Blast-induced progressive collapse of steel moment-resisting frames: numerical studies and a framework for updating the alternate load path method☆

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Abstract

The Alternative Load Path (ALP) method is widely used to assess progressive collapse resistance of steel framed structures. Code-based ALP is threat-independent methodology, implicitly focuses on a very special triggering event, i.e., a small near-field blast, that can lead to complete and sudden column loss. However, in a real blast-induced progressive collapse scenario, characteristics of the triggering event and subsequent initial damage control the structural response. To study these effects, a wide numerical investigation is carried out. First, the code-based ALP method is applied to assess the threat-independent dynamic column removal responses. The results emphasize the importance of initial damage location and building's size. Then, the model structures were analyzed in different blast scenarios. A meaningful difference in the obtained results compared with code-based ALP is observed in both quantity and quality. Finally, a novel methodology (modified ALP) is suggested to update the code-based ALP method to capture the threat-dependent parameters, i.e., column removal time (CRT) and damage level. To serve this purpose, a substructure techniques, i.e., equivalent column model, is developed and validated. The results of three methods (threat-independent code-based ALP, threat-dependent blast analysis (BA) and the proposed modified ALP) are compared, and it is observed that the modified ALP method can effectively adjust the dynamic column removal response to reflect the blast effects.

Keywords: Progressive collapse, Alternate load path, Dynamic column

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1. Introduction

Various natural and man-made hazards can act on civil structures. As long as the effects of their magnitude are smaller than the structural capacity, the load-bearing system works correctly, while structural collapse might occur when abnormal forces or exceptional loading or damage scenarios happen. In framed structures, the majority of collapse incidents occur in a progressive manner. The potential extreme loading conditions that can cause the progressive collapse can be usually traced back to anthropogenic origin, as: bomb explosions, vehicular and aircraft impact, design or construction error, fire, gas explosions, overload, hazardous materials, etc. [1]. Among the various definitions proposed for progressive collapse [2], ASCE 7 defines progressive collapse as "the spread of an initial local failure from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it." [3]. As already pointed out by the Authors, three characteristics should be considered to define a structural failure as progressive collapse [4, 5]: first, the initial failure should be local, second, the initial failure should spread to other structural members and, third, the final collapse state should be much larger than the initial local failure.

Although structural failures have occurred since long times mainly in a progressive manner, however, from modern engineering point of view, the first focus on the specific features of progressive collapse was after the structural failure of Ronan Point Building, in 1968, London. Following this disaster, the studies have been mainly devoted to the understanding of the causes and the ways onto which the collapse propagates. Large scientific contributions have been achieved in the last 20 years since 9/11 terrorist attacks in New York. Heretofore, progressive collapse has been the main topic of several books [6, 7], review papers [2, 4] and thousands of peer-reviewed studies. The results of these scientific works have been reflected in the modern buildings code and dedicated guidelines [8, 9].

Different philosophies are suggested for progressive collapse assessment and design, among them, alternate load path (ALP) is acknowledged by both researchers and practitioners alike. ALP in its original form, that is recommended in progressive collapse guidelines (i.e., code-based ALP), is a threat-independent methodology. This method ignores the triggering events' characteristic, instead, focuses on the structural response after initial local failure, i.e., a column loss. Sudden and complete column loss, as pre-assumed in the code-based ALP is only anticipated in very special scenario, i.e., a small near-field blast.

While ALP is basically a threat-independent method, many attempts have been devoted to consider the features of local failures and triggering events in the code-based ALP, even in its threat-independent nature. These attempts are included to multiple column removal [10, 11], considering column removal time (CRT) [12, 13, 14], strain rate [12, 13, 15, 16] and damage [17, 18]. In addition to these attempts, some researchers have directly focused on the threat-dependent progressive collapse, namely on blast-induced [19, 20], fire-induced [21, 22], impact-induced [23, 24] and multi-hazard progressive collapse scenarios [16, 25].

In real blast-induced collapse scenarios, structural response is drastically dependent on the triggering event and the subsequent initial failure. Parameters, such as charge weight and stand-off distance, control the nature and the extent of the damage. Several recent studies have elucidated that structural responses in blast induced progressive collapse can be different compared with those obtained by code-based threat-independent column removal [26, 27, 28, 29]. As expected, similar results have also been obtained for other triggering events, namely fire [22] and impact [23].

In numerical simulation of blast and progressive collapse, several assumptions and simplifications are usually adopted. Multi-scaling has been frequently used in nonlinear blast and progressive collapse analyses [30, 31], a review on different concepts and philosophies of multi-scaling can be found in [4]. For modeling blast loads, different levels of simplification can be adopted in the simulation. Although simplified equivalent triangular loading is used by some researchers [32, 33], CONWEP model, that is predefined in modern general purpose finite element packages such as LS-DYNA and Abaqus, is adopted by others [27, 29]. A more detailed model of fluid-structure coupling finite element model is rarely used for blast-induced progressive collapse analysis of 3D buildings [34, 35], nevertheless, these techniques are very common in the study of micro models and substructures.

Although considerable research works has been devoted to either threat-independent ALP method or threat-dependent blast-induced progressive collapse analysis, very few attempts are focused on the triggering events characteristics in updating ALP method. The results of the blast-induced progressive collapse studies can be embraced to develop a more realistic ALP method. To this end, a simple and robust approach is suggested. The main purpose of this paper is to provide a framework for adopting the threat-related parameters, i.e., column removal time (CRT) and damage level, from analysis of blast-loaded substructure, i.e., equivalent column, and use these parameters in the ALP analysis of the structure to update the column removal response based on the blast levels, that can lead to more realistic progressive collapse assessment.

2. Research approach

There is a considerable literature concerned with the threat-dependent progressive collapse analysis under different triggering events. These studies provide significant details regarding local failure and the mechanisms of the collapse propagation. However, the impact of triggering events on initial local failure and subsequent progressive collapse has been neglected (or oversimplified) in the current code-based approaches. The opportunity of simulating the effects of an event on the structure, namely a blast acting on a column, instead of modelling its multi-physics, could be very facilitating and helpful in optimized design to resist structural failure. Such details can be adopted for updating and modifying the current code-based ALP method to capture the basic features of the event.

To this end, a numerical approach was adopted for the study of steel moment-resisting frames under different blast-induced initial local damage regimes. Initially, the sample structures were analyzed in code-based threat-independent column removal scenarios, and then, the structures at different simplification levels (multi-scale 3D framed model and equivalent single column) underwent direct blast loads. Finally, based on the results and comparisons of the above-mentioned analyses, a framework based on the blast level is proposed to update the current code-based ALP method. An equivalent single column model is developed and validated to serve this purpose. Details of structural design and finite element modeling are provided in the following subsections.

2.1. Design of the reference model structures

The numerical sample models consist in a 4-story and a 8-story steel moment-resisting frames. The floor height and span length are 3.2 m and 5 m, respectively. A special moment-resisting frame system was adopted. The design was performed by commercial code CSI ETABS [36] based on the criteria of Iranian Standard no. 2800 [37], i.e., the Iranian code for seismic design of buildings. The structures are located in an area with very high seismic hazard according to Iranian seismic risk classification [37]. The response modification factor (R-factor) is 7.5 [37]. The buildings are assumed to be settled on Soil Type 3 (similar to Class D in ASCE 7-16), with shear wave velocity in the range 175–375 m/s at a depth of 30 m. These sample structures are classified as common residential buildings for which a unit importance factor is set. The suggested designing method in Standard no. 2800 is comparable with the one suggested in ASCE 7-16; according to the Iranian code, an equivalent quasi-static method can be adopted for seismic design of regular (in plan and height) buildings that are shorter than 50 m. The obtained shear forces are distributed in buildings' height based on [37], (a similar method is also recommended in ASCE 7-16). Drift criteria are also controlled in the design procedure.

All members were made of St37 steel material with yield stress of 240 MPa, Young's modulus, E = 210 GPa, Poisson's coefficient, $\nu = 0.3$, and density, $\rho = 7800$ kg/m^3. The elevation and the plan of the 4- and 8-story buildings are illustrated in Figure 1 and Figure 2, respectively. Cross-sections of adopted structural members (beams and columns) for 4-story and 8-story model structures are presented in Table 1.

2.2. Finite element modeling

Finite element analysis for both threat-independent and threat-dependent studies was performed using general purpose finite element package Abaqus [38]. While implicit solver was adopted for threat-independent progressive collapse analysis (code-based ALP), explicit method was used in blast analyses. Previous studies revealed that progressive collapse responses of two above-mentioned

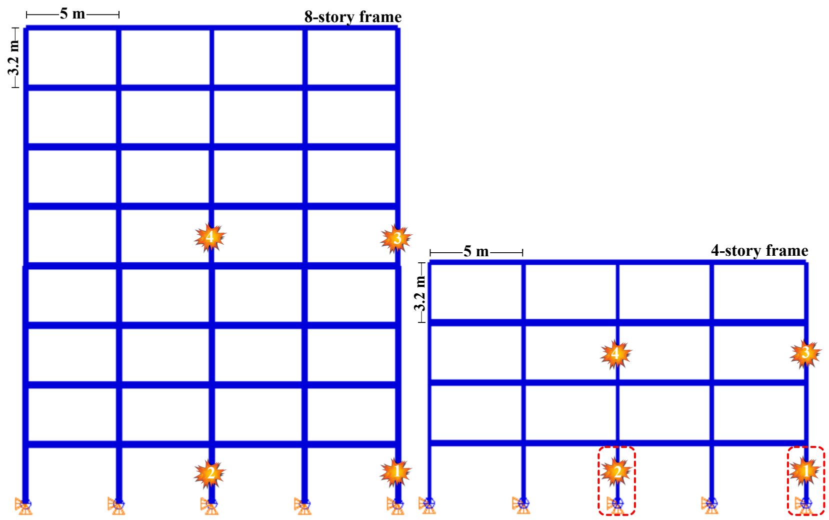


Figure 1: Side views of the model structures and initial failure scenarios that used for threat-independent ALP analysis.

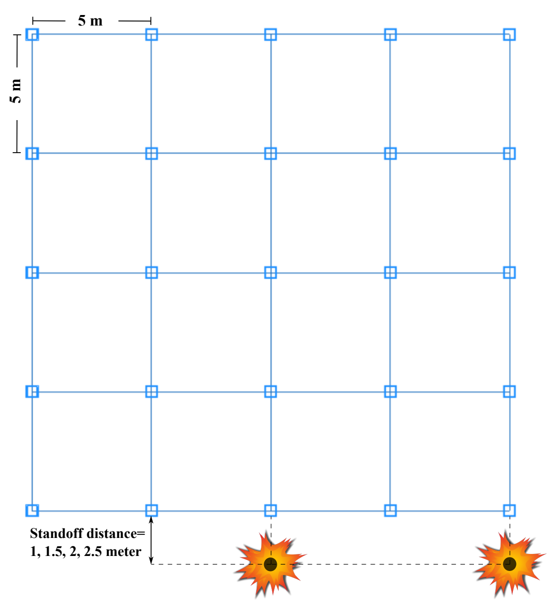


Figure 2: Plan view of the model structures and the positions of explosive charges for threat-dependent blast analysis.

Table 1: Cross-sections of members for model structures.

4-story			8-story		
Story	Column ($B \times B \times t$ mm)	Beam ($h \times b_f \times t_f \times t_w$ mm)	Story	Column ($B \times B \times t$ mm)	Beam ($h \times b_f \times t_f \times t_w$ mm)
1	Box 220×220×18	H 360×170×12.7×8	1-4	Box 340×340×35	H 400×180×13.5×8.6
2	Box 220×220×18	H 360×170×12.7×8	5	Box 300×300×30	H 400×180×13.5×8.6
3	Box 220×220×14	H 360×170×12.7×8	6	Box 300×300×30	H 360×170×12.7×8
4	Box 220×220×14	H 270×135×10.2×6.6	7	Box 280×280×35	H 330×160×11.5×7.5
			8	Box 280×280×35	H 240×120×9.8×6.2

methods are similar, especially when the maximum response is the main interest [12].

Recent studies emphasis that the 3D and slab effects in progressive collapse assessment cannot be ignored [39, 40]. Moreover, type and configuration of the steel connections affect the progressive collapse performance of framed structures [41, 42]. Infills and curtain wall are also not considered in numerical modeling. In the vast majority of existing buildings, curtain walls (if any), are not capable to sustain the blast loads or transfer it to the adjoining columns, while, depending on the material and the adopted configuration the impact of walls can also be important. Therefore, for design purposes that are outside the scopes of the present research paper, a detailed FE model including slab effects, realistic connection behaviour and other possible special structural features is recommended. It should be noticed that current study is devoted to the developing of a new approach. That is, the focus is mainly put on the approach itself, rather than on each single detail of the modeling. Obviously, the proposed framework, can also be used for a more detailed FE model.

In modeling the steel material's mechanical behaviour, the nonlinear part is defined as the true stress versus logarithmic strain. While post-yielding behavior is very effective in cyclic loading conditions, e.g., in an earthquake, for progressive collapse, especially when the absolute column removal response is the main focus, these effects can be safely simplified or even ignored [13]. Therefore, in this study, a bi-linear material model was adopted.

Rate effects in blast-loaded steel structures are common facts, moreover, recent studies have shown that they can also be very important in progressive collapse response, even in a threat-independent column removal scenario [12, 13, 15, 16]. Therefore, strain-rate effects were included using Cowper-Symonds equation in both progressive collapse and blast analyses. Cowper-Symonds equation is a well-accepted model for defining the material behavior at high rates of loading [43]:

$$\frac{\sigma_{yd}}{\sigma_y} = 1 + \left(\frac{\dot{\varepsilon}}{c}\right)^{\frac{1}{q}}. \tag{1}$$

where σ_{yd} is the dynamic yield stress, σ_y is quasi-static yield stress, c and q are Cowper-Symonds material constants and $\dot{\varepsilon}$ is the strain rate, further details can be found in Jones [43]. In current study $c = 40$ s^{-1} and $q = 5$ were adopted, as suggested for progressive collapse assessment of steel frames in [13]. Damping ratio of 5% of the critical damping is applied, as usually adopted for analysis of structures undergoing extreme loads [13]. However, the damping impact on maximum response of framed systems under column removal is negligible and can be safely neglected, as discussed by Kiakojouri and Sheidaii [12].

In nonlinear dynamic analysis, the solution is highly dependent on the mesh configuration. In this study, pre-analyses were performed to ensure that the mesh size is fine enough to not affect the results. Moreover, intervals for output time steps, as highlighted in Kiakojouri and Sheidaii [12], were carefully considered. Other specific modeling details are explained in appropriate section for threat-independent dynamic column removal (Section 2.2.1) and blast analysis (Section 2.2.2) analyses.

2.2.1. Threat-independent dynamic column removal analysis (code-based ALP method)

The code-based ALP analysis were conducted on both 4-story and 8-story model structures, for the all local damage scenarios, to highlight the influences of buildings' height and damage location. The column removal scenarios are shown in Figure 1. For dynamic column removal analysis, a three-step method was adopted [13]. In the first step, the entire building was loaded in a quasi-static manner with the loads of magnitude $1.2DL + 0.5LL$, as recommended in UFC guideline [9]. In Step 2, the selected column was removed. The $*$*Remove* command from the Abaqus library was used to remove the member from the finite element assembly. The duration of the removal is 10 ms [13]. In Step 3, the damped vibrations, i.e., the vertical displacement of column removal point (CRP), were monitored. It was tested that 1 second is sufficient for monitoring the maximum structural response [8, 12]. Static implicit Abaqus solver (*Static, General*) was used for the first step, while dynamic implicit solver was adopted for Steps 2 and 3. Above-mentioned three-step dynamic column removal methodology is graphically depicted in Figure 3.

For threat-independent progressive collapse analysis (code-based ALP), since the global behavior of the model structure in the column removal scenarios is the main focus, all beams and columns are modeled using beam elements from Abaqus library. $B31$-type element with 2 nodes and 6 degrees of freedom per node was used for all members. Mesh size is sufficiently fine for achieving good accuracy based on the results and recommendations reported in Kiakojouri and Sheidaii [12].

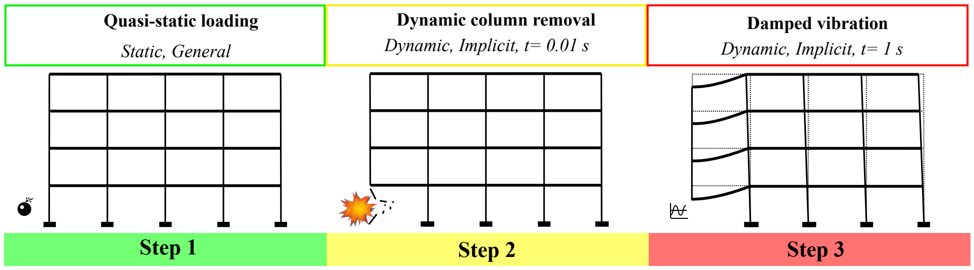


Figure 3: Three-step methodology for threat-independent dynamic column removal (code-based ALP method).

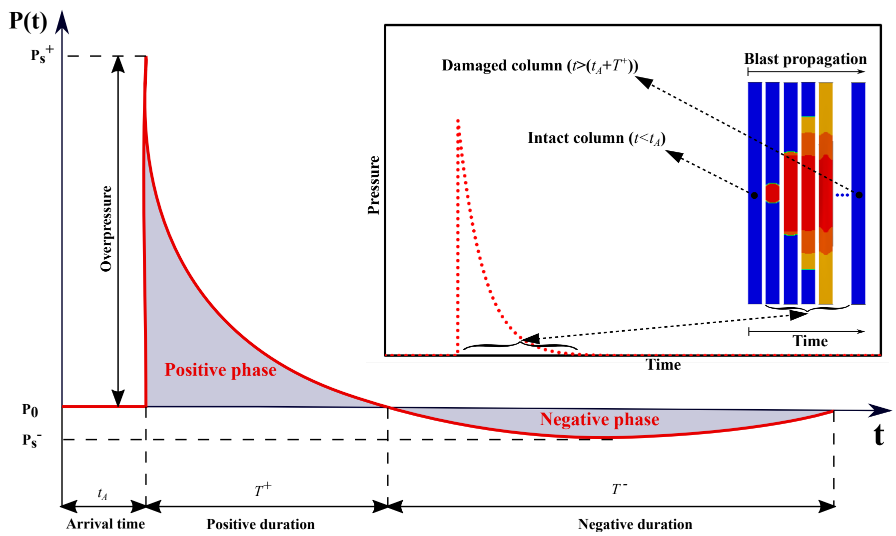


Figure 4: Typical blast pressure time-history, CONWEP pressure time-history and pressure contour on a sample column are shown in the inset.

2.2.2. Threat-dependent blast-induced progressive collapse analysis

The threat for a conventional bomb can be elucidated by two basic parameters; the charge weight (W) and the standoff distance (R). Scaled distance (Z) is usually used to include the influences of both variables ($Z = R/W^{1/3}$). As shown in Figure 4, a typical blast pressure time-history consists of a positive and a negative phase. In the former, the maximum over-pressure, P_s^+, is abruptly developed and it exponentially decays to the barometric pressure, (P_0), in the time (T^+). In the latter, the maximum negative over-pressure, (P_s^-), with lower amplitude but longer duration (T^-) is formed. In the numerical simulation of blast-loaded structures, only positive phase is typically considered. The pressure time-history of Figure 4) can be expressed by Ngo et al. [44]:

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

where $P(t)$ is over-pressure at the time t, P_s^+ is maximum over-pressure and b is an experimental parameter.

In Abaqus, the CONWEP model is predefined and can be employed for blast loading on the model structures [38]. This model uses the scaled distance based on the distance of the structural surface from the blast source and equivalent weight of Trinitrotoluene (TNT). The total pressure on the target is then developed based on the incident pressure, the reflected pressure and the angle of incidence. The total pressure is defined as [38]:

$$P(t) = P_{incident}(t)\left[1 + \cos(\theta) - 2\cos^2(\theta)\right] + P_{reflect}(t)\cos^2(\theta). \quad (3)$$

where $P(t)$ is total pressure, $P_{incident}(t)$ and $P_{reflect}(t)$ are incident pressure and reflected pressure, respectively, and θ is angle between the normal of the loaded surface and the vector that points from the structural surface to the source of the blast [38].

Figure 4 shows a typical blast pressure time-history including positive and negative phases. The blast pressure obtained by element field output variable IWCONWEP from the Abaqus library is shown in the inset of the Figure 4. As depicted in the inset, only the positive phase is produced by CONWEP. The CONWEP model can be used to produce non-uniform time-dependent positive blast pressure distribution on the selected structural surfaces.

In this paper, Scenarios 1 and 2 in 4-story frame were considered for blast-induced progressive collapse analysis (see Section 2.3), as they are the more realistic cases of a voluntary hazardous damage on a construction. The amounts of TNT used as explosive charges in numerical analysis were 100 kg, 225 kg and 450 kg. These weights are associated with the TNT equivalent that can be carried out by a Compact Sedan or a Sedan car [45]. The standoff distances are 1 m, 1.5 m, 2 m and 2.5 m to cover a common range of accessibility and physical protection (see Figure 2). The locations of the blast sources were considered at the mid-height of the columns to maximize the response. Such configuration can be justified, e.g., in the case of difference in level of the street and the building's story levels in a real blast scenario. Figure 5 shows scaled distance ranges used in the numerical study. This range of blast loading produces high strain rate as reported in Forni et al. [16]. In the inset of the Figure 5, the selected charge weights and standoff distances (8 blast scenarios) are listed. It should be noted that the aforementioned loading method is not necessarily suitable for internal blasts, because internal geometry of the building can significantly affect the total pressure which may not be described using Equation 3. In such cases, more advanced blast loading techniques should be applied.

For blast analysis on 3D framed structure, a multi-scale method was adopted. CONWEP can only be used with shell or solid element in Abaqus, therefore, the columns assumed to be under direct blast load in the first story, i.e, Scenario 1 and Scenario 2, were modeled by shell elements (the area indicated by dotted line in Figure 1, penultimate column was also modeled with shell elements), while beam elements were assigned to other members. Four-noded and reduced integration S4R shell elements were adopted for selected columns. The

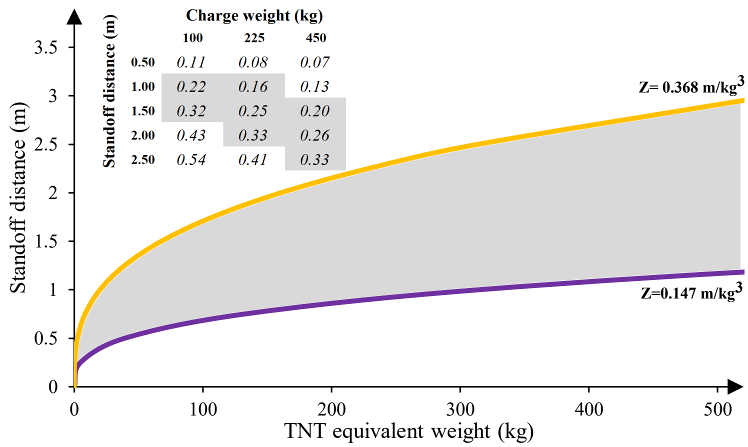


Figure 5: Scaled distance range used in the numerical study, TNT equivalent charge weights are shown in the inset.

S4R element is suitable for structures under extreme loading conditions and has been successfully used in blast-loaded steel structures, because it includes finite membrane strains and arbitrarily large rotations [46, 47]. Enhanced hourglass control was also adopted for the elements that are directly under blast loads. Interaction between shell and beam elements was utilized by $*Coupling$ command from Abaqus library. The selected multi-scale modeling technique is graphically illustrated in Figure 6.

In addition to multi-scale model, the same blast scenarios were also applied to equivalent single column. The equivalent single column model, is a column with geometry and material similar to the column of the frame system. The equivalent column is fixed (displacements and rotations are constrained) at its bottom end. In the top end a slider is considered, i.e. the end is constrained with respect to rotations and displacements, except the vertical displacement, which is allowed. The equivalent column is subjected to a vertical downward force that equals the axial internal force of the element in the whole frame, as presented in Figure 7). The results of blast analysis on equivalent column model are compared to those of the multi-scale model and used for updating the ALP method (see Section 2.2.3).

In real blast scenario, different members usually undergo structural damage. In this paper, blast pressure was only applied on the selected columns, since the main focus is put on developing an approach for improving the ALP method. Although it is completely possible (and in some cases is absolutely necessary) to consider the blast effects on several beams, columns and wall, the above-mentioned simplification is rational for the aim of this study. The scaled distances were considered in a way to cause the major effects on the columns. However, in real blast scenario, other structural components, i.e., beams and slabs, are more or less affected. Also, this simplification can be reasonably ac-

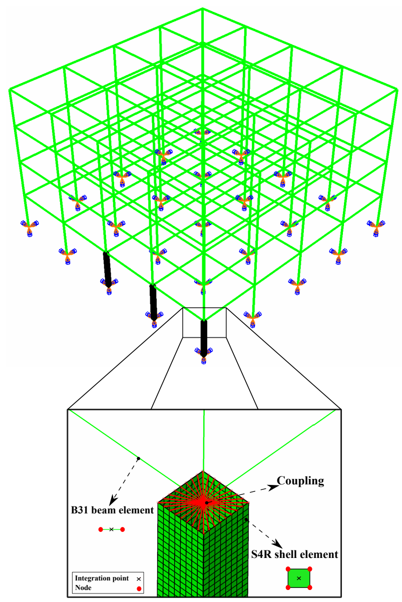


Figure 6: Multi-scale finite element model for blast-induced progressive collapse analysis.

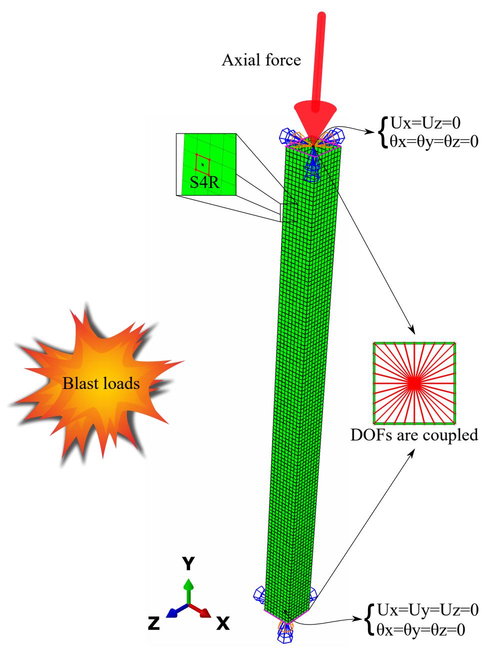


Figure 7: Details of equivalent single column model.

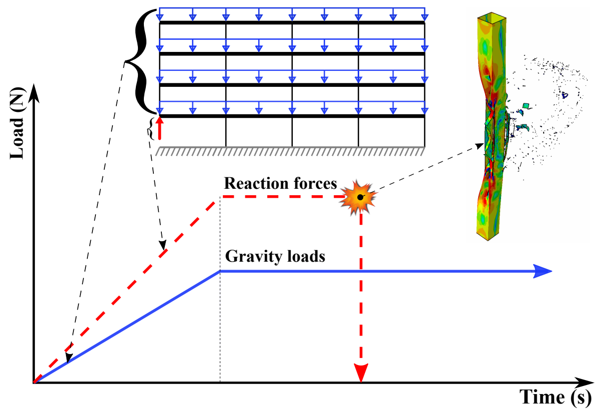


Figure 8: Loading regime for force-based dynamic column removal.

ceptable, because beams and slab are usually more protected, e.g., by cladding and non-structural elements, compared with column at ground level. Moreover, blast-induced uplift forces are neglected in the numerical study. Scaled distances and the locations of blast sources are considered in a way to minimize these effects. Ignoring the slab and 3D effects is common in numerical analysis of structures under extreme loading conditions [48, 49, 50], especially if the focus is on comparing the different scenarios [13] or developing a new method [51, 18]. However, it should be noted that for the design purpose, such effects should be checked in both progressive collapse [13] and blast analysis [52].

2.2.3. Framework for updating the ALP (modified ALP method)

Based on the results of the threat-dependent blast analysis on the equivalent single column (it was first shown that this simple substructure model gives an excellent estimate of maximum response in blast loaded multi-scale multi-story models), a simple and robust method to include threat-related parameters, i.e., damage level and CRT (or damage applying time (DAT)), is proposed. To this end, the dynamic column removal technique firstly suggested by Kim and Kim [41] was adopted and used as basis for further studies. Figure 8 graphically shows this force-based dynamic column removal method. Instead of sudden and complete column removal that is used in code-based ALP method, the new proposed method calibrates the dynamic response to include damage levels and CRT. Damage situation and CRT/DAT are obtained from blast analysis of equivalent column. Then, the forces representing the removed column were modified to update the maximum column removal response based on the blast level. More details and discussions, as well as a numerical example are available in Section 5.

2.3. Performed numerical analyses

Three groups of analyses were performed on model structures. The first group consists of threat-independent dynamic column removal scenarios that were performed on both 4-story and 8-story model structures, for the four defined initial damage scenarios, as depicted in Figure 1. The latter set of simulations was devoted to blast-induced progressive collapse. Blast analyses (BA) were conducted on Scenarios 1 and 2 of 4-story frame (as depicted by dotted line in Figure 2), for 8 blast scenarios on both multi-scale model and equivalent single column model, as shown in Table 2. Finally, based on the results of the aforementioned studies, a framework for including threat-dependent parameters in ALP method is suggested, the proposed method is numerically checked and verified for several different blast loading scenarios on 4-story model (Scenario 1 is checked). A summary of performed numerical simulations is listed in Table 2.

Table 2: Details of the performed threat-independent and threat-dependent numerical simulations.

Simulation set	Model	Scenario	CRT/DAT	Charge weight	Standoff distance
Dynamic column removal analysis	4-story, 8-story	1-4	10 ms	Threat-independent	Threat-independent
BA of multi-scale model	4-story	1-2	Threat-dependent	100, 225 and 450 kg	1, 1.5, 2 and 2.5 m
BA of equivalent column	"4-story"	1-2	Threat-dependent	100, 225 and 450 kg	1, 1.5, 2 and 2.5 m
modified ALP	4-story	1	Obtained from BA	"Threat-independent"	"Threat-independent"

3. Verification of the finite element models

Due to lack of suitable full scale dynamic collapse tests, researchers usually opt for the results of quasi-static tests for partial validation of FE models [4]. This approach has been repeatedly adopted for validation of FEMs under collapse scenarios [13, 10]. However, in this study, the results of the blast loaded steel column is used for verification of blast loading and associated structural response. The FE modeling details used for verification are described in this section and should not be mixed or confused with the modeling details and analysis techniques that described in the Section 2, for core of the paper (Section 4).

Experimental results reported by Nassr et al. were used for validation of the numerical study on a blast loaded structure [53]. A W150×24 section 2.413 m long under a 270 kN vertical load was used in the test (Test 3C1 in [53]). The column was subjected to the blast of 150 kg of ANFO (Ammonium Nitrate and

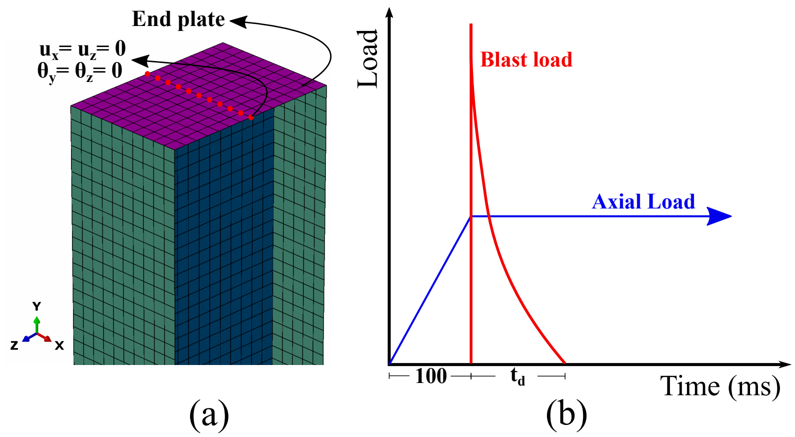


Figure 9: Numerical model adopted for verification of blast loaded column; (a) boundary conditions and (b) loading regime.

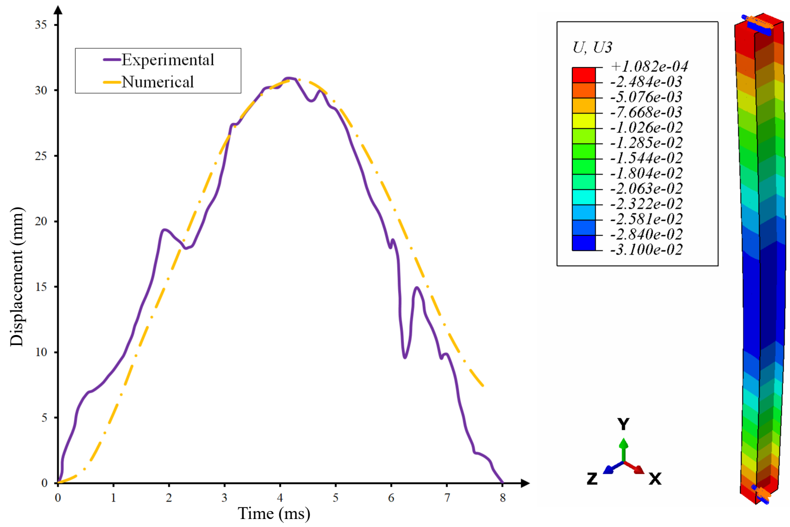
Fuel Oil) at a standoff distance of 9 m. The steel material adopted in the test has a density of 7850 kg/m^3 and its yield strength, elastic modulus, Poisson's ratio are 470 MPa, 210 GPa and 0.3, respectively.

The column was numerically modeled with pinned ends to ensure the setup used in the experimental setup. The adopted technique is descried in Figure 9(a). The recorded values in the experimental test for the reflective pressure and positive phase duration were 1560 kPa and 6.2 ms, respectively. These parameters are employed in the numerical validation to define the blast loads. The loading schemes for axial and blast loads are presented in Figure 9(b). The column was simulated using S4R shell elements and mesh size is sufficiently fine to guarantee the accuracy of obtained FE results.

The mid-span displacement time-history is used for comparison. Figure 10(a) compares that the mid-span lateral displacement time-history in numerical and experimental models. The first 100 milliseconds are not included in this figure for better presentation. A good agreement between experimental and numerical results, specially in the term of maximum displacement, is observed. Figure 10(b) shows peak displacement contour in the column. There is a negligible difference between contour and diagram. This difference is due to different output saving schemes (time intervals) adopted for the time-histories and contours in numerical simulation.

4. Results and discussion

Both threat-independent and threat-dependent results are reported and discussed. In the former, the effects of damage locations (in plan and height of buildings) and size of the structure are assessed, in the latter, main focus is put on the structural response under direct blast loads. Column removal points' displacements are adopted for comparison. CRP is here the top joint (node) in



(a) (b)

Figure 10: Validation of numerical model; (a) Displacement time-history at mid-span as obtained from the developed FE analysis and the experiments reported in [53] and (b) displacement contour at the peak.

the selected column, i.e., the column that is completely removed (in code-based ALP) or only damaged (in blast analysis and modified ALP). A methodology is suggested to include threat-dependent parameters in threat-independent ALP analysis and numerical examples are presented to validate the proposed framework. The terms "response" and "displacement" are used to refer the "vertical displacement in CRP", unless otherwise specified.

4.1. Threat-independent dynamic column removal

The time-histories of CRPs' vertical displacements are shown in Figure 11(a) and Figure 11(b) for 4- and 8-story model structures. After sudden column loss, the models are subjected to vibrations. Among the 4 column removal scenarios, the maximum displacement is observed in Scenarios 3, which refers to a corner column removal in model structures mid-height, for both 4-story and 8-story models. Comparing the column removal in plan (corner column loss versus middle column loss), Scenarios 1 and 3 (corner column removal) show more progressive collapse potential. These observations can be explained based on active ALPs; when a middle column or a column in ground level story is removed, the affected area is larger than corner column removal or column loss at the higher level. Therefore, more ALPs are activated for load distribution [13, 54]. Referring to 8-story frame, less difference between different scenarios was recorded, suggesting a more robust structure [13]. The obtained results also confirm that progressive collapse risk decreases as building's height increase, as recently found in a parametric way by De Biagi [55]. In taller and larger

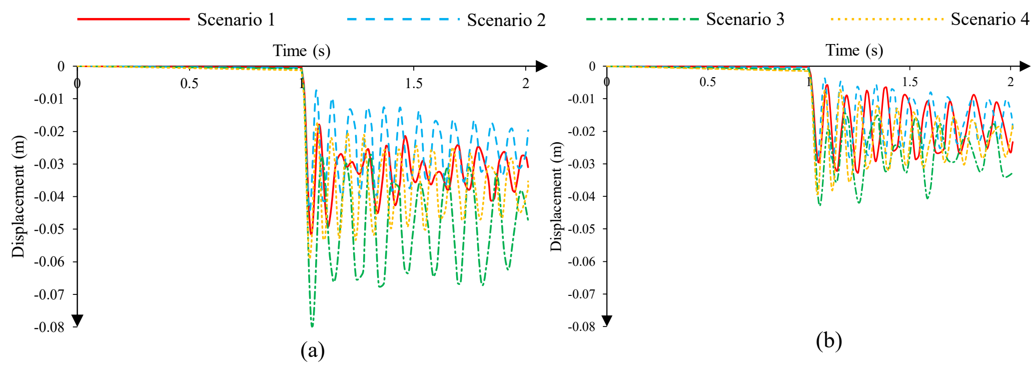


Figure 11: Time-histories of vertical displacement in CRPs for Scenarios 1-4; (a) 4-story model structure and (b) 8-story model structure.

structures, more redundancy and ALPs are available and stronger arches can develop. Moreover, in a tall moment-resisting frames, the size of some members is determined on the basis of drift criteria, therefore, more reserve strength is available to resist abnormal events. A deep comprehensive discussion on influences of initial damage location and building height on progressive collapse potential is presented in Kiakojouri et al. [13].

4.2. Threat-dependent blast analysis

The obtained numerical results emphasis the importance of charge weight and standoff distance on the structural response under blast load, as expected. Figure 12 compares displacement time-histories when the corner column is subjected to 225 kg and 450 kg TNT as explosive charge. With the increase in charge weight, or the decrease of standoff distance, the maximum response is increased meaningfully. While maximum displacements for 1 meter standoff distance (subjected to 225 kg explosive charge) are 51 mm and 55 mm for multi-scale model and equivalent column model, respectively, the recorded values for 2 meter standoff distance are only 3.5 mm and 3 mm for multi-scale model and equivalent column, respectively. These records emphasis the importance of the correct estimation of standoff distance for design of blast loaded structures, as well as non-structural protection to increase the standoff distance.

The maximum displacements in different blast scenarios for both multi-scale model and equivalent single column are listed and compared in Table 3. The results reveal that the maximum vertical displacements in blast loaded models can be larger or smaller than the ones obtained with the code-based ALP method reported in Section 4.1. For the decreased response, observation can be explained by the fact that the column is not completely removed in blast analysis, therefore, it has residual strength that leads to smaller vertical displacement compared with code-based ALP, in which complete and sudden column removal is applied.

The results of numerous experimental and numerical studies revealed that

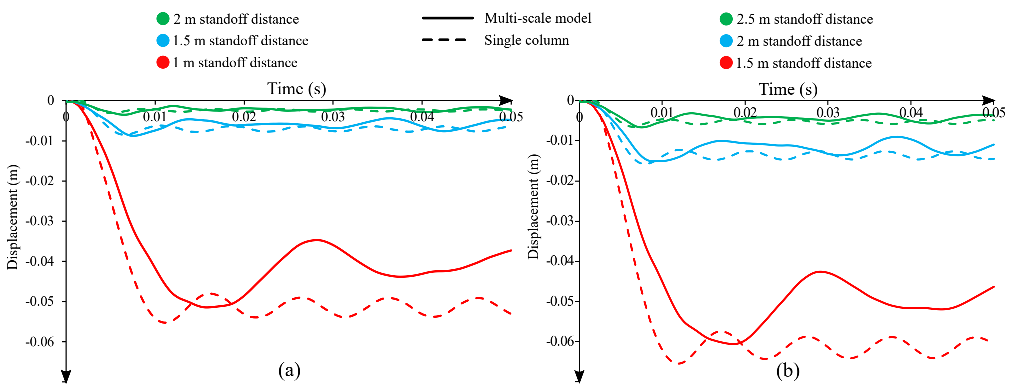


Figure 12: Time-histories of vertical displacement in CRPs for Scenarios 1 at different standoff distances; (a) under 225 kg charge and (b) under 450 kg charge.

steel columns exhibit large ductility and significant plastic deformation without complete tearing, rupture and separation when subjected to blast loads [56, 57, 16, 58]. Even for close-range and contact explosions, complete rupture is not inevitable [59, 60]. Same results are also obtained in the current numerical study. The deformed shapes (cross-sections) of blast loaded corner column subjected to different blast scenarios are shown in Figure 13. The damaged steel column maintains its connectivity to the main structure for a wide range of blast scenarios. Such a column can transfer blast loads to the main structure when it deforms horizontally, that is why the deformations in some threat-dependent cases are larger than in code-based ALP, even though in the latter the column is removed completely.

Table 3: Summary of blast analyses on multi-scale model and equivalent column, displacements at CRP in millimeter.

	Scenario 1		Scenario 2	
Scenario	Multi-scale model	Single column	Multi-scale model	Single column
100 kg @ 1 m	5.6	4.8	6.1	5.9
100 kg @ 1.5 m	1.9	1.6	2.4	2.2
225 kg @ 1 m	51	55	48	64
225 kg @ 1.5 m	8.7	8.5	9	10
225 kg @ 2 m	3.5	3.0	4.2	4.0
450 kg @ 1.5 m	60	65	55	76
450 kg @ 2 m	15	16	15	18
450 kg @ 2.5 m	6.6	6.5	7.1	7.8
ALP	51		44	

In code-based ALP, the overall behaviour of the structure is very important, compared with the members' local response. However, in blast loaded model, both local and global behaviours are equally important. That is why the well-

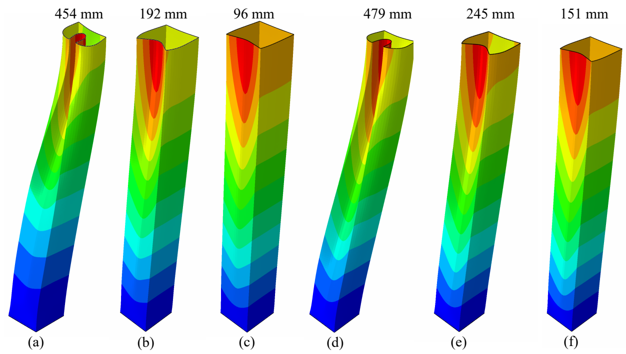


Figure 13: Cross-section of deformed shapes of blast loaded (half) column in Scenario 1; (a) 225 kg at 1 m, (b) 225 kg at 1.5 m, (c) 225 kg at 2 m, (d) 450 kg at 1.5 m, (e) 450 kg at 2 m and (f) 450 kg at 2.5 m standoff distance, quantity of maximum *horizontal* displacement is indicated for each scenario.

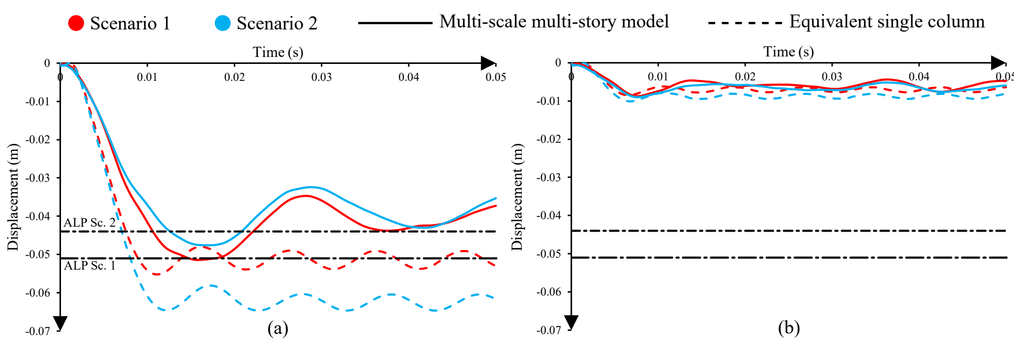


Figure 14: Comparison of Scenario 1 and Scenario 2; (a) 225 kg explosive and 1 meter standoff distance and (b) 225 kg explosive and 1.5 meter standoff distance.

known effects regarding the damage locations may not necessarily be observed here, i.e., middle column damage can lead to larger displacement compared with corner column damage. While more effective alternate load paths are available for the structure in middle column damage scenario, from member-level point of view, this scenario is more critical, because, larger axial force is applied to middle column compared to the corner column. It should be noticed that the same cross-section was used for all columns in ground level due to member classification strategy, therefore, more classification-induced over-strength exists for the corner column, and this over-strength affects the local response of blast loaded column that can lead to, e.g., smaller horizontal and vertical displacements. Time-history of Scenarios 1 and 2 are compared in the Figure 14 for two different blast scenarios. For equivalent single column models, Scenario 2 always leads to larger displacement, while, for multi-scale multi-story model the results are scenario-based, as shown in Table 3.

Comparing different blast loaded models, i.e., multi-scale system and equiv-

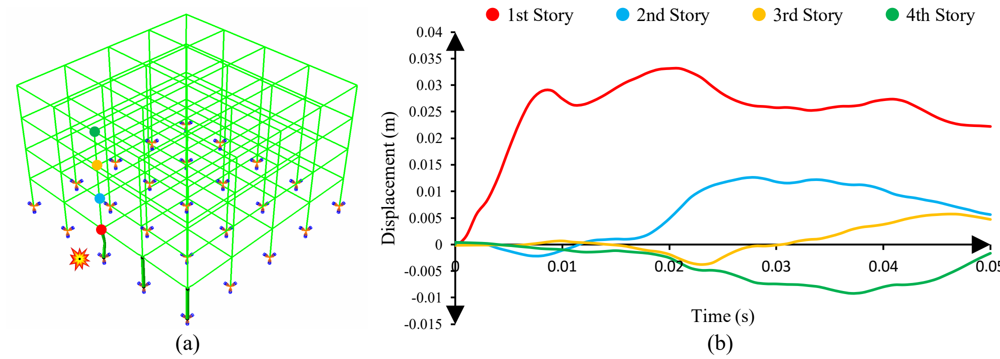


Figure 15: Horizontal displacement in Scenario 2 under 225 Kg charge and 1 meter standoff distance; (a) deformed shape and (b) time-histories of horizontal displacement at different heights.

alent single column, a good agreement in term of maximum displacement is observed, specially for smaller blast loads (see Table 3) with an average difference of about 10%. Therefore, for the specific purpose of the current study, the equivalent single column can effectively be used instead of multi-scale multi-story model. For multi-story buildings with several stories (say more than 3) the above-described boundary conditions for the equivalent single column can be safely used and it is not necessary to include the story effects, e.g., by spring or other more advanced techniques, because these simple fixed-slider model can predict the vertical displacement with a very good accuracy. However, for very short buildings, or for those buildings with very special configurations, more consideration may be necessary. This simplification leads to a huge computational advantage, and it is the base for developing a new ALP method, i.e., modified ALP.

Finally, the focus is put on the *horizontal* behaviour of blast loaded frame. The study of horizontal response of blast loaded models reveals a noteworthy behavior, as shown in Figure 15. While for CRP, the maximum horizontal displacement is always developed concordant to blast wave propagation, i.e., positive axis in Figure 15, for upper stories, the maximum displacement can occur in opposite direction, as observed for the fourth story in Figure 15. Similar results are also obtained for other blast scenarios. This observation can be understood by considering unbalanced state of damaged structures subjected to inertia effects. When the structure reaches the first peak (at CRP) and returns to its original condition, the damaged column can not contribute in the system properly. Similar behaviour also reported in Tavakoli and Kiakojouri [26] for external far-field blasts.

5. A framework for updating alternate load path method (modified ALP)

It was shown in the Section 4.2 that especially when the maximum displacement is needed, an equivalent single column can be modeled and analyzed, instead of multi-story multi-span building under certain blast scenarios. However, for progressive collapse design, the overall response, as well as, forces and rotations in the members are usually required. To this end, a simple and effective framework is suggested for including the triggering event characteristics in threat-independent multi-story FE model.

For assigning the blast effects obtained in equivalent single column model to multi-story building, two important variables should be defined; (i) the damage level, and (ii) the time required for this damage to develop. For the former, a relationship between the displacement obtained by code-based ALP and the displacement obtained in blast analysis can be considered as basis for the suggested method. This displacements are associated to the forces in the damaged column. In this regard, the following formula is proposed:

$$F_{Modified} = \left(1 - \frac{\Delta_{Blast}}{\Delta_{ALP}}\right)\ F_{C,intact} \tag{4}$$

where $F_{Modified}$ is modified reaction force (residual force) after column removal, Δ_{Blast} is maximum vertical displacement in equivalent single column model under specific blast scenario, Δ_{ALP} is maximum vertical displacement obtained by code-based ALP, and $F_{C,\ intact}$ is column force in the intact model.

As discussed in Section 4.2, the vertical displacement in blast loaded column can be larger or smaller compared with code-based ALP method. Equation 4 translates these displacements to associated forces. When Δ_{Blast} is smaller than Δ_{ALP}, the modification factor $(1 - \Delta_{Blast}/\Delta_{ALP})$ is positive and smaller than one, therefore, instead of complete column removal illustrated in force-based method of Figure 7, a modified force-based damage scenario can be applied, as shown in Figure 16(a). On the other hand, when Δ_{Blast} is larger than Δ_{ALP}, modification factor $(1 - \Delta_{Blast}/\Delta_{ALP})$ is negative. That means, not only the reaction force should be reduced to zero (as performed in code-based ALP for column removal, see Figure 7), but also an additional vertical downward force should be applied to the model to increase the displacement computed with the ALP method. This case is graphically illustrated in Figure 16(b).

It should be noted that several simplified approaches, including a work by the Authors [61], based on single degree-of-freedom models using elastic-plastic compliance law, are reported for assessing the progressive collapse response without complex and detailed numerical modelling. Therefore, Δ_{ALP} can be approximated using these methods in satisfying accuracy. That means, for estimating the blast-induce progressive collapse response (with the limitations and assumptions that discussed in Section 2.2.2) of multi-span multi-story moment-resisting frames, detailed FEA on a single target column would be sufficient.

To model the blast-induced damage, the time required for developing such a damage is also required. In other word, instead of “column removal time (CRT)”

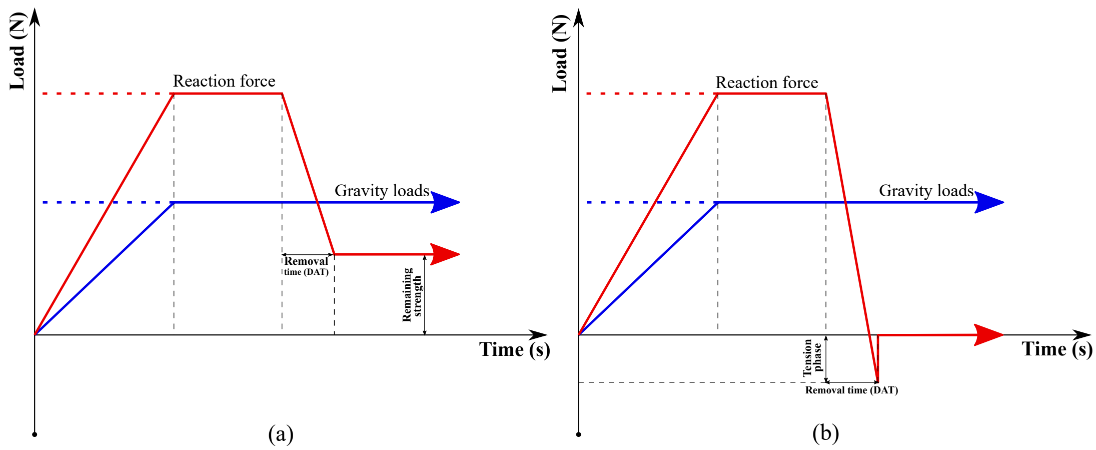


Figure 16: Proposed loading regime for modified ALP; (a) when response of blast-loaded structure is lower than code-based ALP and (b) when response of blast-loaded structure is higher than code-based ALP.

that is used in code-based ALP, "damage applying time (DAT)" should be used. To this end, the times required to develop the maximum plastic deformation in the multi-scale model and the equivalent single column are compared in the Table 4. The differences are in an acceptable range, specially because these time periods are very short, and as discussed by [12, 13], for such short intervals, maximum displacements do not change considerably. That means for DAT, the time periods obtained in blast analysis on single column can be used as suggested in Figure 16(a) and (b).

In Figure 16, the linearly increasing initial part is adopted to eliminate the vibration induced by the sudden application of loads. However, the alternative straight path (the dotted line in the figure) can also be used, as highlighted in Tavakoli and Kiakojouri [51], for sufficient time period before column removal (or applying damage) the two methods leads to same results. A flowchart depicting the proposed framework is shown in Figure 17.

Table 4: Comparison of the required time (in millisecond) for developing the maximum response in multi-scale model and equivalent column under different blast scenarios.

	100 kg		225 kg			450 kg		
Model	1 m	1.5 m	1 m	1.5 m	2 m	1.5 m	2 m	2.5 m
Multi-scale model	6.5	5.9	17	7.7	6.5	19	8.6	7.4
Single column	5.6	4.8	13	6.8	5.4	23	8.3	6.8

Based on the proposed method, Scenario 1 under different blast loads is checked and the results are compared with code-based ALP and also blast analysis on multi-story model. Time-histories of displacement of selected cases are shown in Figure 18. While considerable differences between code-based threat-

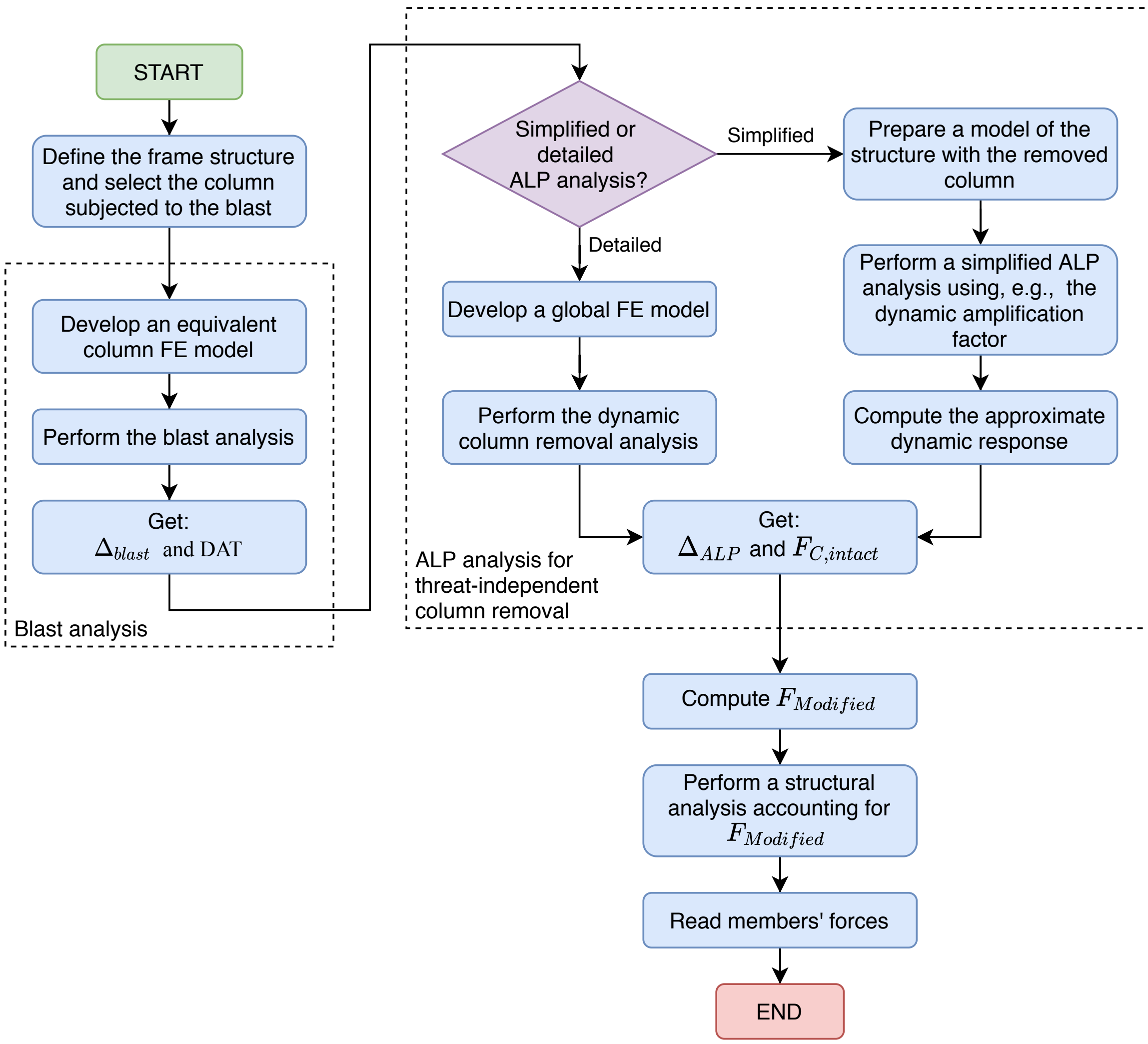


Figure 17: Proposed threat-dependent framework for evaluating progressive collapse.

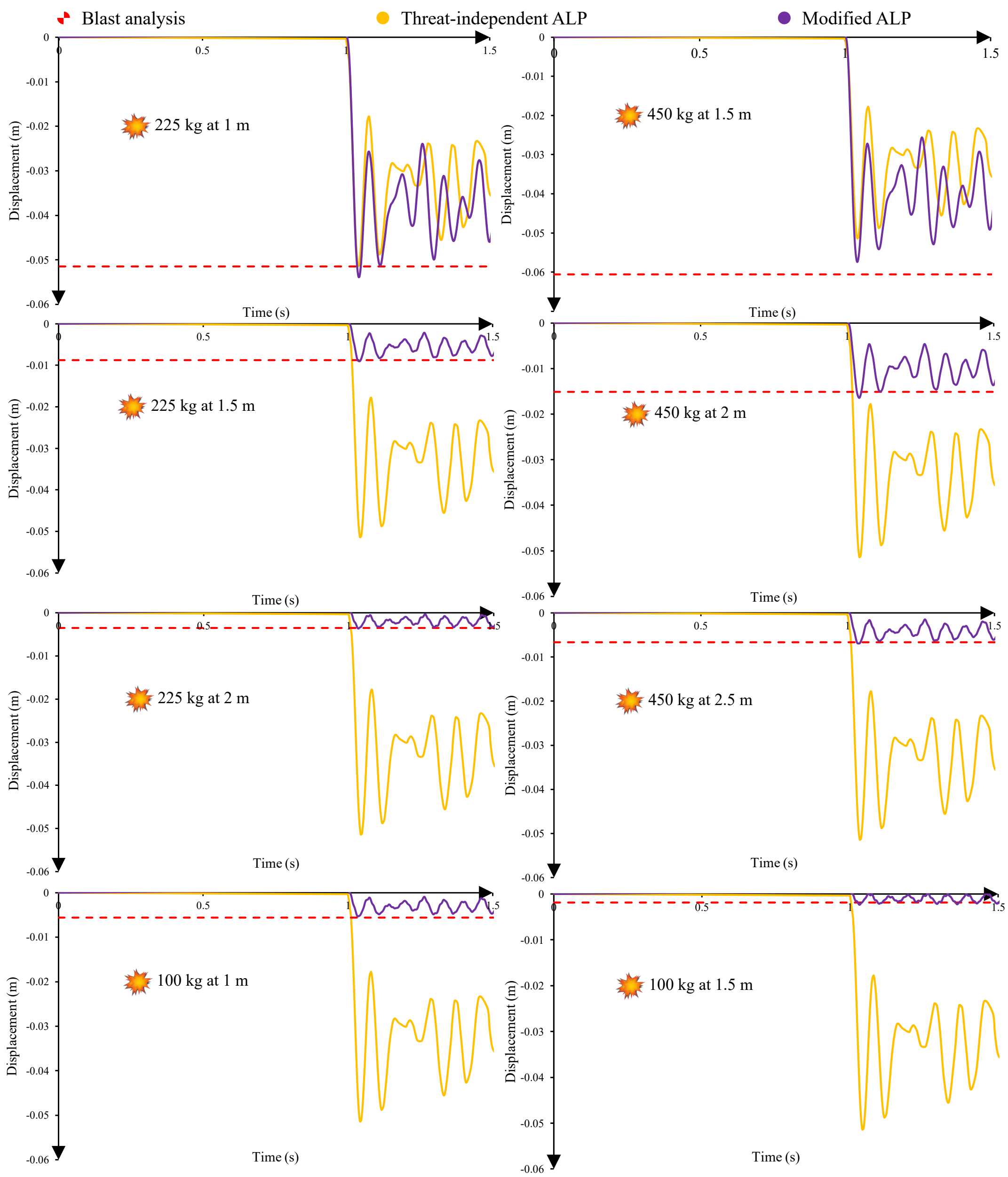


Figure 18: Time-histories of vertical displacement at CRPs for Scenarios 1 under different standoff distances and charges.

independent ALP and threat-dependent blast analysis are observed, the proposed modified ALP method can predict the blast effects on progressive response with a good accuracy, specially in term of the maximum vertical displacement. Thus, leading to the possibility of assessing the robustness of the structure.

It should be noted that the modified ALP should be compared with the code-based ALP, and not to detailed blast-loaded models. The overall response of blast-loaded structure cannot be easily modeled in a simple framework, like what suggested here, i.e., due to the present of horizontal time-variant blast-induced forces that are not considered in the proposed approach. However, this framework can effectively update the code-based ALP and prevent its under- and over-estimations, based on the blast loads level. In other word, the proposed framework, considering the possible threats, i.e., according to importance of the building, exposure and the level of non-structural protection, modifies the ALP response, which can lead to both economical and computational advantages.

6. Conclusions

In this study, sample model structures have been first analyzed in the code-based threat-independent dynamic column removal scenarios. Then, the focus has been put on the blast-induced progressive collapse and the threat-dependent response has been monitored and discussed. Finally, based on the results of blast simulations, suggestions have been made to update the code-based ALP method to capture the features of initial failure more specifically. The major findings are summarized in the following points:

- the results of threat-dependent blast-induced progressive collapse analysis reveal that the progressive collapse response in threat-dependent methodology can be meaningfully different, compared with the one obtained by code-based ALP. The maximum response of the former can be larger or smaller referring to code-based ALP method (threat-independent dynamic column removal), based on the blast level, i.e., charge weight and standoff distance;

- the influences of damage location on progressive collapse potential are not necessarily similar in code-based ALP and in blast-induced progressive collapse. While in the former the response is purely a function of the redundancy of the system and available alternate paths, in the latter member-level behaviour is also important. In this regard, member-oriented over-strength can have a decisive effects on the overall response;

- a framework for including the threat-dependent damage property in code-based ALP is proposed. It was shown that in steel moment-resisting frames, an equivalent single column can be used instead of multi-span multi-story building for specific range of blast loading, to extract the threat-related parameters, i.e., CRT/DAT and damage level. These parameters can be used to update the ALP method. The comparison of the numerical results of proposed method with code-based ALP reveals that modified ALP can predict the dynamic progressive collapse displacement of blast loaded models with very good accuracy. Therefore, the proposed framework can be effectively applied in progressive collapse assessment of framed systems in order to avoid under- and overestimation of structural responses, that lead to more rational and economical design.

The methodology presented here can be easily extended in order to consider other triggering events, e.g., impact. Moreover, a detailed FE model can be applied to include connections and slab effects. Considering the numerous possibilities for charge weight, standoff distance, and blast source location, a more elaborated blast simulation may be necessary for some blast scenarios. In this regard, instead of single equivalent column that used in the current study, substructure model (equivalent substructure) can be adopted and the proposed load schemes should be applied to several damaged members.

Declaration of Competing Interest

The authors declare no conflict of interest.

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