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Influence of Sudden Column Loss on Dynamic Response of Steel Moment Frames under Blast Loading

H. R. Tavakoli [a] *, F. Kiakojouri [b]

[a] *Department of Earthquake Engineering, Babol University of Technology, Babol, Iran*
[b] *Department of Structural Engineering, Islamic Azad University, Takestan Branch, Iran*

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ABSTRACT

Modeling buildings response to blast and subsequent progressive collapse interested more and more researchers during the past two decades. Due to the threat from extreme loading, efforts have been made to develop methods of structural analysis and design. In this paper, progressive collapse capacity of steel moment frames was first investigated using alternate load path method, then a nonlinear dynamic analysis was carried out to examine the response of the steel moment frames in blast and sudden column loss scenario. The structural response of the building under sudden loss of column for different scenarios of column removal, with or without external blast loading was assessed in detail. According to the results, progressive collapse potential are strongly dependent on location of column loss. Loss of column can affect overall response of structure under external blast loads. The obtained results provide better insight into the influence of sudden column loss on dynamic response of steel moment frames under blast loading.

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1. INTRODUCTION

Due to different accidental or intentional events, response of structure or structural members subjected to blast loading has been the subject of considerable attention in recent years. Disasters such as the terrorist bombings of Marine Barracks in Lebanon in 1983, the World Trade Center in 1993, the Murrah Federal Building in USA in 1995, the Khobar Towers military barracks in Saudi Arabia in 1996 and the U.S. embassies in Kenya and Tanzania in 1998 have emphasized the need for a thorough assessment of the response of columns subjected to blast loads and also the overall response of the building in column loss scenario to provide adequate protection against blasts and progressive collapse.

Borvik et al. [1] studied the response of a steel container as closed structure under the blast loads. He used the mesh less methods based on the Lagrangian formulations to reduce mesh distortions and numerical advection errors to describe the propagation of blast load. All parts were modeled by shell element type in LS-DYNA. A methodology was proposed for the creation of inflow properties in uncoupled and fully coupled Eulerian–Lagrangian LS-DYNA simulations of blast loaded structures.

Shope [2] studied the response of wide flange steel columns subjected to constant axial load and lateral blast load. The finite element program ABAQUS was used to model with different slenderness ratios and boundary conditions. Non-uniform blast loads were considered. Changes in displacement time histories and plastic hinge formations resulting from varying the axial load were examined.

Among many different methods for analyzing and designing buildings against progressive collapse, the guidelines recommend the alternate load path method [3, 4]. In this approach, the structure is designed such that if one member fails, alternate paths are available for the load and a total collapse does not occur. Alternative load path method is a threat independent methodology, which means it does not consider the type of triggering event; rather it considers structural response after the local failure.

Most of the published progressive collapse analyses

*Corresponding Author Email: tavakoli@nit.ac.ir (H. R. Tavakoli)

are based on alternative load path method with sudden column removal as recommended in mentioned guidelines. In most of the published numerical studies of progressive collapse, open source or commercial nonlinear FE packages are used, such as ABAQUS [5-7], SAP2000 [8, 9] and OpenSees [10, 11]. Most of the considerations are confined to 2D frames using beam element. Detailed 3D numerical study using shell element is very rare due to required computational resources and poor preprocessing ability of general purpose finite element packages. A good example of complete 3D finite element modeling is provided in the literature [12].

Kim and Kim [10]studied the progressive collapse capacity of 2D steel moment resisting frames using alternate path method. The linear static and nonlinear dynamic analyses were carried out for comparison. It was observed that the results varied significantly depending on the variables such as applied load, location of column removal, or number of building story.

Fu [6] investigated structural behavior of the building under the sudden loss of columns for different structural systems and different scenarios of column removal. It was observed that the dynamic response of the structure is mainly related to the affected loading area after the column removal, which also determines the amount of energy need to be absorbed by the structure.

Usually, the progressive collapse analysis will be done under gravity loads, but lateral loads can intensify the effects of gravity. Since in real collapse scenario structure is subjected to lateral loads, this effect can be important.

During military operations or terrorist attacks structures are subjected to successive explosions. After a blast wave strikes structure, outer column are subjected to serious damage or sudden loss due to either design or construction error or direct damage of close explosion. In the event of external blast, more susceptible members are first story's outer columns. Since the structures can be subjected to successive or simultaneous explosion of different directions, interior columns can also be considered as susceptible members.

In this paper progressive collapse capacity of steel moment frames was first investigated using alternate load path method. Structural response of model under sudden loss of column for different scenarios of column removal was investigated. Since progressive collapse is inherently a nonlinear and dynamic event, nonlinear dynamic analysis is more desirable for the assessment of the progressive collapse potential and the collapse mechanism of frames. Accordingly, in this paper nonlinear dynamic analyses were performed for progressive collapse assessment. Linear dynamic analysis method was used for comparison. Then, a nonlinear dynamic analysis was carried out to examine the response of the frames in external blast and sudden column loss scenario. Influence of mesh dependency and strain rate, which may affect the dynamic response of the structure subjected to blast loading was considered in this study.

2. FINITE ELEMENT MODELING

The analysis and design of structures subjected to extreme loading require a detailed understanding of problem. Difficulties that arise with the complexity of the problems nature, which involves high rate of loading and nonlinear material behavior, have motivated various assumptions to simplify the finite element modeling and analysis.

In this study, finite element analysis is performed using the general purpose finite element package Abaqus/Explicit version 6.10. An explicit method solves dynamic response problems using an explicit direct-integration procedure. In an implicit dynamic analysis, the integration operator matrix must be inverted and a set of nonlinear equilibrium equations must be solved at each time increment. In an explicit dynamic analysis displacements are calculated in terms of quantities that are known at the beginning of an increment; therefore, the global mass and stiffness matrices do not need to be formed, it means that each increment is relatively inexpensive compared to the increments in an implicit method. Therefore, explicit method is more efficient than the implicit method for solving extremely short-term events such as blast and impact [13].

2. 1. Analytical Model The model structure is 2D five story steel moment frame, the floor height is 3.2 m and span length is 5m as shown in Figure 1. This steel moment frame is designed to resist both gravity and lateral loads due to strong earthquake according to Iranian building codes [14]. Member sizes of the model structure are presented in Table 1.

In this paper, the beam element in the Abaqus element library was used to model the beams and columns. Selection of the type of element to be used is based on the fact that the investigation considers the global response of the frame under blast loads. For this purpose, beam theory is sufficient. All beam elements in Abaqus are beam-column elements that mean they allow axial, bending, and torsional deformation [13]. However, torsion is not applicable to in-plane behavior of the 2D frames. The beam properties are input by defining the cross-section from the predefined cross-section library. At each increment of the analysis, the stress over the cross-section of beam elements is numerically integrated to define the beams response as the analysis proceeds. The influence of mesh size has been studied and is sufficiently fine to ensure the accuracy of model structure.

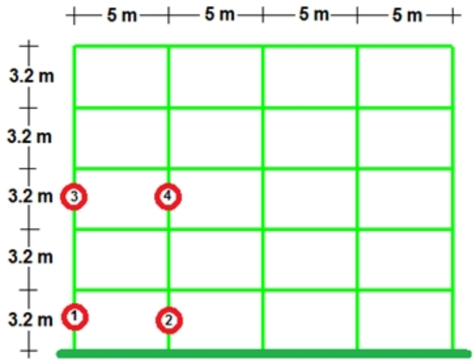

Figure 1. Elevation of model structure and column removal cases

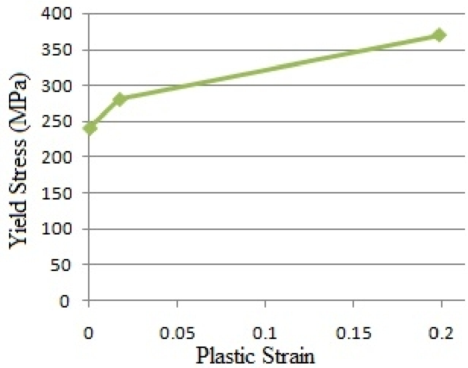

Figure 2. Yield stress versus plastic strain

TABLE 1. Member sizes (All dimensions in cm)

Story	Column (Box)	Beam (H)
1	25×25×2.3	45×19×1.46×0.94
2	25×25×1.9	45×19×1.46×0.94
3	25×25×1.6	45×19×1.46×0.94
4	20×20×1.6	40×18×1.35×0.86
5	20×20×1.3	40×18×1.35×0.86

2. 2. Material Property The adopted material properties were: Young's modulus, E= 210 GPa, Poisson coefficient, υ = 0.3, and density ρ =7850 kg/m^3. The static yield stress was f_y=240 MPa. The plastic property is shown in Figure 2. ABAQUS provides the classical metal plasticity; the elastic part is defined by Young's modulus and Poisson's ratio [13]. The plastic part is defined as the true stress and logarithmic plastic 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 joining 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 [13].

Metallic materials such as constructional steel show an increase in the yield stress with increasing strain rate. In the case of blast or explosion, the rate of loading is very high (in the range of 10^2 - 10^4 s^{-1}); therefore strain-rate dependency is likely to be important. In this paper, strain rate effects are only considered in blast and column removal scenario (Section 5-2). Strain-rate effects are included by adjusting the material dynamic yield stress at each Gauss point according to Equation (1), recognized as Cowper-Symonds relation [15]:

$$\sigma_y = \sigma_0 \left[1 + \left| \frac{\dot{\varepsilon}}{D} \right|^{\frac{1}{n}} \right] \tag{1}$$

where σ_y is dynamic yield stress, σ_0 is static yield stress and D and n are experimentally defined material constants. On the basis of this relation, it is obvious that static and dynamic yield stress ratio depend on deformation speed.

In this numerical study, 3 sets of values for D and n were adopted: (1) D = 40s^{-1} and n= 5; (2) D = 240 s^{-1} and n = 4.74; (3) D = 6844 s^{-1} and n= 3.91. D can be used as a measure of the sensitivity of the strain rate effects and n is a measure of the hardening characteristics of the material [16].

The analyses were conducted with 5% mass proportional damping, which is common for analysis of structures undergoing extreme loads.

3. APPLIED LOADS

3. 1. Applied Loads for Dynamic Column Removal Analysis For nonlinear dynamic analysis, the load DL+0.25LL was uniformly applied in the entire span of frame as vertical load. To carry out dynamic analysis, the reaction forces acting on a column is determined before its removal. Then, the column is removed and replaced by concentrated loads equivalent of its forces. To simulate the phenomenon of progressive collapse, the member forces are removed after a certain time is elapsed as shown in Figure 3, where R denotes the reaction forces and W is the vertical gravity load. In this paper, the forces were increased linearly for five seconds until they reached their maximum amounts, and then kept unchanged for two seconds until the structure reached stable condition, then the concentrated forces were suddenly removed at seven seconds to simulate the dynamic effect caused by sudden and complete removal of the column [10]. Four different cases for column removal are presented in Table 2.

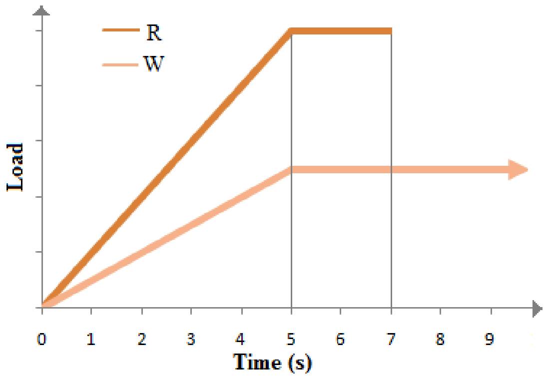

Figure 3. Applied loads for column removal analysis

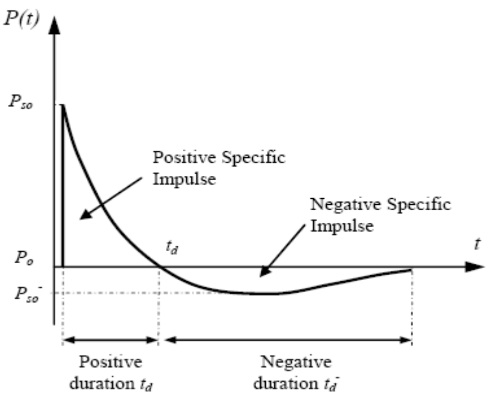

Figure 4. Exponential load pattern [17]

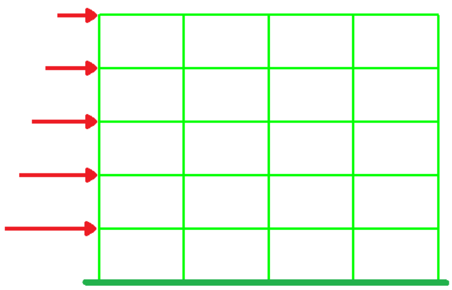
Figure 5. Application of external blast loads on model structure

TABLE 2. Column removal analysis cases

Case	Story	Column
1	First	Corner
2	First	Second
3	Third	Corner
4	Third	Second

3. 2. Blast Loads Figure 4 shows a typical blast pressure profile. The pressure time-history is divided into a positive and a negative phase. In the positive phase, maximum overpressure, P_s^+, is developed instantaneously and decays to atmospheric pressure, P_0, in the time T^+. For the negative phase, the maximum negative pressure, P_s^-, has much lower amplitude than the positive overpressure. The duration of the negative phase, T^-, is longer compared to the positive duration. The pressure time-history in Figure 4 can be approximated by the exponential equation as shown in Equation (2) [17]:

$$P(t) = P_s^+ (1 - \frac{t}{T^+}) e^{\frac{-bt}{T^+}} \tag{2}$$

The positive phase is more relevant in studies of blast effects on structures because of its high amplitude of the overpressure and bigger area under the positive phase of the pressure–time curve [18]. Then, Equation (2) is often simplified by a linearly decaying pressure-time history (Equation (3)) representing triangular load pattern [17]:

$$P(t) = P_{\max} (1 - \frac{t}{T_d}) \tag{3}$$

where P(t) is overpressure at time t, P_{max} and P_s^+ are maximum over pressure in triangular and exponential loading pattern, respectively and b is a experimental constant.

Blast loading can be qualified based on the charge weight and stand-off distance. The amount of charge of explosive in terms of weight is converted to an equivalent value of TNT weight by a conversion factor. That means TNT is employed as a reference for other explosives materials. Estimations of peak overpressure due to blast is based on scaled distance. All equations use scaled distance (Z) for calculating the P_{max} and t_d, which is derived as follows [18]:

$$Z = \frac{R}{W^{\frac{1}{3}}} \tag{4}$$

where R is the distance from the centre of the explosive source in meters, and W is the charge mass of equivalent TNT in kilograms. In this paper, stand-off distance and charge weight assumed to be 15 and 30 m and 1000 Kg of TNT, respectively. The discussion in this section is limited to external air or surface blast. Numerical values and formulas for calculation of t_d and P_{max} are obtained according to relations, which presented in articles [17, 19]. This data is used to determine the dynamic loads on story level that are subjected to such blast pressures and then equivalent concentrated forces are calculated for each story level. (See Figure 5) For structural analysis, constant gravity loads are first applied to the structure and then, lateral blast load is applied and the response time history is calculated.

4. MESH DEPENDENCY

It is well known that the nonlinear explicit analysis for blast loading depends on mesh density. On the other hand, the mesh size is also limited by the computer capacity and the dimensions of the model. One of the major challenges in the numerical study of blast loaded structure is the use of an adequate mesh size.

In this study, three different models consisting of beam elements of size 1, 0.5 and 0.1 m representing coarse, medium and fine meshes respectively, were used to verify the accuracy of the finite element models. According to current results, refining the mesh leads to changes in the response of frame under blast loads, as shown in Figure 6. As expected, using finer mesh increases the displacement, but while using meshes finer than 0.1, results do not change noticeably. That indicates that the mesh size is adequate and model has sufficient accuracy.

In this paper, all other comparisons are made with reference to fine mesh to ensure the accuracy of numerical study.

5. RESULTS AND DISCUSSION

Nonlinear dynamic analysis is performed using the general purpose finite element package ABAQUS/Explicit version 6.10. Unless otherwise specified, all comparisons are made with reference to the rate independent material and fine mesh. In this paper, word "displacement" is used to refer "vertical displacement of column removal point", for horizontal displacement, complete phrase is used.

5. 1. Column Removal Analysis When the corner column in the first story, was removed suddenly (case 1), the node on the top of the removed column vibrated and reached a maximum vertical displacement of 70 mm in linear procedure and 98 mm in nonlinear procedure. For case 2, when the second column in first story was removed suddenly, the node on the top of the removed column vibrated and reached a peak vertical displacement of 51 mm in linear procedure and 59 mm in nonlinear procedure. From the comparison of case 1 and case 2, it can be observed that the building is more vulnerable to the removal of corner columns. Time history of column removal point vertical displacement for two cases is shown in Figure 7 and Figure 8, respectively. It is obvious that maximum vertical displacements obtained by linear analysis are meaningfully smaller than those obtained by nonlinear analysis.

When a column at a higher story was removed, displacement of column removal point significantly increased. This is because less structural member is contributing in energy absorption after column removal. In this analysis, when the corner column in the third story was removed suddenly (case 3), the node on the top of the removed column vibrated and reached a peak vertical displacement of 96 mm in linear procedure and 186 mm in nonlinear procedure. For case 4, when the second column in the third story was removed suddenly, the node on the top of the removed column vibrated and reached a peak vertical displacement of 64 mm in linear and 81 mm in nonlinear procedure. This conclusion can be obtained for any higher story; a column removal at a higher story will induce larger displacement than a column removal at first story. Displacement of column removal point for case 3 and case 4 is shown in Figure 9 and Figure 10, respectively.

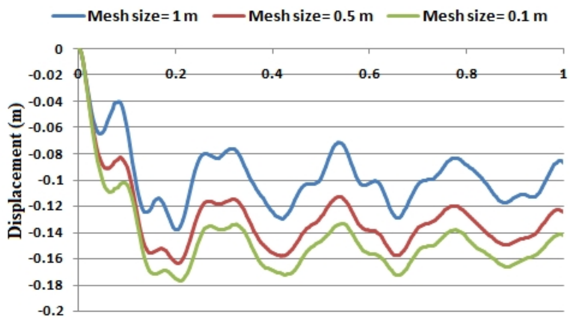

Figure 6. Influence of mesh size on the dynamic response of frame under blast loads. (displacement in column removal point, corner column in first story is removed)

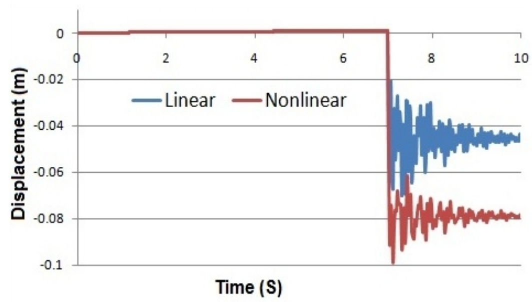

Figure 7. Displacement time history of case 1

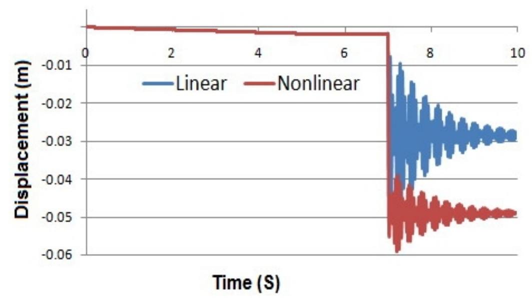

Figure 8. Displacement time history of case 2

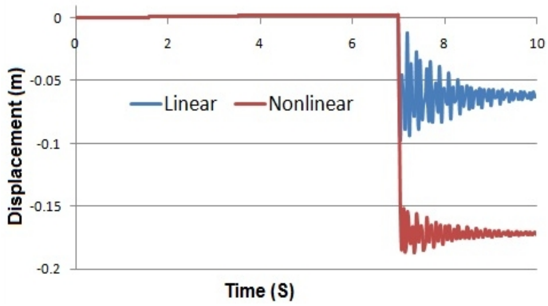

Figure 9. Displacement time history of case 3

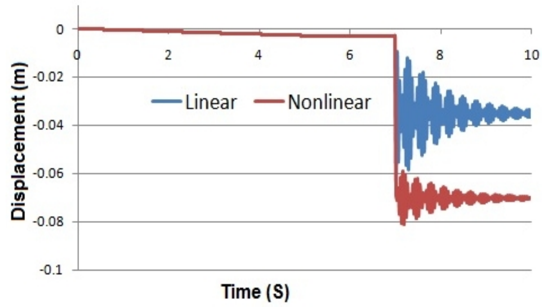

Figure 10. Displacement time history of case 4

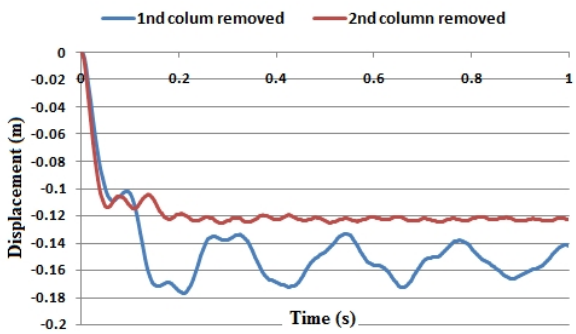

Figure 11. Displacement time history of column removal point under blast loading (stand-off distance is considered as 15 m).

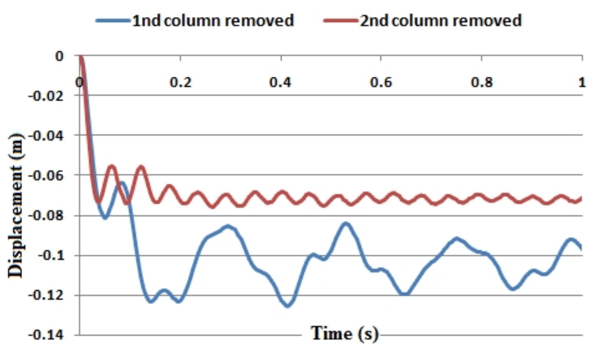

Figure 12. Displacement time history of column removal point under blast loading (stand-off distance is considered as 30 m).

In nonlinear dynamic column removal analysis, the GSA guidelines specify maximum plastic hinge rotation and ductility as acceptance criteria for progressive collapse potential. Ductility is the ratio of the maximum displacement and the yield displacement. The GSA guideline recommends the ductility limit of 20 for steel beams and columns regardless of the connection types. Rotation angle is obtained by dividing the maximum displacement to the length of the beam. The software automatically calculate rotation angle for each analysis step. The acceptance criterion for plastic hinge rotation for steel beam and column is 0.21 radian. According to the current results the limit state for ductility and rotation does not exceed in considered cases.

5. 2. Blast Loading Analysis Time history of displacement of column removal point under blast loading is shown in Figure 11 and Figure 12 for 15 and 30 m stand-off distance, respectively.

From the comparison of column removal analysis with and without external blast loads, it can be observed that displacement of the frame increases for all considered cases under blast loads. The amount of change depends on the location of column removal and scaled distance. The results also indicate that structure is more vulnerable to the removal of corner columns, as observed in the alternative load path method, in the absence of external blast loads.

Location of column removal can be very important, since it can affect drastically the overall response of the frame under blast loads. As indicated in Figure 13, when corner column in first story (case 1) is removed, maximum responses (horizontal displacement) under blast loads will be developed in the opposite direction of blast loads, this is because the structure is unbalanced and tends to unsymmetrical deformation. The same conclusions apply to another stand-off distance, as observed in Figure 14.

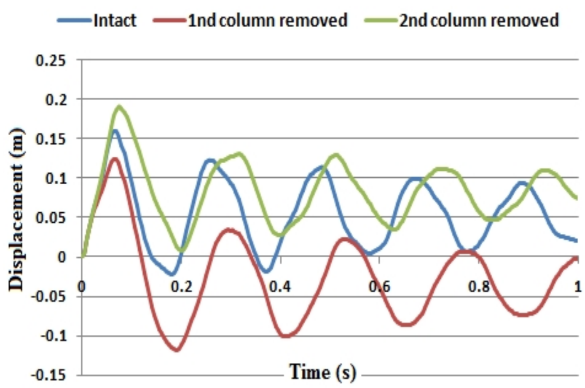

Figure 13. Horizontal displacement time history of roof under blast loading (stand-off distance is considered as 15 m)

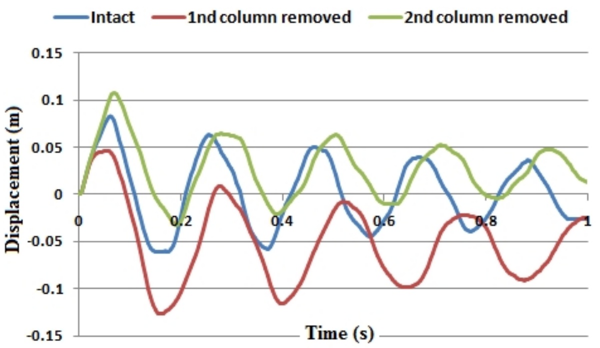

Figure 14. Horizontal displacement time history of roof under blast loading (stand-off distance is considered as 30 m)

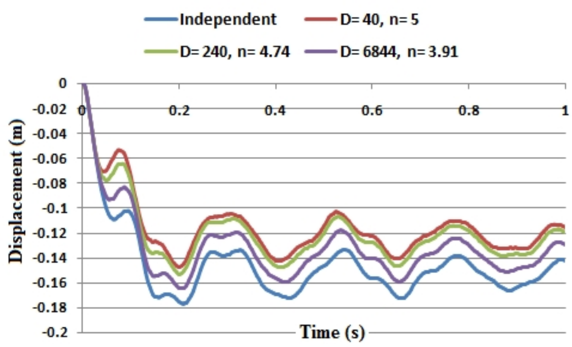

Figure 15. Influence of strain rate on the dynamic response of frame under blast loads

5. 3. Effect of Strain Rate Blast loads produce very high strain rates (10^2-10^4 s^{-1}). This high loading rate would alter the dynamic mechanical properties of target structures. When rate dependency is included, the yield stress increases as the strain rate increases. Because the elastic modulus is higher than the plastic modulus, a stiffer response is expected. Norris et al. [20] investigated steel with two different static yield strengths (330 and 278 MPa) under tension at strain rates ranging from 10^5 to 0.1 s^{-1}. According to their results, strength increase of 9-21% and 10-23% were observed for the two different steel types, respectively. This fact is further confirmed by the observation of results obtained by numerical study. For instance, for case 1 in 15 meter stand-off distance, the column removal point's displacement is 176 mm without strain rate and 147 mm, when strain rate (D = 40 s^{-1}, n = 5) is included. As shown in Figure 15, when strain rate is considered, displacement of frame under blast loads decreased in all considered cases. However, the rate of decrease of displacements is dependent on the values of adopted material constant, D and n and also stand-off distance, which also determine the time duration and maximum pressure of blast loads. Therefore, more precise data would be required for design purpose.

6. CONCLUSION

In this paper progressive collapse capacity of steel moment frames was first investigated using alternate load path method. Nonlinear dynamic analyses were performed for progressive collapse assessment. Linear dynamic analysis method was used for comparison. Then, a nonlinear dynamic analysis was carried out to examine the response of the frames in external blast and sudden column loss scenario. The results of this study can be summarized as follow:

- In column removal analysis, nonlinear dynamic analysis provided larger structural response than linear dynamic analysis. Response of frame in nonlinear procedure is more susceptible to parameters such as location of column loss.
- Potential for progressive collapse is highest when a corner column was suddenly removed, either in first or higher story.
- Column removal at a higher level will induce larger vertical displacement than a column removal at first story, because less structural member contributed in energy absorption after column removal at higher level.
- From the comparison of column removal analysis with and without external blast loads, it can be observed that displacement of the frame increases for all considered cases under blast loads. The amount of change depends on the location of column removal and scaled distance.
- Horizontal vibration time history and development of maximum response under blast loads is dependent to location of column removal, maximum responses will be developed in the opposite direction of blast loads, if the structure is unbalanced and tends to unsymmetrical deformation due to column loss.
- When strain rate is considered, displacement of frame under blast loads decreased. Therefore, the effects of strain rate should be incorporated in numerical study of blast loaded structures. However, the rate of decrease of displacements depends on the variables such as adopted material constant and stand-off distance, the latter also determine the time duration and maximum pressure of blast loads.
- One of the major features in the numerical study of blast loaded structures is the use of an adequate mesh size, because explicit analysis for blast loaded structures depends on mesh configuration. In linear beam, the mesh density does not influence the results of the analysis but for nonlinear analysis, the exact solution of the nonlinear differential equations for a beam is highly complex, therefore, mesh density influences the results of a nonlinear beam element analysis. According to numerical

results, refining the mesh leads to considerable changes in the response of frame under blast loads.

- According to the results, structures' response is very sensitive to stand-off distance, because distance between the charge and the target is one of the main parameters that characterizes blast loads. Therefore, increasing distance will reduce the structural damage under blast loads. Such measures can include either control of public access or barriers and bollards to protect building against vehicles attack.
- The common structures are usually modeled by either brace or shear wall or moment resisting frame, however, in this study, only moment frame has been used for studying the effect of blast and sudden column loss. Therefore, the results apply only to the steel moment resisting systems with almost same height; however, some general conclusions may be applicable to other framed structures.
- As far as is known, progressive collapse potential decreased as the number of story increased. Therefore, low rise frames are more susceptible to progressive collapse due to column loss, with or without lateral blast loads. Further study is still required for accurate evaluation of blast and sudden column loss in steel moment frames with various numbers of stories.

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Influence of Sudden Column Loss on Dynamic Response of Steel Moment Frames under Blast Loading

H. R. Tavakoli [a], F. Kiakojouri [b]

[a] *Department of Earthquake Engineering, Babol University of Technology, Babol, Iran*
[b] *Department of Structural Engineering, Islamic Azad University, Takestan Branch, Iran*

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چکیده

مدل کردن پاسخ ساختمان ها در برابر انفجار و خرابی پیشرونده ی متعاقب آن، محققان بسیاری را در دو دهه ی گذشته به خود جذب کرده است و تلاش هایی برای توسعه ی روش های تحلیل و طراحی در برابر بارگذاری های شدید صورت گرفته است. در این مقاله، ابتدا ظرفیت خرابی پیشرونده در قاب های خمشی فولادی با روش مسیر جایگزین بار بررسی شده است، سپس یک تحلیل دینامیکی غیرخطی به منظور تخمین پاسخ قاب ها در سناریوی انفجار و حذف ناگهانی ستون صورت پذیرفته است. پاسخ سازه برای حالات گوناگون حذف ستون، با یا بدون بارگذاری خارجی انفجار مورد بررسی دقیق قرار گرفته است. بر اساس نتایج، پتانسیل خرابی پیشرونده اساسا وابسته به موقعیت حذف ستون است. حذف ناگهانی ستون، پاسخ کلی سازه در بارگذاری انفجار خارجی را تحت تاثیر قرار می دهد. نتایج حاصل بینش بهتری را برای درک تاثیر حذف ناگهانی ستون بر پاسخ دینامیکی قاب های خمشی فولادی تحت بار انفجار بدست می دهد.

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