservative [9]. Hosseini et al. [10] and Yousefi et al. [11] investigated the vulnerability of a 10-story office steel moment resisting frame, and concluded that removing a corner column in the ground floor leads to failure of the adjacent bay. Presented numerical case studies based on the linear static analysis showed the importance of incorporating 3-dimensional effects, especially at the part of the structure where a column is notionally removed. A review of recently published numerical studies of progressive collapse behavior shows some clear tendencies. In most of the work, commercial nonlinear FE programs are implemented, such as: ABAQUS, ADAPTIC, FEAP, LS-DYNA, and SAP2000. Beam element models dominate, and most of the considerations are confined to 2D subsystems. Numerous simplifications applied in the models are justified by the required limitation of the computational time and resources. However, in most of the works where the APM method has been used, the damage that might be induced in the adjacent structural members by blast loads has been neglected. These simplifications may lead to inaccurate prediction of the structural collapse. Therefore, in this paper a procedure has been proposed for progressive collapse analysis of common steel building structures subjected to blast loading. A 3-D numerical model with the direct simulation of blast load has been used to study the real behavior of a 7 story building under the blast loading.

2. Numerical model

2.1. Prototype structure

A 3D prototype model of a 7-stroy steel building has been developed in SAP 2000 [12] with the typical storey plan shown in Fig. 1. The floor height adopted is 3.20 meters for each level. The floor system is a full shear interaction metal deck with a slab thickness of 150 mm; the shear studs are evenly distributed along the steel beams. The steel rebar used in the rebar mesh for the slabs is A252. Intermediate steel moment resisting frame acts as the lateral force resisting system of the building in both X and Y directions. The connections between beams and columns are rigid and are made of the St37 steel. The yield and ultimate stress of St37 steel are 2400 kg/cm2 and 3700 kg/cm2 respectively. The conventional design of the structure was carried out according to the tenth topic of the Iranian Building National Regulations [13] by using the SAP2000 software. The 3D view of the building in SAP 2000 is shown in Fig. 2. Dead, live and earthquake loads were calculated based on the Sixth topic of the Iranian Building National Regulations [14]. The structural design of the building was done to ensure the selection of near-optimal levels of stresses and lateral displacements of the structure on one hand and on the other hand, design of components to a have simple and uniform arrangement. In order to study the progressive collapse of structures in future, the effect of each of the various members on the general behavior of the structure can be analyzed in an appropriate and comprehensible manner. The results of structural design are presented in Table 1.

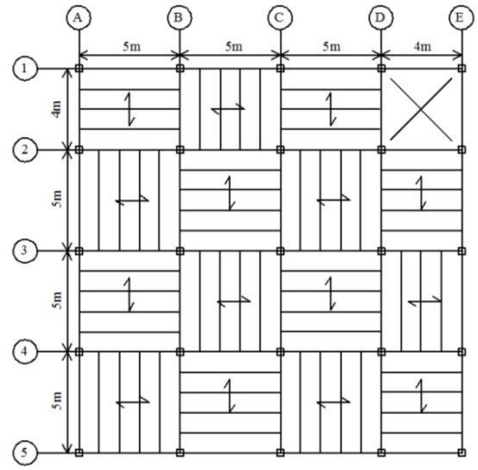


Fig. 1. Typical plan of 7-story prototype building

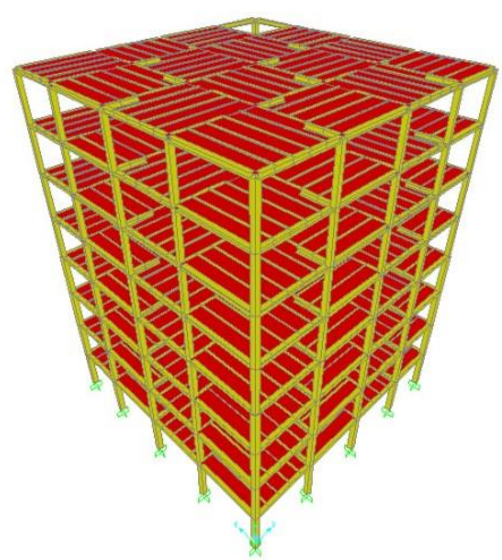


Fig. 2. view of the building in sap 2000

Table 1. Column and beam sections prototype structure

Story	Column	Main beam	Composite beam
Base	Box 35×35×1.6	2IPE 300	IPE 300
1	Box 35×35×1.6	2IPE 270	IPE 300
2	Box 35×35×1.6	2IPE 270	IPE 300
3	Box 35×35×1.6	2IPE 270	IPE 300
4	Box 30×30×1.6	2IPE 270	IPE 300
5	Box 25×25×1.0	2IPE 270	IPE 300
6	Box 25×25×1.0	2IPE 270	IPE 300
7	Box 25×25×1.0	2IPE 270	IPE 300

2.2 Finite element modeling

Finite element modeling of the structure was performed using ABAQUS package [15]. BEAM element has been used to model all the beams and columns. The slabs are modeled using the four noded Shell element. Reinforcement was embedded in each shell element using the REBAR element as in smeared layers. The beam and shell elements are coupled together using rigid beam constraint equations to ensure the composite action between the beam elements and the concrete slab. Nonlinear material characteristics have also been incorporated in the model. The material properties of all the structural steel components were modeled using an elastic–plastic material model available in ABAQUS. The plastic part is defined as the true stress and logarithmic strain. During the analysis, ABAQUS calculates values of yield stress from the current values of plastic strain. It approximates the stress-strain behavior of steel with a series of straight lines that join the given data points to simulate the actual material behavior. For this purpose, any number of points can be used. In this study bilinear model was used. The material will behave as a linear elastic material up to the yield stress of the material. After this stage, it goes into the strain hardening stage until reaching the ultimate stress. Steel is an isotropic material which has good ductility and strength. It generates significant deformation prior to failure. The reinforced concrete material was modeled using a concrete damage plasticity model. The adopted concrete properties were: Young's modulus, 24757×106 Pa, Poisson coefficient, J= 0.2, and density, ρ =2400 kg/m3. The material properties of the rebar in the elastic and plastic ranges have been shown in Table 2.

Table 2. Material properties of the rebar in the plastic range

Density (ρ)	7.85×10^{-6} Mpa
Poisson's ratio(μ)	0.3
Modulus of elasticity (E)	2.05×10^{-6} Mpa
Yield stress (Mpa)	Plastic strain
280	0
370	0.09

The shell elements are integrated at 9 points across the section to ensure that the concrete cracking behavior is correctly captured. The models are supported at the base of the ground floor columns. The mesh representing the model has been studied and is sufficiently fine in the areas of interest to ensure that the developed forces can be accurately determined. The continuity across the connection is maintained by the composite slab acting across the top of the connection.

2.3 Material behavior

A schematic of the stress-strain curve of the steel material considered for modeling is shown in Fig 4. The nonlinear behavior and dynamic effects of the material due to blast or impact loading are also considered in the simulation. The mechanical properties of the structural steel members under blast loading are affected by the rate at which straining takes place. According to UFC4-023-03 [2] regulations, the Strength Increase Factor (SIF) was used since the yield strength of steel is approximately 25% greater than the characteristic strength. Also, in accordance with the regulations the coefficient of the Ultimate stress of steel is equal to 1.05.

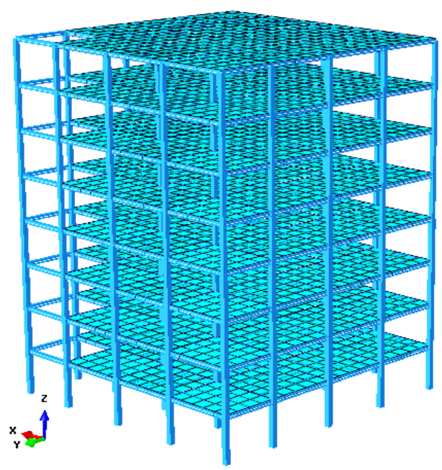


Fig. 3. 3D finite element model of the building

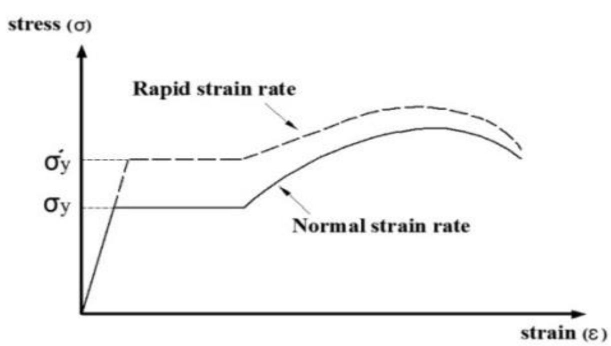


Fig.4. The Diagram of Strength Increase Factor [16]

2.4 Validation of the finite element method used in this study

In order to validate the proposed model, a 5-storey steel frame building which was tested by physically removing four first story columns prior to buildings' scheduled demolition [17] was built using ABAQUS. In the mention study that was conducted by Song and Sezen, both experimental and analytical assessments of the progressive collapse potential of existing buildings were conducted. An actual steel frame building, the Ohio Union building in Columbus, Ohio was tested by physically removing four first story columns prior to buildings' scheduled demolition. Before the building's demolition, four first-story columns were removed in the following order: (1) two columns near the middle of the longitudinal perimeter frame, (2) column in the building corner, and (3) column next to the corner column. As shown in Fig. 5, four of the nine exterior columns were first torched near the top and bottom. Only a small portion of the flange was left intact when the cross sections were cut. The middle column segment between the torched sections was then pulled out by a bulldozer using a steel cable (Fig. 6). During the field tests the changes in column axial forces were measured, and the recorded strains were compared with the analysis results from computer models. A commercially available computer iii program, SAP2000 was used to model and analyze the test buildings, following the General Services Administration guidelines [3]. Two-dimensional (2-D) as well as three-dimensional (3-D) models of each building were developed to analyze and compare the progressive collapse response. The 3D-finit element model of Ohio Union building modeled in this study using the geometric properties and characteristics available of the materials and the first case scenario removing of columns that contains the simultaneous removal of columns 1 and 2 (A5 and A6) as shown in Fig. 5, were chosen to model. Vertical displacement of the building and strain gauges graphs (Number 2 and 15), is shown respectively in Fig.7 to 9.

Fig. 5. Four first-story columns exposed in song's study [14]

Fig. 6. Before and after removal of middle part of a column [14]

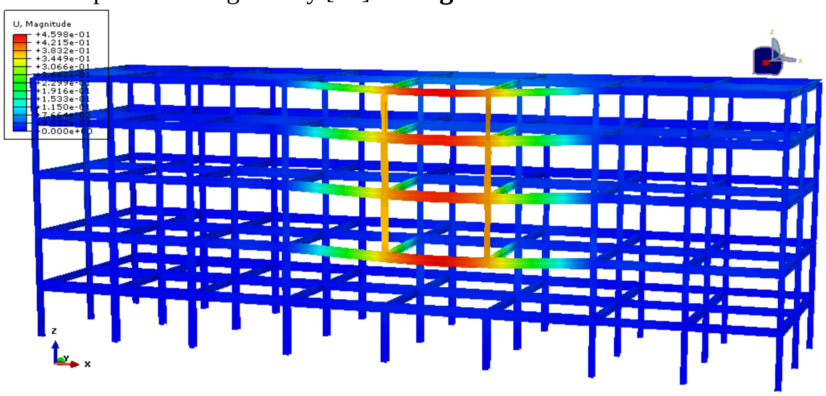


Fig. 7. Displacement of the OHIO building after removing two columns (With the finite element method of this study)

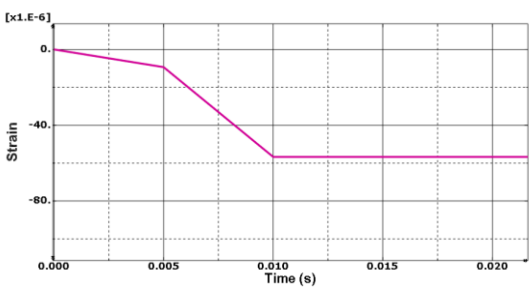


Fig. 8. Strain gauge (Number 2) measurements during column removals

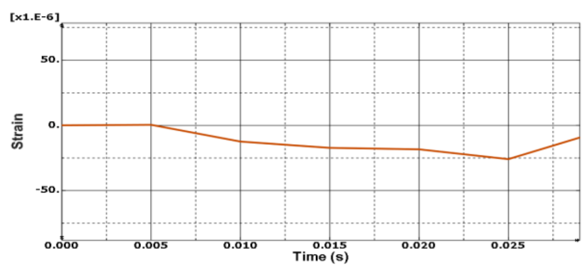


Fig. 9. Strain gauge (Number 15) measurements during column removals

The strain values obtained from analytical models and experimental tests of the strain gauges 2 and 15 during the removal of two columns has shown in Table 3. Comparison between the test results and the modeling results showed a good agreement. One reason for this difference is the lack of floor system modeling. Because there was not enough information about the floor details in hand. More details about this validation are available in reference [18].

Table 3. Comparison between Strain values obtained from numerical model and experimental results

Strain gauge	experimental tests [14]	Nonlinear dynamic analysis (SAP2000) [14]	Nonlinear dynamic analysis (ABAQUS)
2 (Column)	-55×10^{-6}	-32×10^{-6}(42%)	-56×10^{-6}(1/81 %)
15 (Beam)	-37×10^{-6}	-46×10^{-6}(24%)	-25×10^{-6}(32/43 %)

3. Determination of the blast loads on the prototype building

According to Blanc Et al. [19], it is possible to evaluate precisely the shock propagation around the structure and the structure's response, using a fluid–structure interaction method. However, it leads to the complex models. Another possibility is the use of empirical models to compute the load on the structure. This solution is computationally effective and is adopted in the current study with the program ATBLAST [20]. It will be explained in detail in this section. There are many ways in which an explosive device may deliver an attack e.g. the conventional devices like Vehicle bombs, Package bombs, Mortar bombs, Culvert bombs and Incendiary devices [21]. In this paper, the scenario of package bombs is selected for the study, as this type of attack is more difficult to prevent than other attack scenarios such as vehicle bombs.

Loads applied to the structure involve the weight of the structural components (beam, column), dead, live and blast loads. The load combination of DL+0.25LL has been used in dynamic analysis according to the loading pattern presented in the GSA2003 [3]. For simulating and applying the blast loads and assessment of their direct effects on structures, a blast load equivalent to 1 ton TNT was considered at a distance of 4 meters from the corner of the structure (Fig. 10). The pressures of this blast in four loading cases (as shown in Table 4) are applied to the adjacent structural members, and the structural response will be examined. This amount of TNT can simulate explosion of a powerful bomb near a residential building and hence has got technical and practical importance [22].

The principle of the scaling law is used extensively to determine blast-wave characteristics in most design guidlines such as TM5-1300 [23]. It is based on the conservation of momentum and geometric similarity. The empirical relationship, formulated independently by Hopkinson [24] and Cranz [25], is described as cube-root scaling law and is defined as:

$$Z = \frac{R}{W^{\frac{1}{3}}} \qquad (1)$$

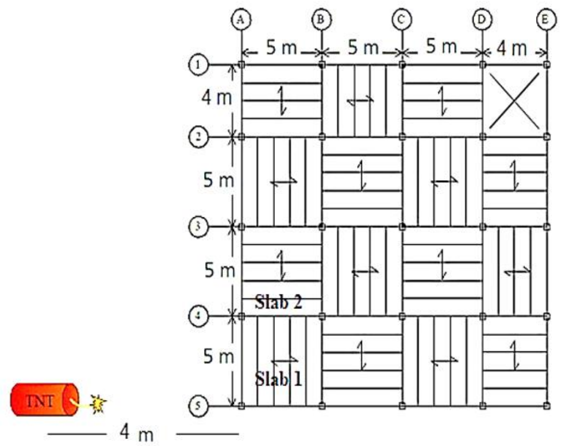


Fig. 10. Location of explosives

Table 4. Different cases of applying the load blast

Case	Column location	Slab location
1	A5 (base)	Slab1 (base)
2	A4 and A5 (base)	Slab1, Slab2 (base)
3	A5 (base and 1st)	slab1 (base and 1st)
4	A5, A4 (base and 1st)	(base and 1st) Slab1,slab2

A general purpose program ATBLAST for predicting explosive effects is used in this study. It is a commercial software for evaluating potential blast damages. It is designed based on the empirical formula of TM5-1300. It calculates the blast loading parameters from an open hemispherical explosion based on the distance from the device. The program allows the user to enter the weight of explosive charge, a reflection angle, minimum and maximum ranges to the charges and the calculation interval. From this information, it can calculate the shock velocity, time of arrival, overpressure, impulse and load duration of the blast loading. According to Yandizo et al. [21], it is usually adequate to assume that the decay (and growth) of blast overpressure is linear. For the positive overpressure phase, a simplification is made where the impulse of the positive phase of the blast is preserved and the decay of overpressure is assumed to be linear as shown in Fig. 11. This simplification is also applied in ATBLAST. The purpose of the study is to provide a fast blast evaluation, and to investigate the response of the building when the blast wave just starts to act on the structure. Therefore, to simplify the model, the effect of blast wave reflections on structural and non-structural elements after the denotation was neglected.

For determining the time-history of blast loading on structure, the weight of explosive charge (W), the reflection angle (α) and the ranges to the charges (R) were given as input to the ATBLAST program. The shock velocity (V), time of arrival (t*), overpressure (Ps), impulse and load duration of the blast loading for A4 and A5 Columns on the base and the first story and the slab1 and slab2 on the base story were obtained. These parameters (Table 5) were then put in the Friedlander blast load equation, which is one of the most accurate and most complete examples of the numerical solution of the blast waves [26]. These parameters are also shown in a time-pressure graph (Fig. 12).

$$P_{(t)} = p_{so} e^{-\frac{t}{t^*}} (1 - \frac{t}{t^*}) \qquad (2)$$

The Friedlander equation was solved using Maple17 and the time history of the blast loading on the structure was obtained and the same was applied to the 3D finite element model in ABAQUS. One such graph is shown in Fig. 13.

In the simulation model, the blast load was applied as an area load acting directly on the slabs and line load acting directly on the beams and columns. All other related information has been shown in Fig. 14. For the applied blast loadings, the times of arrival and load durations are all different due to their distance from the blast charge locations. Therefore, the propagation of the blast waves was also simulated. This is shown in Fig. 14, which clearly shows the blast pressure propagation through the slab at different times.

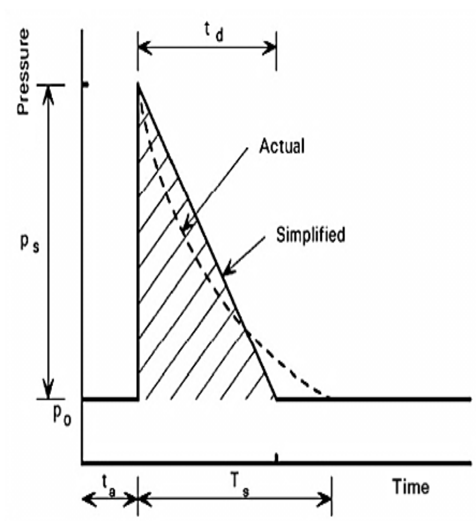


Fig. 11. Simplified blast-wave overpressure profile with impulse by Yandizo et al. (1999) [21]

Table 5. Different cases of applying the load blast

t^*(ms)	P_s (psi)	α (deg)	R(ft)	W(lb)	Story	Load position
1.11	6729.67	20.48	14.01	2204.6	Base	A5
2.45	2416.1	30.9	22.76	2204.6	Base	Slab1
3.41	1467.5	38.47	27.53	2204.6	Base	A4
6.22	655.66	38.7	38.37	2204.6	Base	Slab2
1.66	4042.5	48.18	18	2204.6	One	A5
3.66	1505.36	42.5	28.65	2204.6	One	Slab1
3.84	15.42	46.43	27.85	2204.6	One	A4
7.03	535.3	44	41	2204.6	One	Slab2

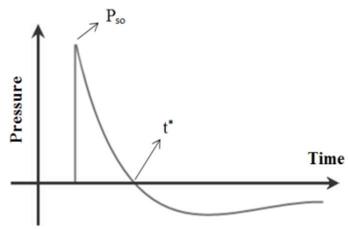


Fig. 12. Time history of the blast loading in a given time-pressure graph for slab 1

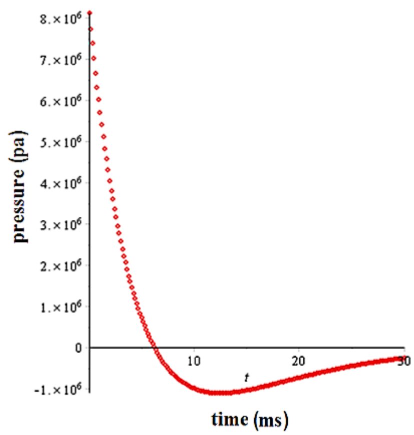


Fig. 13. The parameters of Friedlander equation (Maple17 output)

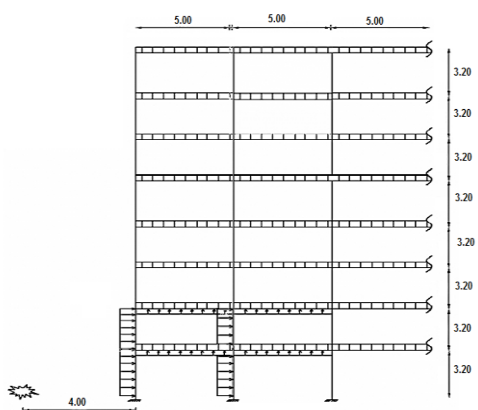


Fig. 14. The manner of applying the blast load on the given structure components (e.g., case4)

4. Evaluation of the structural response to blast loading

The response of the prototype building under the blast loading has been accessed using nonlinear dynamic analysis method with 3-D finite element technique in ABAQUS. The loads are computed as dead loads (which is the self-weight of the floor) plus 25% of the live load in accordance with the acceptance criteria outlined in GSA guidelines. In the analysis, the internal forces, such as axial force, shear force, bending moment, displacements and rotations for each of the members involved in the scenario were recorded.

4.1. Results of Analysis

The results of the analysis for the prototype building have been evaluated in this section. The displacement of structure in the vertical direction (U3), axial force and bending moment diagrams respectively have been presented in Figures (15) to (26). The response of the structure will be investigated through axial force changes, DCR (ratio of demand to capacity), ductility and rotation of members.

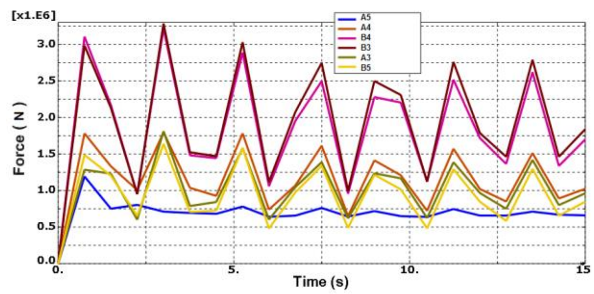


Fig. 15. Axial force of columns (case1)

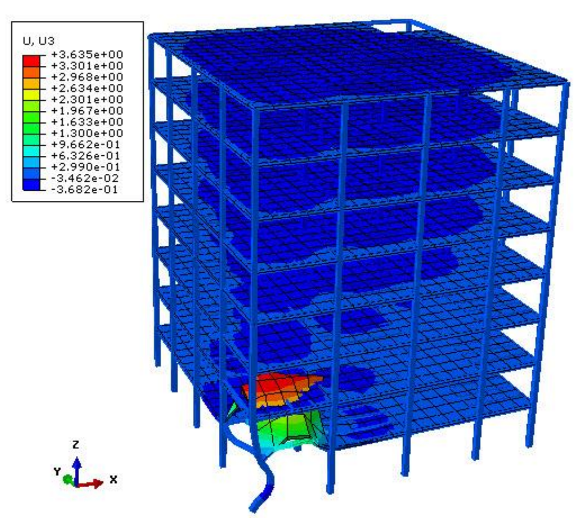


Fig. 16. Vertical displacement contour (case1)

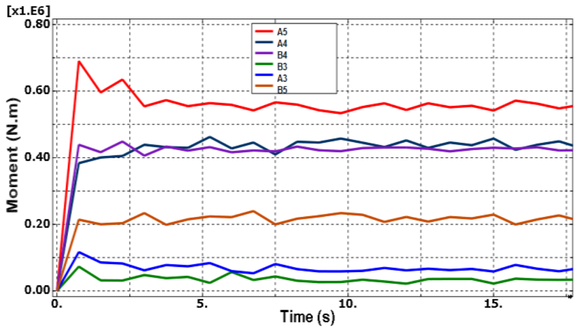


Fig. 17. Moment of columns (case1)

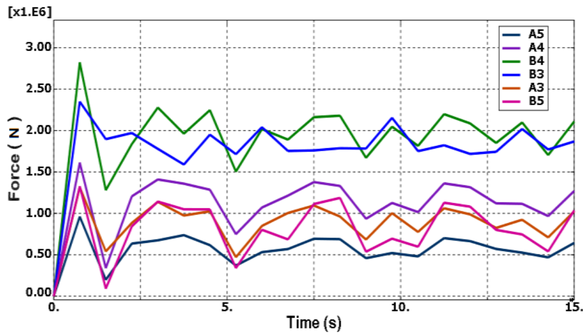


Fig. 18. Axial force of columns (case2)

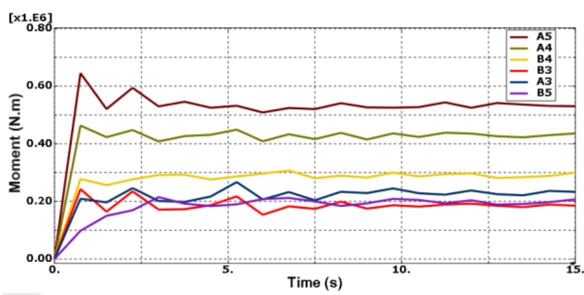


Fig. 19. Moment of columns (case2)

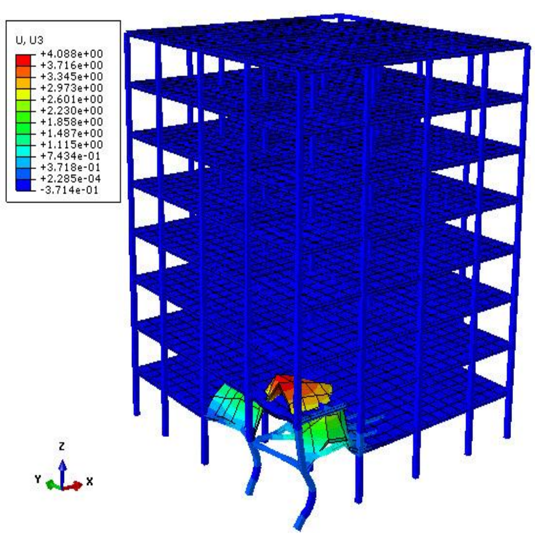


Fig. 20. Vertical displacement contour (case2)

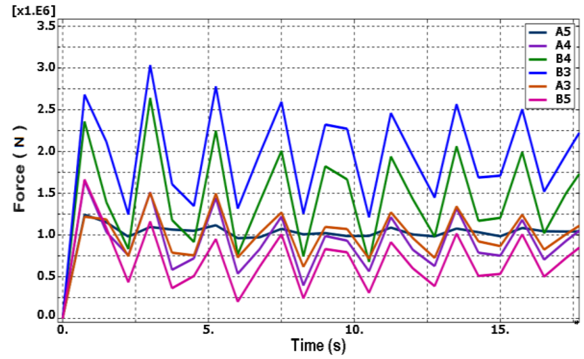


Fig. 21. Axial force of columns (case3)

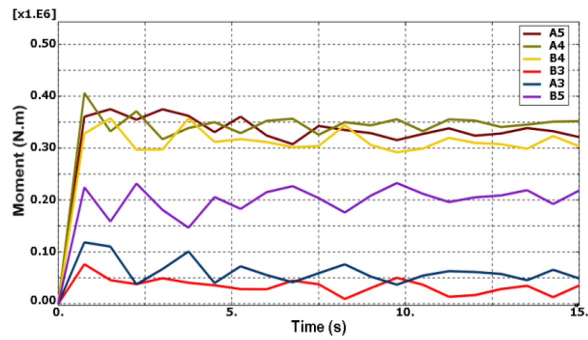


Fig. 22. Moment of columns (case3)

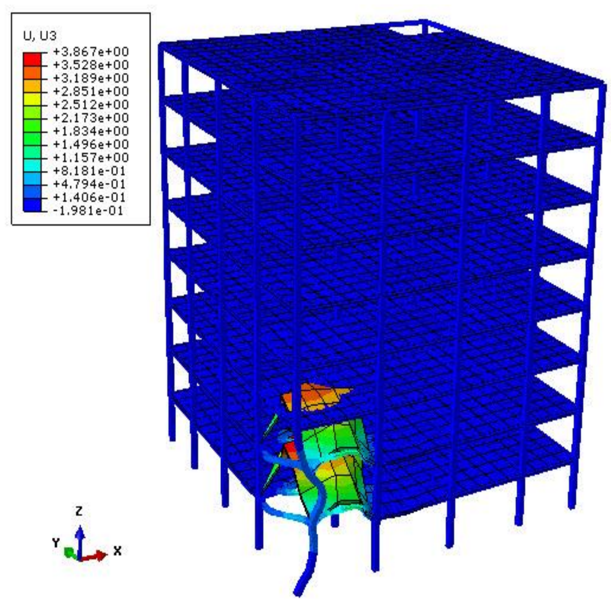


Fig. 23. Vertical displacement contour (case3)

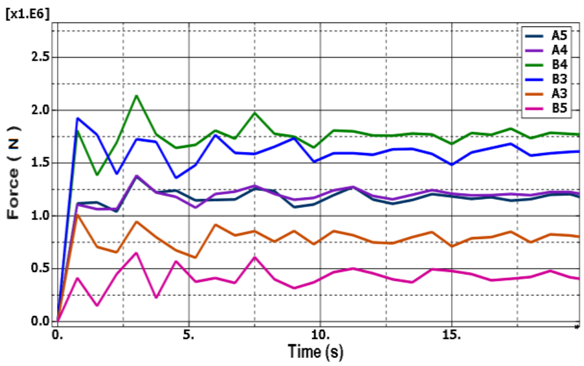


Fig. 24. Axial force of columns (case4)

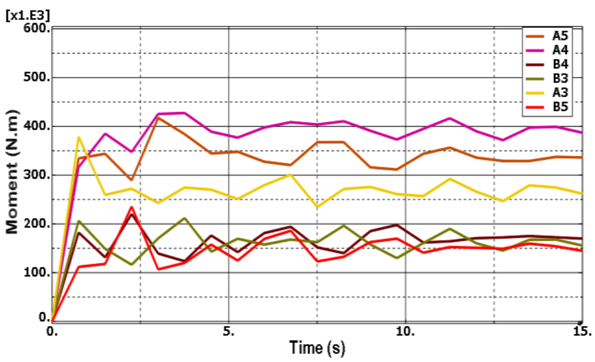


Fig. 25. Moment of columns (case4)

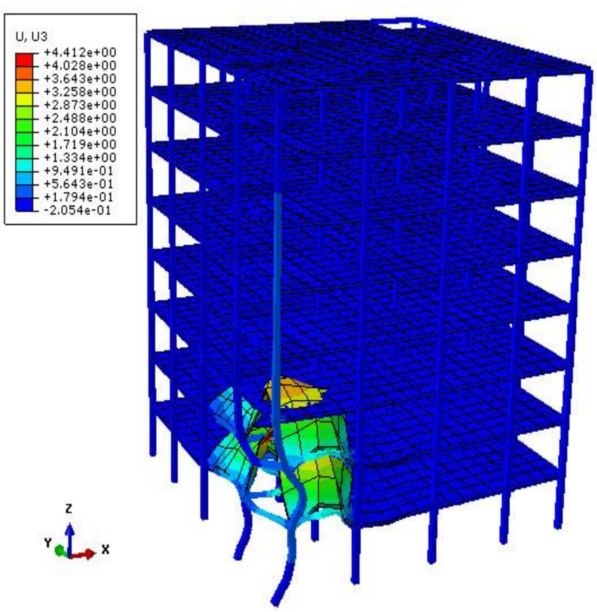


Fig. 26. Vertical displacement contour (case4)

4.2 Acceptance Criteria for Progressive Collapse

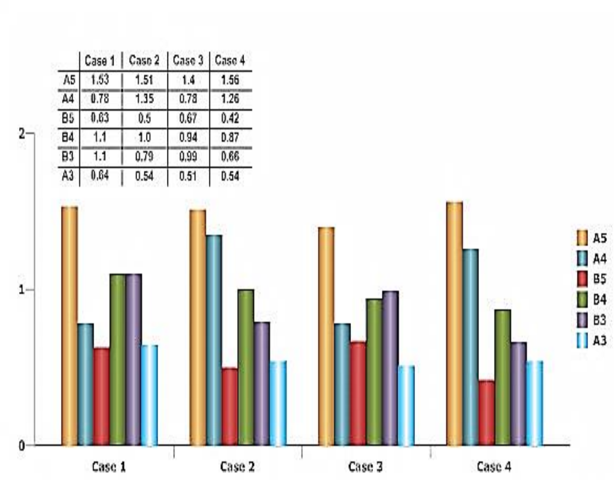
The acceptance criteria for progressive collapse are demand to capacity ratio (DCR), plasticity index and rotation of the members. These criteria will be used to determine the behavior of structures against progressive failure.

4.2.1 Evaluation DCR criteria

To evaluate the results of analysis, the magnitude and distribution of predicted demands are determined by Demand-Capacity-Ratio (DCR). DCR for a given structural component is defined as the ratio of the maximum demand (D) (e.g., moment, M_{max}) of the beam or column to its expected capacity (D) (e.g, ultimate moment capacity, M_p).

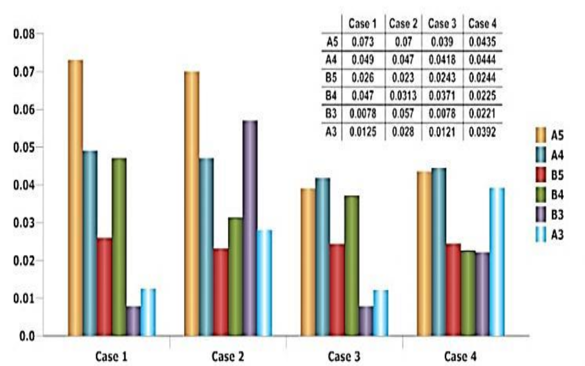
$$DCR = \frac{D}{C} = \frac{M_{\max}}{M_P} \tag{3}$$

Where, the moment demand (M_{max}) of the beam or column is calculated from analysis, and moment capacity, M_p is calculated as the product of plastic section modulus and yield strength. While calculating the M_p for columns, the effect of the axial load has been neglected in this study since the column axial loads were relatively small and did not affect the moment capacity of the cross section significantly. If a DCR value is greater than 2.0, theoretically the member has exceeded its ultimate capacity at that location. However, this alone does not signify failure of the structure as long as other members are capable of carrying the forces redistributed after the initial plastic hinge formation or failure. In this paper, the DCR criterion for bending moment and shearing forces of adjacent members to the blasting location was calculated and the results are shown in Figures (27) and (28).



	Case 1	Case 2	Case 3	Case 4
A5	1.53	1.51	1.4	1.56
A4	0.78	1.35	0.78	1.26
B5	0.63	0.5	0.67	0.42
B4	1.1	1.0	0.94	0.87
B3	1.1	0.79	0.99	0.66
A3	0.64	0.54	0.51	0.54

Fig. 27. Shear DCR values in different load cases



	Case 1	Case 2	Case 3	Case 4
A5	0.073	0.07	0.039	0.0435
A4	0.049	0.047	0.0418	0.0444
B5	0.026	0.023	0.0243	0.0244
B4	0.047	0.0313	0.0371	0.0225
B3	0.0078	0.057	0.0078	0.0221
A3	0.0125	0.028	0.0121	0.0392

Fig. 28. Bending DCR values in different load cases

According to the values of shear DCR and its comparison with bending DCR, it can be concluded that in evaluating the potential of progressive collapse occurrence by considering the blast loading as the initial reason of failure, DCR criteria should be controlled to shear due to the dominance of shear force.

4.2.2 Criteria for Deformation of Members

The performance evaluation criteria for nonlinear dynamic analysis procedures are based on plastic hinge rotation and displacement ductility. Table 5 shows the measurement of plastic hinge rotation angle after the formation of plastic hinge [3]. Based on Fig. 29, plastic hinge rotation angle for beam members on each side of the removed column can be measured between horizontal line and tangent to maximum deflected shape, which is defined by Equation (4):

$$\theta = \tan^{-1}\left(\frac{\delta_{\max}}{L}\right) \tag{4}$$

Where, θ is maximum hinge rotation, δ_{max} is maximum displacement of columns at the location where the column is exposed to blast loads, and L is beam length or column spacing in the longitudinal direction.

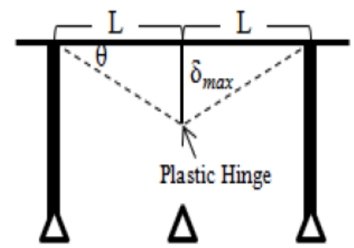


Fig. 29. Measurement of plastic hinge rotation

Displacement ductility ratio (μ) is defined as the ratio of maximum displacement to elastic limit.

$$\mu = \frac{\delta_{\max}}{\delta_e} \tag{5}$$

where, δ_{max} is maximum displacement of columns or beams at a reference point, which can be calculated from ABAQUS program, and δ_e is the elastic deflection limit at that point, which is the vertical displacement when the first plastic hinge forms [3].

In the present study, the prototype steel building is a common building that its performance level is considered on the collapse prevention. The values of displacement ductility and plastic hinge rotation are respectively 2 and 0.035. The maximum value of ductility and plastic hinge rotation in members are calculated and presented in Table 6. As shown, the values of ductility and rotation of structural members in all cases are in the range of regulations.

Table 6. Displacement ductility and rotation of hinges in different load cases

Case 1		Case 2	
μ	θ_p	μ	θ_p
0.0155	1.504	0.0193	1.006
Case 3		Case 4	
μ	θ_p	μ	θ_p
0.0211	0.731	0.031	0.457

5. Progressive analysis of the 7st building using APM method

In order to compare the result with the alternative path method, another identical model was also built. This model used the APM for the analysis.
In the analysis, the column A5 on ground floor was suddenly removed at the same location of its counterpart. The response of the building was recorded, and extracted from the 3-D finite element model. And the results are shown in the following figures. The comparison of the two methods is shown in Table 7.

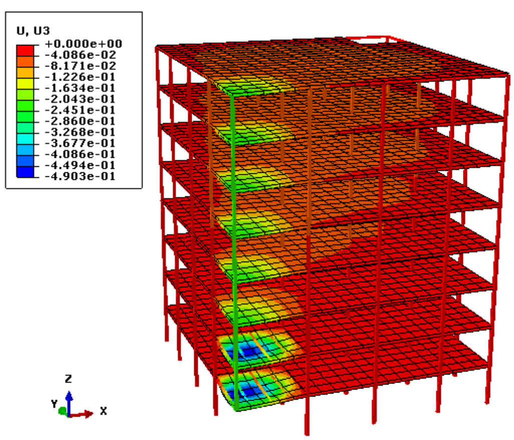


Fig. 30. Vertical displacement of building (APM method)

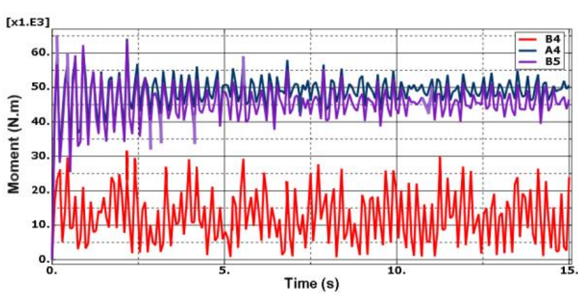


Fig. 31. Moment of columns (APM method)

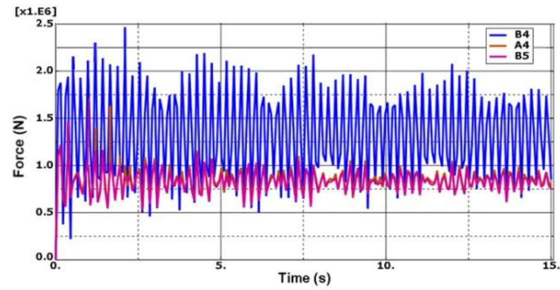


Fig. 32. Axial force of columns (APM method)

Table 7 is the comparison between the direct applying blast loading method and alternative load path method, it can be seen that, with the alternative path method, after the column was suddenly removed, the axial forces of the adjacent columns increased due to the redistribution of the load. While in the direct applying blast loading method the columns surrounding the explosion are less tolerant of axial forces. Therefore it can be concluded that, the alternative path method is stuffier in predicting the axial force in the columns. Also, as can be seen in Table 7 shear forces of columns in the direct applying blast loading method is much greater than alternative load path method. Therefore, when using the alternative load path method to appraise the strength of the structure, the shear capacity of the column should also be controlled.

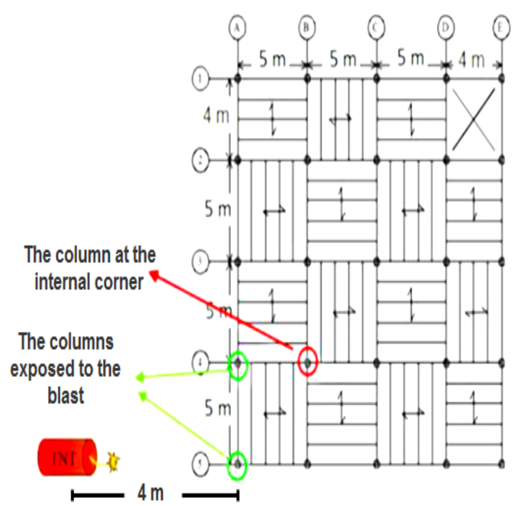


Fig. 33. The key member in the design of the building against progressive collapse due to blast

Table 7. Result comparison of two methods

Force (KN)	Column Location at ground level	Direct applying blast loading method	Alternative load path method
Shear	A4	1100	193
	B4	1100	187
	B5	570	103
Axial	A4	1375	1605
	B4	2125	2475
	B5	1250	1703

6. Conclusions

Following few points can be concluded from this study:

- Structural members close to the blast locations are more vulnerable to blast effects. This implies that external structural members are more exposed to the blast loads and hence, more likely to get damaged.
- Applying blast forces to the structures in a linear or centralized form on the stories can lead to error in the results. But in this study, the percentage of error analysis is decreased due to the application of nonlinear loads considering time.
- The column which is placed at the internal corner of the building takes the maximum axial force when an external column is exposed to the blast load (Fig. 33). In other words, this internal column reaches the plastic stage later compared with the columns exposed to the blast. Therefore, this column can be considered as a key member in the design of the building against progressive collapse.
- The alternate load path method is a method in which the primary cause of the collapse is neglected. While ignoring the primary cause of collapse (Especially blast loads) can lead to an incorrect prediction of the behavior of structures. On the other hand, based on the results of this study the alternative path method ignores the large shear force applied to the column due to the blast loading. Therefore, when using the alternative load path method to appraise the strength of the structure, the shear capacity of the column should also be controlled.

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