Journal of Constructional Steel Research 153 (2019) 71–84

Contents lists available at ScienceDirect

Journal of Constructional Steel Research

A new method for progressive collapse analysis of steel frames

Shu-Chao Lin [a,b,c], Bo Yang [a,b], Shao-Bo Kang [a,b], Shao-Qian Xu [a,b,*]

[a] Key Laboratory of New Technology for Construction of Cities in Mountain Area (Chongqing University), Ministry of Education, Chongqing 400045, China
[b] School of Civil Engineering, Chongqing University, Chongqing 400045, China
[c] Post-doctoral Mobile Station of Civil Engineering, School of Civil Engineering, Chongqing University, Chongqing 400045, China

ARTICLE INFO

Article history:
Received 12 March 2018
Received in revised form 7 August 2018
Accepted 30 September 2018
Available online xxxx

Keywords:
Steel frame
Progressive collapse
Blast load
Non-zero initial condition
Initial damage
Finite element analysis

ABSTRACT

As a massive explosion happens inside a building, a number of structural members (columns, beams, slabs and so on) are damaged or failed, along side with non-zero initial conditions (displacement, velocity, acceleration and so on), and the progressive collapse of the building structures is most likely to occur. However, limited research works about blast load effect of structural members inside a building can be found. In view of this, based on the substructure model, a new method for progressive collapse analysis of steel frames under blast load is proposed. First, the massive explosion scenario inside a building is introduced. Then, the substructure model within effective areas of blast influence is established. After that, the calculation method of non-zero initial conditions and initial damage for structural members is given, and finally the specific steps of the proposed method are described. By way of example of a steel frame with 5 stories in height, 4 bays in the longitudinal direction, 3 bays in the transverse direction, direct simulation method, alternative load path method and proposed method are all employed to simulate the progressive collapse process, respectively. Through the example analyses, it is shown that blast load effect of structural members cannot be ignored on the ground floor, and it can be ignored on the other floors by the effect of the reinforced concrete slab. The non-zero initial conditions and initial damage of structural members can be predicted well by the substructure model, and the proposed method is also reliable and accurate.

1. Introduction

In recent years, with the soaring urban population and the tightening supplies of land resources, land prices are rising sharply, which provides a vast development prospect and an infinite market potential for mushroom development of super-tall buildings. High-rise civil buildings and large-scale mixed-use buildings are constantly emerging, and developing towards large-scale, complexity and diversification. In the long service life, the building structures may encounter different types of accidental disasters, such as an extremely rare earthquake, vehicle or aircraft impact, gas explosion, bomb attack, human error, fire and so on. It brings indelible potential risk to the building structures.

With the constant progress of the society and the sustainable development of the economy, natural and man-made disasters that occur suddenly with great destruction emerge in endlessly. As a special type of disaster, the explosion enters into our daily life. Taking into account the peculiarity of blast load, the dynamic response characteristics and failure modes of the structural members become very complicated, especially in the case of a massive explosion inside the building. A number of structural members (columns, beams, slabs and so on) are damaged or failed, along side with non-zero initial conditions (displacement, velocity, acceleration and so on). It has become an important and hot topic, and the researchers and engineers make greater efforts in this connection.

Traditional alternative load path method [1,2] is the evaluation method of building structures to resist progressive collapse by analyzing the dynamic response of building structures in case of removal of one or more columns within a certain time. The source of the accidental load is not involved. Therefore, it is not suitable to analyze the problem of progressive collapse of building structures under accidental load, since the accidental load effect of the structural members cannot be ignored [3–6]. Gerasimidis et al. [7] studied the collapse mechanisms and the dynamic response characteristics of the whole building structure by properly introducing the partial damage index in the building structural systems. Finally, a new partial distributed damage method (PDDM) was presented for the steel frames in this reference. Ettouney et al. [8] detailed the importance of investigating global effects when evaluating the potential for progressive collapse of building structures. The simulation results showed that the necessity for considering the global response of a damaged structure became apparent following the evaluation of the overall stability of these systems. Sideri et al. [9] examined the effect of damage distribution on the progressive collapse potential

* School of Civil Engineering, Chongqing University, Chongqing 400045, China.
E-mail address: xsq1964@cqu.edu.cn (S.-Q. Xu).

https://doi.org/10.1016/j.jcsr.2018.09.029
0143-974X/

of steel buildings when subjected to an external blast detonation scenario. Luccioni et al. [10] investigated progressive collapse process of building structures under blast load by using the TNT-air-structure refined three-dimensional finite element model. The simulated results were compared with that of the collapsed building. It was shown that direct simulation method could simulate the progressive collapse process of building structures under blast load accurately and reliably. Hao et al. [11] carried out a finite element analysis of reinforced concrete frame under the blast load specified in the design guideline TM5-1300 [12]. The accuracy and the applicability of the design guidelines GSA [1] and DoD [2] were evaluated. Finally, a new method of progressive collapse analysis was proposed, namely DYN method. The complicated finite element model was used by the direct simulation method and DYN method, and it had a high requirement for computer configuration. It was necessary to have a detailed understanding of detonation physics process, propagation of shock wave, tedious numerical simulation technique and complicated fluid-solid coupling mechanism. Shi et al. [13] predicted non-zero initial condition (displacement and velocity) by using the equivalent single-degree-of-freedom model of the column and initial damage by using the pressure-impulse curve. It was confirmed that the simulated result considering the initial damage and non-zero initial conditions was more close to results by using direct simulation method. Finally, a new method for progressive collapse analysis of reinforced concrete frames under blast loading was put forward. However, the spatial effect of the frame was not considered, especially the effect of the slab. The column damage was equated with material damage. It was only applicable to the explosion outside the building.

On this basis, a new method for progressive collapse analysis of steel frames under blast load is proposed. The non-zero initial conditions and initial damage of the structural members are accurately predicted by using the substructure model, which considers the effect of the slab. By way of example of a 5-story steel frame, direct simulation method, alternative load path method, and proposed method are used to simulate the progressive collapse process, respectively. The proposed method is checked and verified.

2. Finite element model of steel frame

2.1. Prototype building

The building selected in this study is a seismically designed steel frame [14], and it has 5 stories in height, 4 bays in the longitudinal direction with a span length 5 m, 3 bays in the transverse direction with a span length 5 m. Fig. 1 shows the aerial view of the steel frame. Fig. 2 shows the typical floor plan of the steel frame. The height of first floor is 6 m, and the height of other floors is 4 m.

The column and beam of the steel frame adopt fabricated H sections, which are welded by three steel plates. The column section is H400 × 300 × 12 × 14 (H section depth × flange width × web thickness × flange thickness), as shown in Fig. 3(a), and the beam section is H300 × 250 × 10 × 12, as shown in Fig. 3(b).

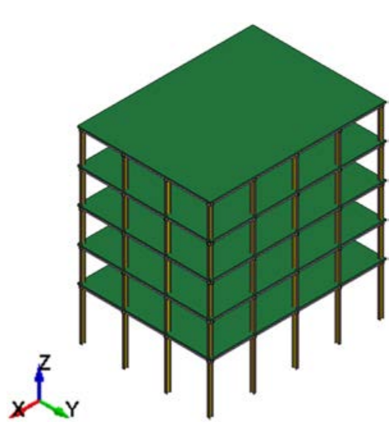


Fig. 1. Aerial view of a steel frame.

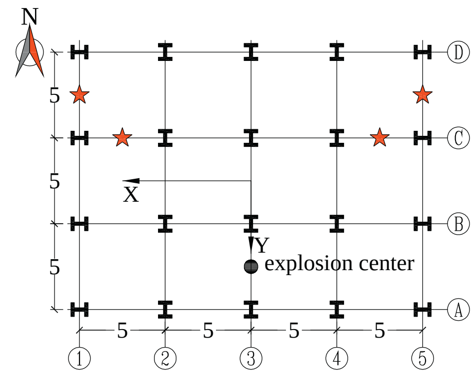


Fig. 2. Typical floor plan of steel frame.

The building used for numerical modelling in this study has 20 columns, of which 14 are exterior columns, and the rest are interior columns. The beam-exterior column joints shown in Fig. 4(a) use moment-resisting connections, and the beam-interior column joints shown in Fig. 4(b) use fin plate (simple shear) connections in gravity frames.

The thickness of reinforced concrete slab is 140 mm, and the concrete cover thickness is 30 mm. There are double-layer two-way steel reinforcements with a diameter of 8 mm and spacing of 0.15 m located at the top and bottom of the concrete slab. The filling wall, which serves as enclosures and separators, does not sustain any vertical load, and its weight is borne by the column and beam. It is assumed that the column foot is fixed on the basement.

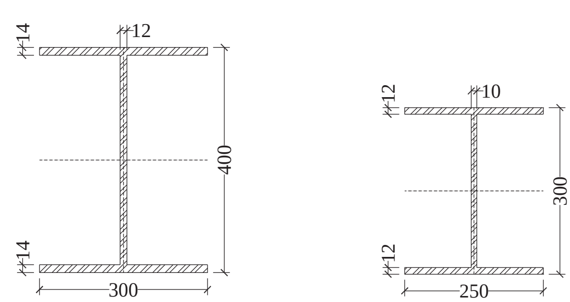
Steel frame - reinforced concrete slab is mainly composed of concrete and steel. The basic parameters of the concrete material are shown in Table 1. Structural steel Q345 is selected for beam and column, and its basic parameters are shown in Table 2. Steel HRB335 is selected for rebar, and its basic parameters are shown in Table 3.

For dynamic analysis purposes, the following vertical load combination specified in the design guideline GSA2013 [1] is applied to the structure in this study:

$$1.00 \times DL + 0.25 \times LL \tag{1}$$

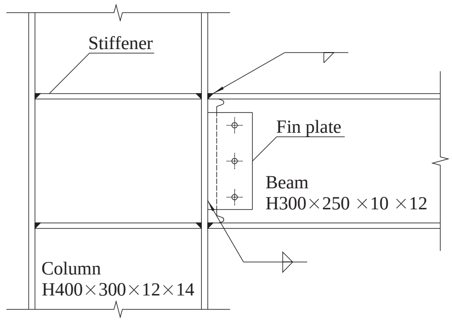
where DL is dead load; LL is live load.

In addition to the self-weight of the steel frame, the dead load of floor plug topping, the dead load of the ceiling and other dead load are considered in this study. It is assumed that the extra dead load is 2.5 kN/m^2, and the live load is 6.0 kN/m^2. The filling wall is applied to the beam in the form of uniformly distributed load, and the value is 18.0 kN/m.

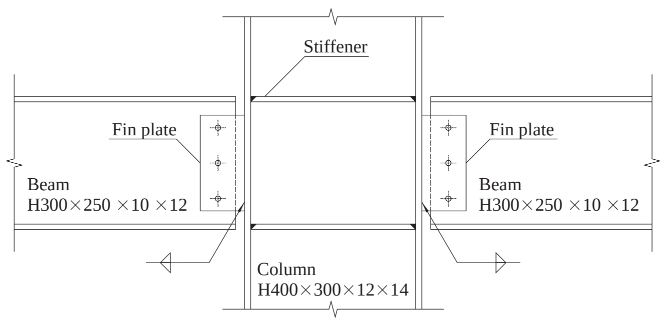


(a) H400 × 300 × 12 × 14 (column) (b) H300 × 250 × 10 × 12 (beam)

Fig. 3. The section of column and beam.



(a) Beam-exterior column joints



(b) Beam-interior column joints

Fig. 4. Beam-column joints.

2.2. Finite element model

The above 5-story steel frame is taken as a computational model. The surface burst explosion is adopted with a certain quantity of hemispherical explosive charge, such as TNT. In order to reasonably solve the problem of the relative size of fluid-structure elements, stability propagation of shock wave and improve the computational efficiency of finite element model, the parameters for material model behavior, element characteristic and fluid-solid coupling mechanism should be validated carefully. A refined three-dimensional finite element model is shown in Fig. 5, and the details of the model are given in Table 4.

The material model of *Mat_Plastic_Kinematic (*Mat_003) [15] is used for the steel, which is suited to model isotropic, kinematic, or a combination of both hardening plasticity with the option of including rate effects. The material failure can be considered accurately. It is suitable for a one-dimensional element like rebar and two-dimensional element like columns and beams.

The material model of *Mat_CSCM_Concrete (*Mat_159) [15] developed by the U.S. Federal Highway Administration is used for the concrete. It is a cap model with a smooth intersection between shear yield surface and hardening cap. The high strain rate effect can be described accurately, especially for $\dot{\varepsilon} > 100$, and the strain rate effect can also be limited. It is suitable for three-dimensional element like slab.

In this study, the material model of *MAT_NULL (*MAT_009) [15] is used to model the air, which satisfies the state equation of *EOS_LINEAR_POLYNOMIAL, and its parameters are listed in Table 5.

Table 1
Mechanical properties of concrete material.

f_{ck}	E	ν	ρ	d
30	3×10^{10}	0.2	2500	19

Note: f_{ck}, the compressive strength of concrete, MPa; *E*, elastic modulus, N/m^2; ν, Poisson's ratio; ρ, mass density, kg/m^3; *d*, maximum aggregate size, mm.

Table 2
Mechanical properties of Q345.

f_y	E	E_P	ν	ρ_s	ε_f	C	P
345	2×10^{11}	E/1000	0.3	7830	0.1	40	5

Note: f_y, the yield strength of steel, MPa; *E*, elastic modulus, N/m^2; E_P, tangent modulus, N/m^2; ν, Poisson's ratio; ρ_s, mass density, kg/m^3; ε_f, failure strain; C and P, Cowper and Symonds model parameters.

Table 3
Mechanical properties of HRB335.

f_y	E	E_P	ν	ρ_s	ε_f	C	P
300	2×10^{11}	E/1000	0.3	7830	0.1	40	5

High explosives are typically modelled by using the material model of *MAT_HIGH_EXPLOSIVE_BURN (*MAT_008) [15], with state equation of *EOS_JWL, and the parameters are given in Table 6.

In the simulation, the ALE algorithm is used for air and TNT, and the lagrange algorithm is used for the steel frame. The fluid-structure interaction mechanism is built between ALE materials and lagrange materials by using the function of *Constrained_Lagrange_In_Solid, and its calculation adopts the penalty function in the software LS-DYNA.

In order to avoid the distortion of element shape of reinforced concrete slab caused by the progressive collapse of the steel frame, advanced erosion technology is required. The erosion is not a physical phenomenon, but a basic numerical simulation technique. It can be used to solve the problems related to mesh distortion caused by the general motion of lagrange mesh. The function of *Mat_Add_Erosion in the software LS-DYNA provides the similar function [15,16], but it is independent of material constitutive model. Each parameter must be determined by trial and error to achieve the best numerical simulation results. In this study, the maximum principal strain and shear strain are adopted, and the values must be carefully selected. According to the empirical value given by relevant literature, the initial value of maximum principal strain is 0.010 [17], while the initial value of shear strain is 0.800 [18,19]. Through trial and error, the failure criterion parameters are adjusted gradually. Finally, the maximum principal strain value of 0.008 is adopted, while the shear strain value of 0.640 is used in this study.

It is easy to cause local damage of reinforced concrete slab under a massive explosion inside a building. In order to accurately simulate the structural performance of reinforced concrete slab, a separate finite element model is adopted to treat concrete and rebar as different elements. The reinforcing bars are embedded in the concrete by using the function of *Constrained_Lagrange_In_Solid, thus they can work together. Only in this way can the performance of reinforced concrete structural members be simulated accurately.

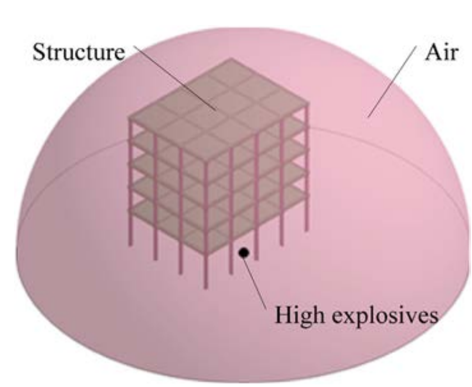


Fig. 5. Finite element model of TNT-Air-Structure.

Table 4
Finite element model of steel frame.

	Material model	Equation of state	Element type	Element algorithm	Element length/mm	Element number
Frame	*Mat_003	–	Shell163	1	50–100	82,710
Slab	*Mat_159	–	Solid164	2	60–120	159,300
Rebar	*Mat_003	–	Beam161	3	150	345,015
TNT	*Mat_008	*EOS_JWL	Solid164	4	50	4000
Air	*Mat_009	*EOS_Gruneisen	Solid164	4	200–300	1,000,000

Note: 1, Belytschko-Wong-Chiang; 2, constant stress solid element; 3, truss (resultant); 4, 1 point ALE multi-material element.

In order to ensure that the steel-concrete composite beam works together, a certain number of shear connectors are set up between the steel beam and the reinforced concrete slab. The shear connection is represented as thin steel plates, which embedded into the reinforced concrete slab, and its element nodes are shared with top flange of steel beam and reinforced concrete slab, thus the case of composite beams with complete shear connection can be simulated.

In order to ensure the precise occurrence of the fluid-solid coupling mechanism, the normal direction of shell element should point to explosion center, but the relation between the shape, direction and locations of shell element and explosion center is uncertain. In order to simplify the problem, the shell element is considered as a directed plane, and its normal direction can be directed to explosion center so as to determine whether to change the normal direction of shell element of column or beam.

3. Finite element model validation

In order to verify the rationality of finite element model and the reliability of finite element analysis method, the test specimen [20] of steel frame inside the bunker with the explosive charge is analyzed with the highly non-linear finite element analysis software LS-DYNA. The column section is H260 × 260 × 10 × 17.5, and the beam section is H220 × 110 × 5.9 × 9.2. The mechanical properties of steel frame material are shown in Table 7.

Fig. 6 shows the comparison of failure modes obtained from test and numerical model. The analytical results demonstrate that the failure length of column web is about 0.6 m. However, the plastic deformation of column flange predicted by numerical simulation is slightly larger. This may be due to the fact that the shape of cylindrical explosive charge is not fully consistent with that of experimental test. But as a whole, the failure mode of steel frame obtained from finite element analysis is very close to that of the test. In this way, the parameters of TNT-Air-Structure field and the rationality of numerical model are verified.

4. Massive explosion scenario inside a building

When a massive explosion happens inside a building, the limited space can amplify the intensity of shock wave. A number of structural members can be damaged or failed, along side with non-zero initial conditions, and the progressive collapse of building structures is most likely to occur. Nevertheless, limited research works have been conducted about this scenario. Therefore, the massive explosion scenario inside a building is selected in this study, and its basic principles are as follows:

There are many reasons for triggering the massive explosion inside a building, for instance, the attack of targets of military importance shown in Fig. 7(a), the terrorist bombings shown in Fig. 7(b), inadequate management of hazardous chemical materials shown in Fig. 7(c), and combustible and explosive gas leakage shown in Fig. 7 (d). All kinds of explosions can be represented by a certain quantity of hemispherical explosive charge, such as TNT.

The explosive location should have the limited space, and the shock wave will go through a series of complicated changes, such as reflection, refraction, diffraction, convergence and superposition. Thus, the distribution of fluid field is very complicated.

For the steel frame in this study, there may be more than one explosion scenarios, in other words, several combinations of explosion position and explosion quantity. In order to fully get understanding of the progressive collapse resistance of the steel frame, it is necessary to carry out progressive collapse analysis for each combination.

Table 5
Modelling parameters of air.

Parameter	Value
Density	1.292 kg/m^3
Initial internal energy per unit reference volume	0.25 MJ/m^3
Ratio of specific heats	1.4

Table 6
Modelling parameters of TNT.

Parameter	Value
Density	1640 kg/m^3
Detonation velocity	6930 m/s
Chapman-Jouget pressure	21 GPa
A	374 GPa
B	3.74 GPa
R_1	4.15
R_2	0.9
ω	0.35
Detonation energy per unit volume	6×10^3 MJ/m^3

5. Comparison of the existing analytical methods about progressive collapse

For a specified combination of explosion position and explosion quantity, direct simulation method and alternative load path method are used to carry out the progressive collapse analyses of the steel frame, respectively. As a result, the effect of blast load inside a building on progressive collapse of the steel frame is investigated. At the same time, the analytical results can be used to validate the new method proposed in this study.

5.1. Direct simulation method

The whole process including detonation progress, shock wave propagation process, the interaction between shock wave and building structures, dynamic response and damage of the structural members under blast load, and the progressive collapse process of the building structures is simulated by using the analysis software LS-DYNA. The

Table 7
Mechanical properties of steel frame material.

	f_y	f_u	Agt
Beam flange, IPE220	345	464	28.0
Beam web, IPE220	353	463	30.4
Column web, HEB260	420	529	27.0
Column flange, HEB260	407	539	27.0
Bolt, M16, class 10.9	965	1080	12.0

Note: f_y, the yield strength of steel, N/mm^2; f_u, the ultimate strength of steel, N/mm^2; Agt, total elongation at maximum stress, %.

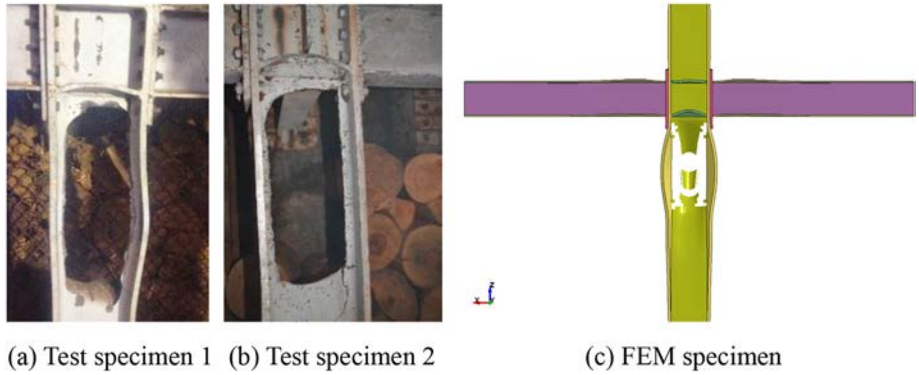


Fig. 6. Comparison of the failure modes.

combination of dead load and live load defined in design guideline GSA2013 [1] is applied to building structures slowly in the range of 0.0 to 0.2 s and goes on for 0.2 s. After a massive explosion happens inside a building, shock wave propagates to the slab quickly. With the increase of the overpressure, the reinforced concrete slab will sustain tremendous thrust. The shear failure occurs on some slabs or beam ends. At the same time, there is an obvious plastic deformation of H-shaped steel column on the ground floor under the action of self-weight of the structures, load from overpressure of beam and slab and lateral overpressure load. Local buckling occurs on some flanges of the columns, and shear failure occurs at the tops of some columns. When a massive explosion happens inside a building, a number of the structural members on the ground floor are damaged or failed. The level of damage of the structural members on the other floors is not obvious by the restraint effect of the reinforced concrete slab. The progressive collapse process of the steel frame is shown in Fig. 8.

5.2. Alternative load path method

The alternative load path method is completely independent of the accidental event. It assumes that columns or beams are removed completely, and ignores the particularity of the accidental load. The Column A3 on the ground floor are removed in Section 5.2.1 in accordance with the GSA [1] and DoD [2], while the Column A2, A3, A4, B2, B3, B4, C2, C4 on the ground floor are removed in Section 5.2.2 on the basis of the results of direct simulation method, and the non-zero initial conditions and initial damage of other structural members on the ground floor are ignored. Dynamic response analysis in the following sections is conducted on the remaining structure with the software LS-DYNA.

5.2.1. One column-removal scenario

In this study, the Column A3 is removed purposely in according to the position of explosive charge in the direct simulation method and the dynamic response process of the remaining structure is studied in detail. The combination of dead load and live load defined in the guideline GSA2013 [1] is applied to building structures slowly in the range of 0.0 to 0.2 s and keeps constant for 0.2 s. After the specified Column A3 on the ground floor is removed suddenly, the internal force of the steel frame will be redistributed to seek the new load transfer path. The adjacent Columns A2, A4 and B3 will carry a large portion of the load previously carried by the removed Column A3. There are no new structural members to meet failure criteria. Finally, the residual structure reaches a new stable state and does not collapse. The dynamic response process of the steel frame after the removal of Column A3 is shown in Fig. 9.

5.2.2. Eight column-removal scenario

The combination of dead load and live load defined in the guideline GSA2013 [1] is applied to building structures slowly in the range of 0.0 to 0.2 s and goes on for 0.2 s. After the specified columns on the ground floor are removed suddenly, the slab load and the filling wall load shall be passed to the column through steel-concrete composite beams. Firstly, the steel beam fractures under catenary action, and then shear failure occurs on the reinforced concrete slabs, leaving only the tensile membrane action from the slab. Local

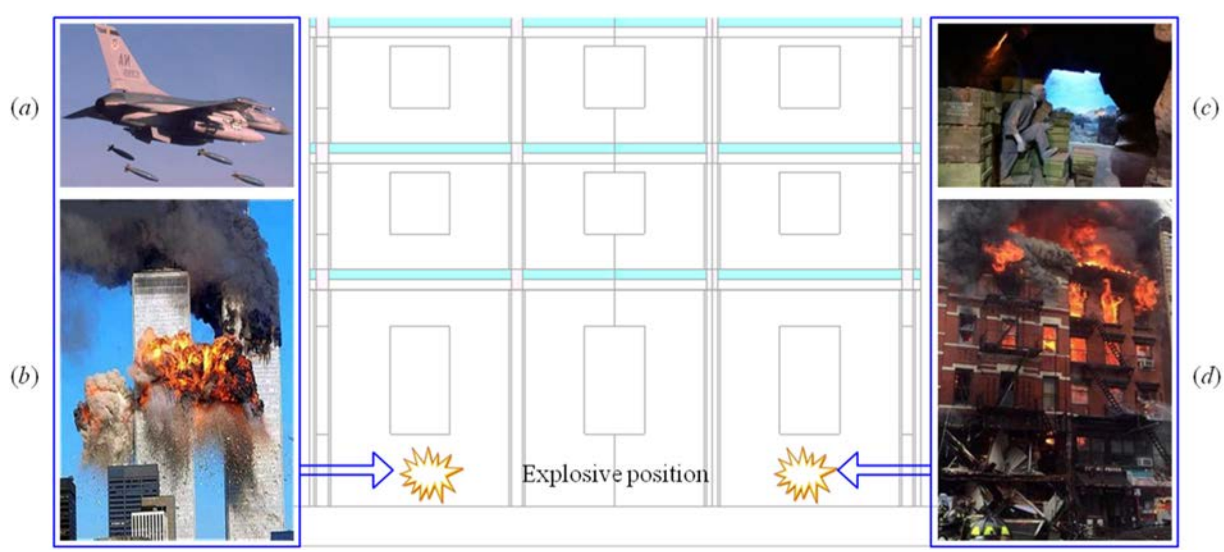


Fig. 7. The massive explosion scenario inside a building.

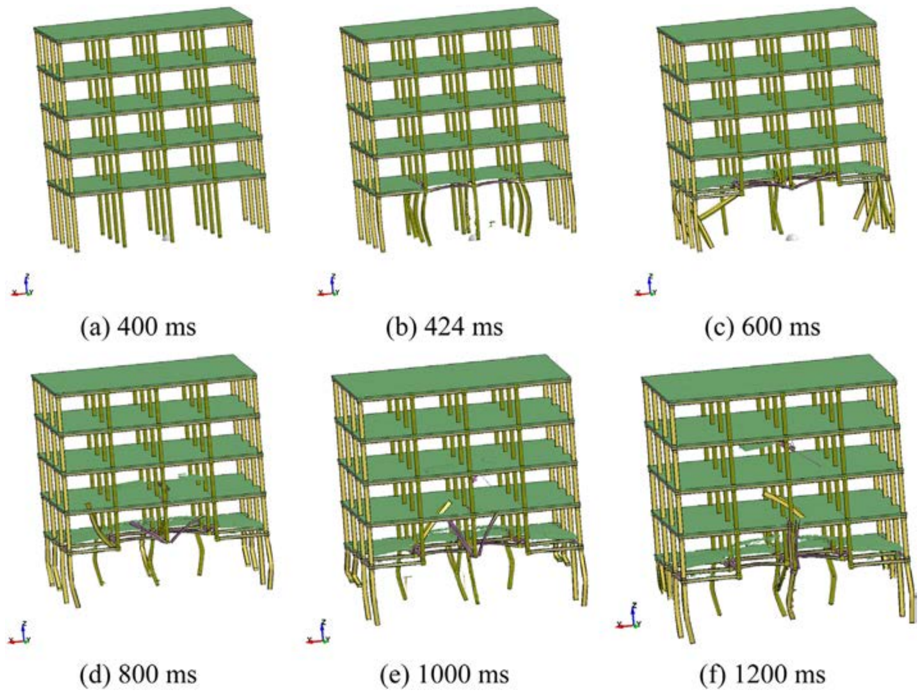


Fig. 8. Progressive collapse process of steel frame (Direct simulation method).

buckling of some columns is observed, for instance, Column C3. The compression-shear failure occurs at the tops of some column, such as Column A1 and so on. The progressive collapse process of the steel frame is shown in Fig. 10.

By comparing the Fig. 9 with Fig. 10, it can be found that the remaining structure will not collapse under one column-removal scenario while the remaining structure will collapse under eight column-removal scenario, this indicates that the existing design of buildings to resist progressive collapse may not be conservative in the case of massive explosion inside a building.

The progressive collapse processes of steel frame obtained from direct simulation method and eight column-removal scenario are shown in Figs. 8 and 10. The failure modes of composite beams, failure modes of reinforced concrete slabs and time sequence of progressive collapse process of the steel frame are obviously different. It should be noted that the non-zero initial conditions and initial damage of the structural members are very significant [3,4], but the blast load effect is not taken into account under eight column-removal scenario.

Therefore, a new method is urgently needed to improve the traditional alternative load path method to be more accurate and reliable.

6. Substructure method

Based on the alternative load path method in the design guidelines GSA [1] and DoD [2], the substructure method is proposed by using the substructure model which can accurately predict blast load effect. Through the comparison of dynamic responses between direct simulation method and substructure method, the reliability of non-zero initial conditions and initial damage can be verified.

6.1. Proposed substructure method

The substructure method takes into account three-dimensional effect of the frame and the slab, and combines with the strain rate effect of building material. It is suitable for the conditions of massive explosion inside a building. Because of the simple algorithm of beam or shell element for the structural members, the substructure model is of less computation and is not high for computer configuration requirements, and therefore it can be used to predict the non-zero initial conditions and initial damage of the structural members rapidly and accurately, thus providing basic data for the validation of substructure method, and the proposed method is also checked and verified.

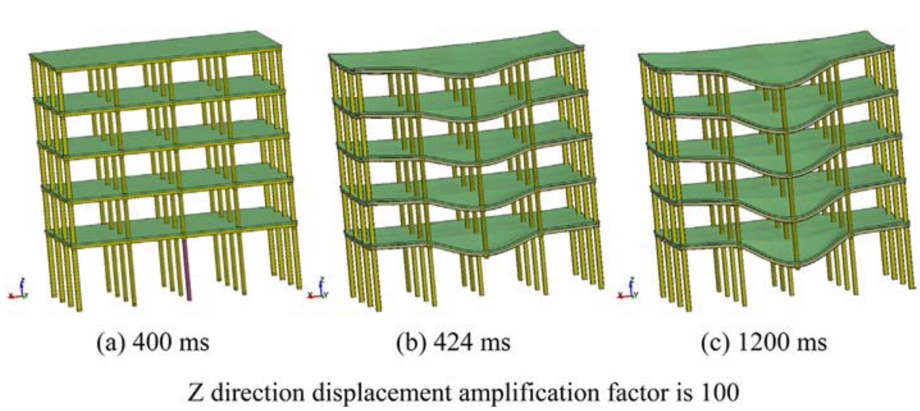


Fig. 9. Dynamic response process of steel frame (One column-removal scenario).

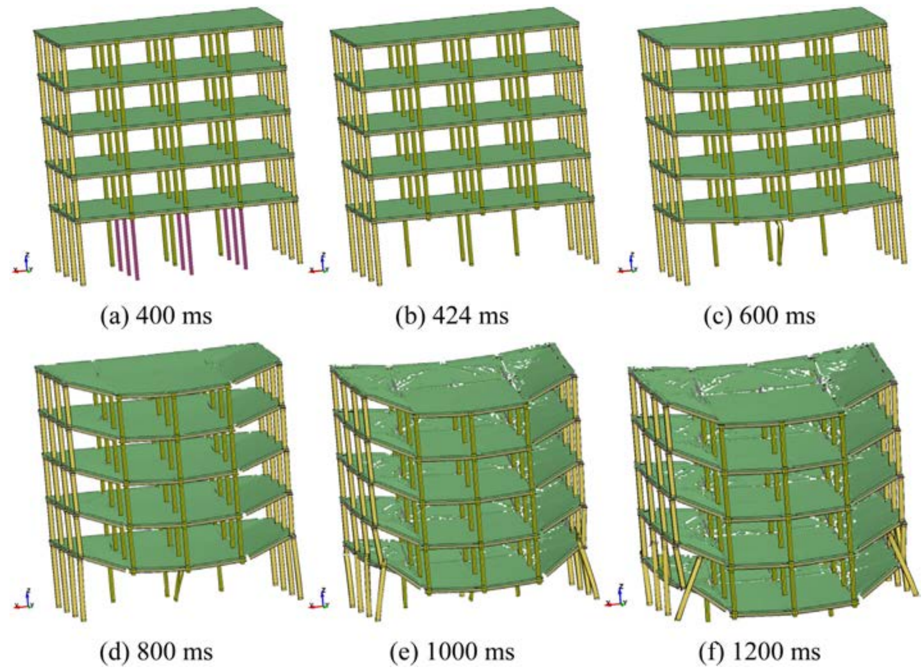


Fig. 10. Progressive collapse process of steel frame (Eight column-removal scenario).

6.1.1. Blast load effect

The effective areas of blast influence are confined to the explosive floor and its adjacent floors due to limited energy of a certain amount of TNT and the effect of the slab. Therefore the blast load effect of structural members can be ignored on the other floors. In the present paper, the non-zero initial conditions and initial damage of the structural members are analyzed by using the substructure model shown in Fig. 11.

The existing commercial finite element analysis software, for instance, ANSYS, ABAQUS and MSC.Marc, has the functions of beam or shell section modelling so as to accurately simulate the mechanical performance of the structural members. The software LS-DYNA has a similar function like beam and shell section modelling [15]. It should be noticed that the numerical modelling can also be implemented by other commercial software like ANSYS and ABAQUS.

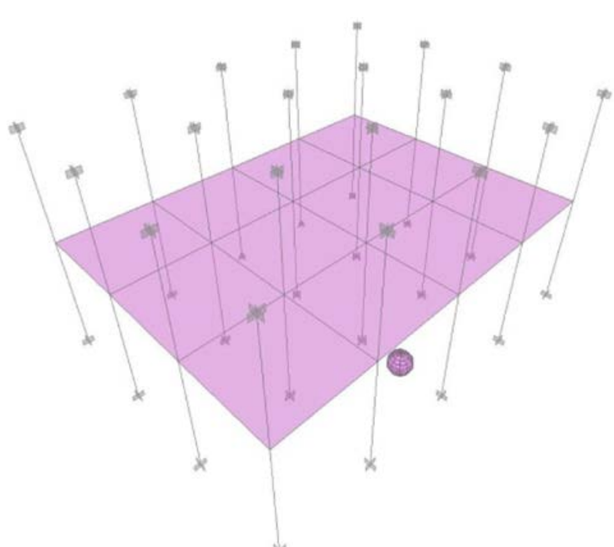


Fig. 11. The substructure model.

6.1.1.1. Non-zero initial conditions. As a massive explosion happens inside a building, it releases a lot of energy in a very short period of time, and then generates a lot of high temperature and high pressure gas expanding enormously, and then forms a shock wave. When the overpressure drops to zero within effective areas of blast influence, it is regarded as the beginning of progressive collapse analysis of building structures. The displacement, velocity, and acceleration of the structural members are defined as the non-zero initial conditions at this time.

6.1.1.2. Initial damage. In order to evaluate the damage of the structural members under blast load, the damage criterion should be reasonably defined. A failure criterion based on material damage is adopted in this study. The damage effect of the structural members within effective areas of blast influence is defined as the initial damage at the beginning of progressive collapse analysis of building structures.

As far as concrete is concerned, initial damage [21] is mainly composed of two parts: tensile damage and compression damage.

$$D_M = \alpha_t d_t + \alpha_c d_c \tag{2}$$

where d_t and d_c are damage variables in tension and compression, respectively; α_t and α_c are weighting coefficients in tension and compression, respectively.

The damage variables d_t and d_c are defined as a function of the variable κ:

$$\begin{aligned} d_t &= 1 - \frac{\kappa_0(1-A_t)}{\kappa} - \frac{A_t}{\exp[B_t(\kappa-\kappa_0)]} \\ d_c &= 1 - \frac{\kappa_0(1-A_c)}{\kappa} - \frac{A_c}{\exp[B_c(\kappa-\kappa_0)]} \end{aligned} \tag{3}$$

where $\kappa_0 = f_t / E_0$; f_t is tensile strength of concrete; E_0 is initial elastic modulus; κ is the maximum value of the equivalent strain ever reached during the loading history; the factors A_t and B_t are the constants in the description of tensile strength, which are fitted from the response of the material under uniaxial tension, and the factors A_c and B_c are the constants in the description of compressive strength, which are fitted from the response of the material under uniaxial compression.

The weighting coefficients α_t and α_c are defined as.

$$\begin{cases} \alpha_t = \sum_{i=1}^{3} \left(\frac{\langle \varepsilon_i^t \rangle \langle \varepsilon_i \rangle_+}{\overline{\varepsilon}^2} \right)^{\beta} \\ \alpha_c = \sum_{i=1}^{3} \left(\frac{\langle \varepsilon_i^c \rangle \langle \varepsilon_i \rangle_+}{\overline{\varepsilon}^2} \right)^{\beta} \\ \overline{\varepsilon} = \sqrt{\sum_{i=1}^{3} \left(\langle \varepsilon_i \rangle_+ \right)^2} \end{cases} \tag{4}$$

where $<\cdot>_+$ is the Macauley bracket; ε_i are the principal strains; the variables ε_i^t and ε_i^c are the principal strains corresponding to positive and negative stresses, and the factor β is fitted from the response of the material under shear.

As far as steel is concerned, the initial damage [22] is defined by the following expression:

$$D = \sum \frac{\Delta\varepsilon}{\varepsilon^f} \tag{5}$$

where $\Delta\varepsilon$ is increment of equivalent plastic strain, which occurs during an integration cycle; ε^f is equivalent strain to fracture.

The strain at fracture is given by.

$$\varepsilon^f = [D_1 + D_2 \exp(D_3\sigma^*)] \times [1 + D_4 \ln(\dot{\varepsilon}^*)] \times [1 + D_5 T^*] \tag{6}$$

where σ^* is the ratio of pressure divided by effective stress; $\dot{\varepsilon}^*$ is ratio of effective plastic strain rate divided by quasi-static threshold rate; $T^* = (T - T_{room}) / (T_{melt} - T_{room})$, T_{room} is room temperature, T_{melt} is melt temperature; D_i are the factors in Johnson-Cook material model.

The whole process of structural members from beginning deformation to failure is taken as a process of gradual deterioration of building materials under blast load. As far as structural members are concerned, the changes of physical properties, such as elastic modulus and strength, are the assessment of material degradation.

6.1.2. Blast load

The incident wave interacts with the structural members, such as columns, beams, slabs and so on. The overpressure of reflected wave increases immediately and forms reflective high pressure zone near the blast side. The shock wave continues to propagate bypassing the structural members or passing through the structural members. To simplify the problem, the explosion load on the blast side is determined by using an empirical formula in accordance with the earlier researches of the author. The blast load is directly applied to the substructure model, and the load time difference of the structural members within effective areas of blast influence can be ignored.

In terms of the blast side, the best-fitted relation of the normal peak pressure of reflected wave to the peak pressure of incident wave is derived as.

$$p_r/p_i = 2.930(p_i)^{0.244} \quad 0.5 \le p_i \le 100\text{MPa} \tag{7}$$

where p_r is the normal peak pressure of reflected wave; p_i is the peak pressure of incident wave.

Considering the non-regular reflection caused by incident angle, the peak pressure of reflected wave at any position on blast side can be written as follow.

$$p_r(d)/p_r(0) = \begin{cases} 1-0.0012[p_r(0)]^{0.7147} d^{1.4296} & r/r_0 = 3.0 \\ 1-0.0012[p_r(0)]^{0.8781} d^{1.3910} & r/r_0 = 6.0 \\ 1-0.0006[p_r(0)]^{0.46} d^2 & p_r(0) \le 10\text{MPa} \end{cases} \tag{8}$$

where d is the in-plane distance; $p_r(0)$ is normal peak pressure of reflected wave; r is distance between slab and explosion center; r_0 is radius of explosive charge.

Many research results showed that the pressure-time variation of the reflected wave was identical to that of the shock wave in the infinite air [13,23,24]. According to Eq. (8) and the typical time attenuation model of overpressure in the air, it is easy to get the time attenuation model of the reflected wave. Therefore, the blast load can be applied correctly to the columns, beams, slabs and other structural members.

The shock wave travels through the reinforced concrete slab and continues to propagate. The overpressure of the reformed wave reaches the maximum immediately and then decreases quickly. The downward trend of overpressure-time is relatively smooth. The maximum value of the overpressure is smaller, but the maximum value of the impulse is larger. In terms of the structural members on the other side, the blast load is applied by using Eq. (9) in the light of my own personal research.

$$\begin{cases} p_r/p_p = \left(0.2700\overline{r}^2 - 6.4955\overline{r} + 60.9487\right) \\ \times\left(0.1028\overline{t}^2 - 0.0762\overline{t} + 0.6504\right) \times \left(-0.0119\overline{f}^2 + 0.1473\overline{f} + 0.6639\right) \\ I_r/I_p = \left(-0.3250\overline{r}^2 + 7.6286\overline{r} + 7.5287\right) \\ \times\left(0.0149\overline{t}^2 - 0.0110\overline{t} + 0.1432\right) \times \left(-0.0108\overline{f}^2 + 0.1425\overline{f} + 0.6607\right) \end{cases} \tag{9}$$

where p_r and I_r are peak pressure and impulse of reflected wave, respectively; p_c and I_c are peak pressure and impulse of reformed wave behind the slab, respectively; $\overline{r} = r / r_0$; $\overline{t} = t / t_{min}$, t is slab thickness, t_{min} is minimum slab thickness, which meets the limit ratio of the span to the thickness in the Eurocode 2: Design of concrete structures [25]; $\overline{f} = f_{ck} / 10$, f_{ck} is concrete compressive strength, MPa.

6.2. Basic steps of substructure method

On the basis of alternative load path method of GSA [1] or DoD [2], the blast load effect of the structural members within effective areas of blast influence will be considered in the proposed method. In this section, the new method namely by substructure method for progressive collapse analysis of steel frames under blast load is proposed. The main process of this method will be presented step by step below, and its flow chart is shown in Fig. 12.

(1) Based on advanced numerical simulation technology, such as birth and death technique of element, large displacement technique, the fluid-solid coupling mechanism and so on, and combines with the strain rate effect of building material, the complicated finite element model is established by using the nonlinear explicit dynamic finite element analysis software LS-DYNA.
(2) A typical massive explosion scenario inside a building can be selected according to the occurrence probability of the explosion. Do repeat steps (3) to (7).
(3) The combination of dead load and live load specified in the design guidelines GSA [1] or DoD [2] is applied to building structures slowly and lasts for hundreds of milliseconds until the internal force distribution of each structural member becomes stable. The fluctuation of the total reaction force of computational model is no >5%.
(4) As far as the structural members on the blast side are concerned, according to the space attenuation model of overpressure in the free air, the incident wave parameters can be obtained, and the reflected wave parameters at any position on the slab can be reasonably predicted. The parameters of the reformed wave behind the slab can be accurately predicted based on the parameters of reflected wave on the blast side.
(5) The detailed LS-DYNA analysis is conducted on the substructure model under the action of the typical pressure-time load [26]. The displacement, velocity, and acceleration of the structural members within effective areas of blast influence can be

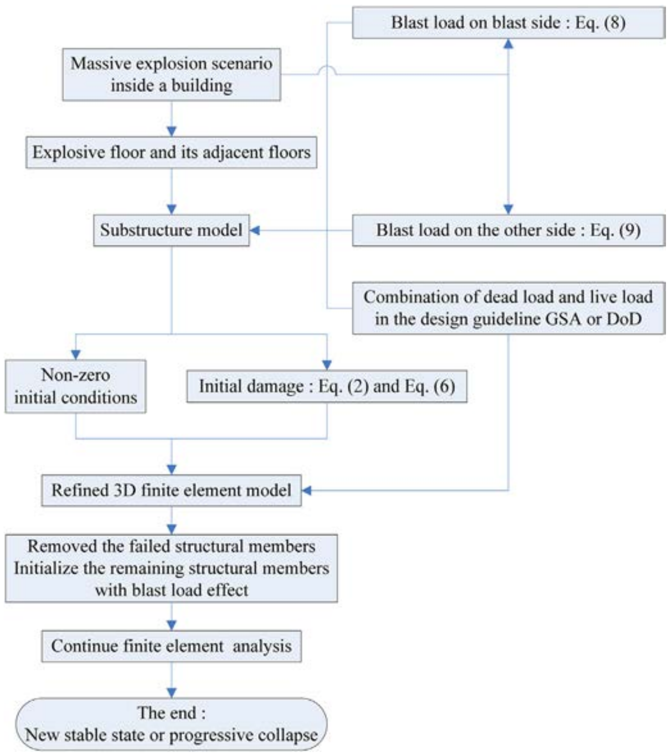


Fig. 12. Flow chart of substructure method.

obtained. The initial damage of the structural members within effective areas of blast influence can also be obtained by using the Eqs. (2)–(6).

(6) The corresponding failed structural members are removed in the three-dimensional finite element model. The remaining structural members within effective areas of blast influence are initialized with blast load effect.

(7) Continue to carry out the LS-DYNA analysis on the remaining structure. If there are new structural members to meet failure criteria, the structural members are automatically removed, and the corresponding load is redistributed to adjacent structural members until the remaining structure reaches a new stable state, or progressive collapse occurs.

6.3. Validation of substructure method

6.3.1. Comparison of failure modes

Figs. 8(c) and 13 depict the failure modes predicted by direct simulation method and substructure method, respectively. There is a slight difference in local failure between these two methods. But as a whole, the failure mode of the steel frame obtained from substructure method is close to that of the direct simulation method.

6.3.2. Comparison of blast load effect

After a massive explosion inside a building, a number of structural members are damaged or failed, along side with non-zero initial conditions. The average dynamic response of the column section is taken as the non-zero initial condition of the structural member section, and the maximum damage of the column section is taken as the initial damage of the column, so the mechanical performance of the column can be accurately evaluated in the progressive collapse process of the steel frame. Due to the boundary conditions of the column, the Z direction dynamic response can be ignored, only X and Y direction dynamic responses need to be considered. The non-zero initial conditions and initial damage of the column are given in Table 8.

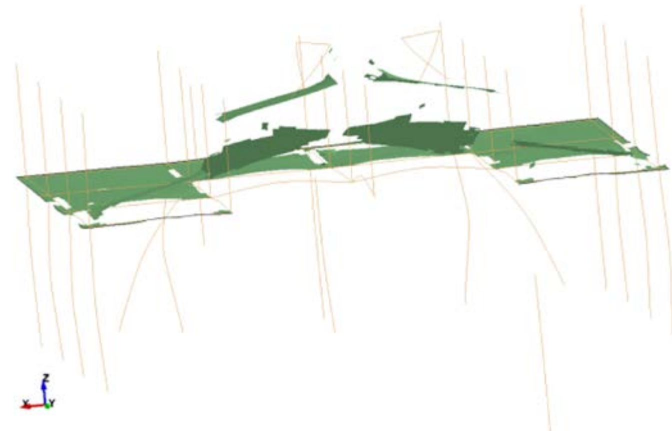


Fig. 13. Failure mode of substructure model.

After a massive explosion inside a building, the slab is subjected to not only the overpressure load, but also the deformation of the steel frame. The slab is damaged along the steel beam, and it presents typical shear failure mode. The average dynamic response of the slab block is taken as the non-zero initial condition of the slab block, and the average damage of the slab is taken as the initial damage of the slab block, so the mechanical performance of the slab can be accurately evaluated in the progressive collapse process of the steel frame. Because of the stiffness of the slab and deformation restricted by the steel frame, the in-plane dynamic response can be ignored, only the out-of-plane dynamic responses need to be considered. The non-zero initial conditions and initial damage of the slab are given in Table 9.

Tables 8 and 9 describe the blast load effect for the column and reinforced concrete slab. Whether the non-zero initial conditions or the initial damage, the maximum relative error between substructure method and direct simulation method is <5%, and thus the non-zero initial conditions and initial damage for structural members can be predicted well by the substructure model.

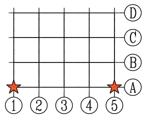
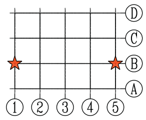
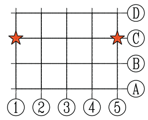
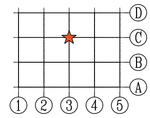
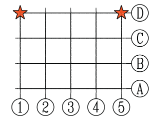
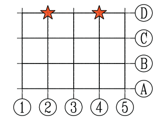
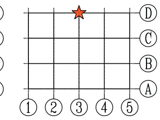
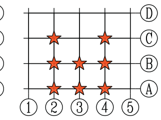
Considering the length limit of the paper, the non-zero initial conditions and initial damage of the beams will not be given in the form of a table. The maximum displacement difference along Y direction for longitudinal beams between substructure method and direct simulation method is 4.05%, and the location of the corresponding longitudinal beam is labelled by ★ in Fig. 2. The maximum displacement difference along Z direction for transverse beam between two methods is 4.88%, and the location of the corresponding transverse beam is also labelled by ★ in Fig. 2. It can be found that for longitudinal and transverse beams, the predictions obtained from direct simulation method and substructure method agree very well with each other.

In conclusion, the non-zero initial conditions and initial damage for structural members can be predicted well by the substructure model.

6.3.3. Comparison of dynamic response characteristics

After the massive explosion, the steel frame will collapse because of the limited load-carrying of remaining columns on the ground floor, and the remaining structure rotates around the Y axis over time. In order to accurately describe the dynamic responses of the remaining structure, which is similar to the rigid body, several typical reference points are selected in this study, such as reference points 1 2 and 3, and they are on the top of frame Columns D3, A3 and A1 or A5, respectively. In order to accurately describe the failure process of the bottom columns under gravity load, a reference point is selected in this study, such as reference point 4, and it is the midpoint of Column D3 on the ground floor. In order to accurately describe the release process of deformation energy of the building structure, several typical reference points are selected in this study, such as reference points 5, 6 and 7, and they are the midpoints of Columns D3, A3 and A1 or A5 on the second floor, respectively.

Table 8
Non-zero initial conditions and initial damage of column.

Item									
X-d (m)	DSM	0.121	0.122	0.037	−0.001	0.023	0.033	0.000	–
	SSM	0.119	0.120	0.037	−0.001	0.023	0.034	0.000	–
	η	−1.65%	−1.64%	0.00%	0.00%	0.00%	3.03%		
Y-d (m)	DSM	0.128	−0.109	−0.082	−0.044	−0.077	−0.045	−0.014	–
	SSM	0.133	−0.113	−0.083	−0.044	−0.079	−0.045	−0.014	–
	η	3.91%	3.67%	1.22%	0.00%	2.60%	0.00%	0.00%	
X-v (m/s)	DSM	1.426	−0.195	−3.042	0.096	0.114	2.156	0.013	–
	SSM	1.407	−0.186	−3.127	0.099	0.114	2.182	0.013	–
	η	−1.33%	−4.62%	2.79%	3.13%	0.00%	1.21%	0.00%	
Y-v (m/s)	DSM	7.804	−8.260	−6.674	−4.797	−12.567	2.023	2.672	–
	SSM	7.501	−7.883	−6.394	−4.938	−12.379	2.041	2.764	–
	η	−3.88%	−4.56%	−4.20%	2.94%	−1.50%	0.89%	3.44%	
D	DSM	0.02	0.08	0.05	0.14	0.02	0.00	0.00	1.00
	SSM	0.02	0.08	0.05	0.14	0.02	0.00	0.00	1.00
	η	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%

Note: ★ represents the location of the column. X-d, X direction displacement, m; Y-d, Y direction displacement, m; X-v, X direction velocity, m/s; Y-v, Y direction velocity, m/s. *D*, initial damage value of the column obtained by using Eq. (5); DSM, direct simulation method; SSM, substructure method; η, difference.

6.3.3.1. Dynamic response characteristics of remaining structure. The comparison of *Z*-direction displacement of reference points obtained from different methods is shown in Fig. 14. The *Z*-direction displacement of reference point 2 obtained from one column-removal scenario increases rapidly to the maximum, that is 31 mm at about 0.48 s, and then gradually fluctuates around a constant value, and finally, it tends to the constant value because of structural damping, indicating that the building structure does not collapse.

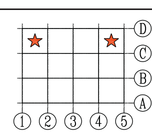
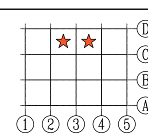
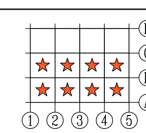
The displacement at reference points increases rapidly over time. The Z-direction displacement of reference point 1 from eight column-removal scenario is always smaller than that from direct simulation method and the Z-direction displacement of reference point 2 from eight column-removal scenario is always larger than that from a direct simulation method. There are two main reasons: Firstly, the non-zero initial conditions and initial damage of remaining column on the ground floor are ignored. Secondly, the failure time of specified column on the ground floor is not considered. The Z-direction displacement of reference point 2 from the substructure method is smaller in the range 0.4 to 0.8 s and then it is going to be larger all the time, the similar phenomenon is observed on reference point 3, but the Z-direction displacement of reference point 1 from the substructure method is always larger. There are three main reasons: Firstly, shock wave propagation process is ignored, and loading time of each structural member is not considered. Secondly, the blast load effect of the structural members on the other floors is not considered. And thirdly, the shape features of H-shape steel are not considered in detail due to the local failure of beam or column, which adopts beam element.

The comparison of Y-direction displacement of reference point 1 obtained from different methods is shown in Fig. 15. The Y-direction displacement of reference point 1 obtained from one column-removal scenario is smaller than those from other methods. This is mainly because that the massive explosion scenario inside a building is not taken into account in the existing design of buildings to resist progressive collapse.

The Y-direction displacement of reference point 1 increases rapidly. It increases constantly by using eight column-removal scenario, but it gradually decreases after it reaches maximum value by using direct simulation method. The non-zero initial conditions and initial damage is slightly overestimated by using the substructure method. The Y-direction displacement of reference point 1 from substructure method is slightly larger than that from a direct simulation method. The non-zero initial conditions and initial damage is slightly overestimated by using the substructure method. The main reason for this is that the dynamic response of the structural members on the other floors is not considered.

The comparison of *Z*-direction velocity of reference point 1 obtained from different methods is shown in Fig. 16. The Z-direction velocity of reference point 1 obtained from one column-removal scenario fluctuates more frequently with the removal of the Column A3, and finally, it goes to the zero due to the redistribution of internal force of the adjacent structural members.

Table 9
Non-zero initial conditions and initial damage of reinforced concrete slab.

Item				
Z-d (m)	DSM	0.020	0.034	–
	SSM	0.020	0.035	–
	η	0.00%	2.94%	
Z-v (m/s)	DSM	3.450	3.770	–
	SSM	3.341	3.747	–
	η	−3.16%	−0.61%	
D	DSM	0.24	0.52	1.00
	SSM	0.25	0.50	1.00
	η	4.17%	−3.85%	0.00%

Note: ★ represents the location of the slab. Z-d, Z direction displacement, m; Z-v, Z direction velocity, m/s. *D*, initial damage value of the slab obtained by using Eq. (2); DSM, direct simulation method; SSM, substructure method; η, difference.

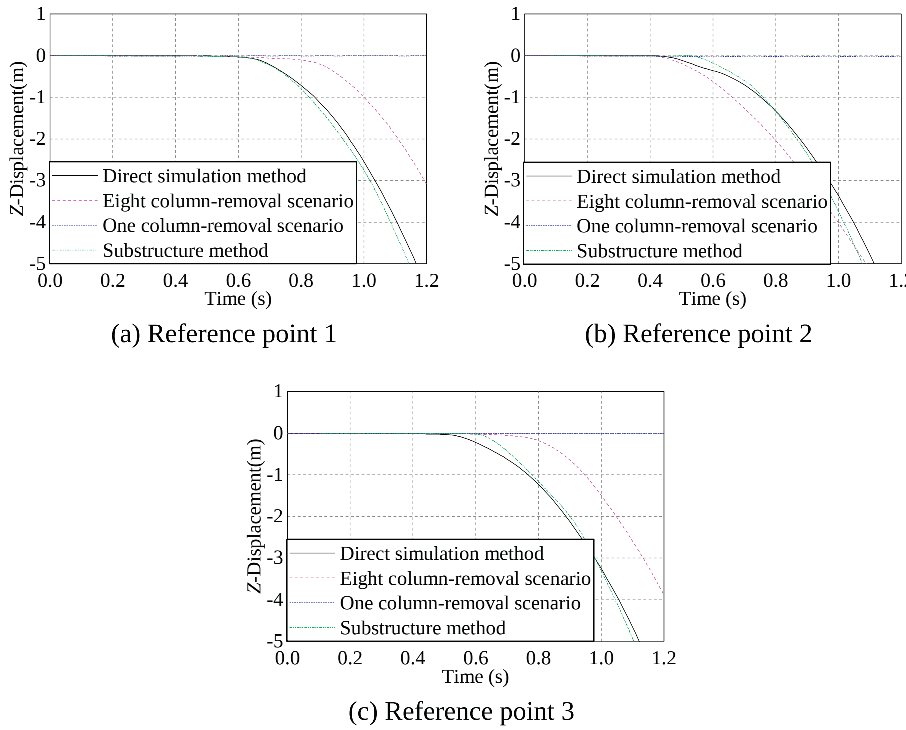


Fig. 14. Comparison of *Z*-direction displacement.

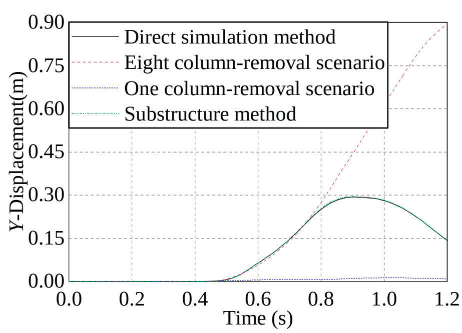


Fig. 15. Comparison of Y-direction displacement of reference point 1.

The steel columns on the ground floor are damaged by the explosion, and the remaining columns yield or lose stability quickly due to insufficient resistance to the self-weight of the remaining structure. Therefore the Z-direction velocity of reference point 1 is always increasing. Because of ignoring blast load effect of remaining columns on the ground floor, the Z-direction velocity of reference point 1 from eight column-removal scenario is growing slowly in the range 0.6 to 0.8 s. The Z-direction velocity of reference point 1 from substructure method is generally close to that from direct simulation method.

The comparison of Y-direction displacement of reference point 4 obtained from different methods is shown in Fig. 17. After the specified Column A3 on the ground floor is removed suddenly, the residual structure does not collapse, and it hardly has any influence on Y-direction displacement of reference point 4.

The Y-direction displacement of reference point 4 from eight column-removal scenario is growing slowly in the range 0.6 to 0.8 s, and then grow rapidly. This indicates that the Column D3 cannot bear the dead weight of the superstructure and becomes invalid. The Y-direction displacement of reference point 4 from substructure method is generally close to that from direct simulation method.

6.3.3.2. Axial stress of frame column. Fig. 18 shows the comparison of axial stress of frame columns on the second floor. After the specified columns on the ground floor are removed suddenly, the axial stress of the corresponding upper columns will be released quickly, such as reference point 6 under eight column-removal scenario and one column-

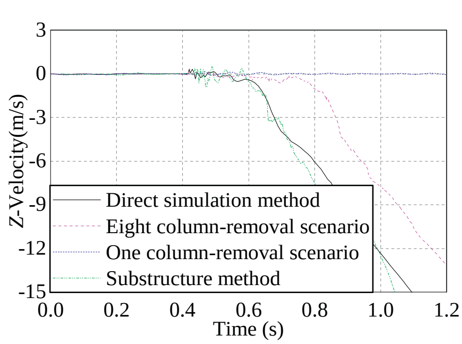


Fig. 16. Comparison of Z-direction velocity of reference point 1.

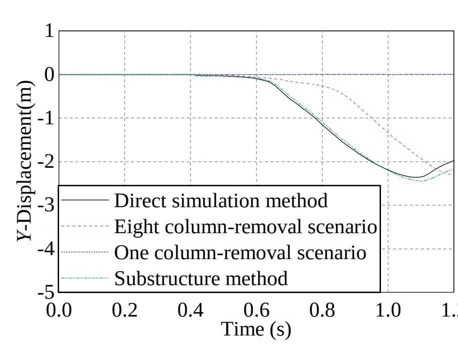


Fig. 17. Comparison of Y-direction displacement of reference point 4.

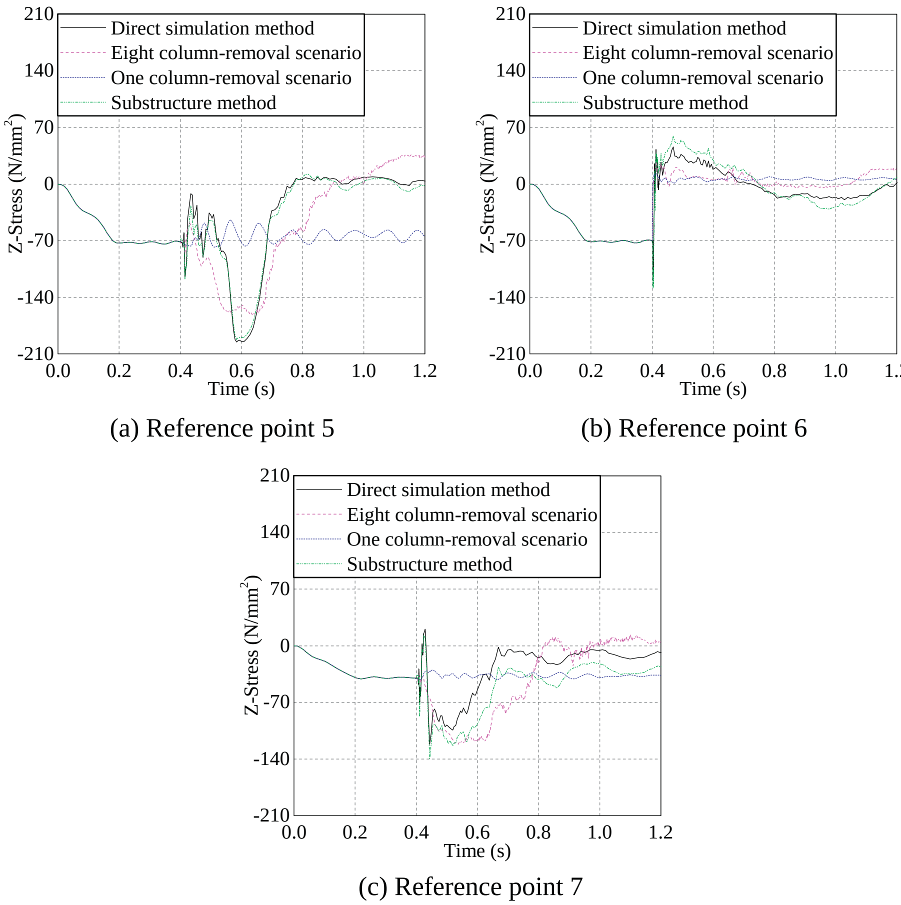


Fig. 18. Comparison of axial stress of frame columns.

removal scenario. The internal force of the steel frame will be redistributed to seek the new load transfer path. The adjacent columns will carry a large portion of the load previously carried by the removed columns, and are easy to lose stability or be severely damaged, such as reference point 7 and 5 under eight column-removal scenario.

There are some differences in axial stress of frame columns between eight column-removal scenario and direct simulation method because of blast load effect. On the whole, the variation rule of the axial stress from substructure method is consistent with that of the axial stress from direct simulation method.

6.3.3.3. Total reaction force. The comparison of *Z*-direction total reaction force obtained from different methods is shown in Fig. 19. The Z-direction total reaction force of the whole structure obtained from one column-removal scenario fluctuates more frequently around its dead weight, and finally, it gradually tends to be stable due to structural damping.

There is no negative value of Z-direction total reaction force obtain from eight column-removal scenario, and the result is always higher than that from direct simulation method. This is mainly because that the blast load effect of structural members is not taken into account on the ground floor. The results obtained from direct simulation method and substructure method agree very well with each other.

The comparison of Y-direction total reaction force obtained from different methods is shown in Fig. 20. After the specified columns on the ground floor are removed suddenly, the Y-direction total reactions force obtained from eight column-removal scenario and one column-removal scenario fluctuate more frequently around zero, and finally, they gradually tend to be zero because of structural damping. But they are much smaller than that from direct simulation method because of the lack of blast load effect of structural members. The result obtained from substructure method is consistent with that of direct simulation method.

All in all, the non-zero initial conditions and initial damage of the structural members cannot be ignored under the condition of a massive explosion inside a building. The substructure method is reliable and can predict the structural responses accurately.

7. Conclusion

When a massive explosion happens inside a building, a number of structural members are damaged or failed, along side with non-zero

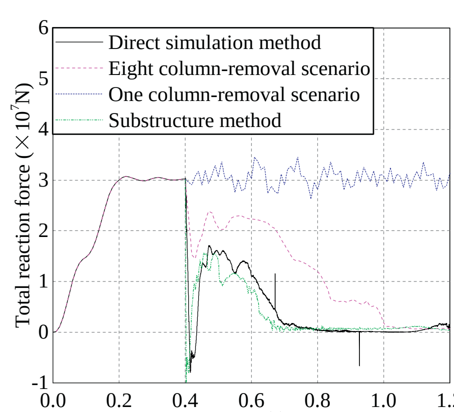


Fig. 19. Comparison of Z-direction total reaction force of the whole structure.

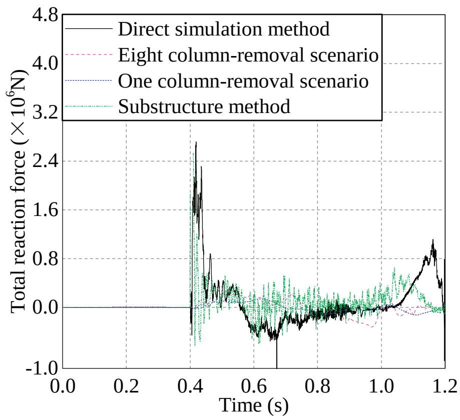


Fig. 20. Comparison of Y-direction total reaction force.

initial conditions. Aiming at this problem, this paper presents a new method for progressive collapse analysis of steel frames under blast load.

By way of example of a steel frame with 5 stories in height, 4 bays in the longitudinal direction, 3 bays in the transverse direction, direct simulation method, alternative load path method and proposed method are used to simulate the progressive collapse process, respectively. Through the example analyses, it is shown that blast load effect of the structural members cannot be ignored on the ground floor, and it can be ignored on the other floors by the effect of the reinforced concrete slab. The non-zero initial conditions and initial damage of the structural members can be predicted well by the substructure model, the proposed method is also reliable and accurate.

By using the substructure method, blast load is applied to the structural members without regard to the chemical explosion process and blast load effect of structural members outside effective areas of influence. It can effectively reduce the computational expensiveness of finite element model and computer hardware configuration requirements.

It is imperative that the finite element analysis software has the function of geometric and material nonlinearities, but by using Eqs. (2)–(6), the substructure model does not involve the fluid-structure interaction between explosive charge and building structure. In the direct simulation, very complex simulation techniques [15,16], such as the detonation process of explosive charge, propagation process of blast shock wave, fluid-solid coupling mechanism, while in the substructure method, these simulation techniques are not involved. The proposed substructure method can significantly simply the simulation process and also greatly improve the computational efficiency. In addition, the proposed substructure method can predict the same failure mode and reflect the same loading transfer mechanism with the direct simulation method. Also, it should be notice that the analysis by using the traditional alternate load path method is also quite complex and normally the geometric and material nonlinearities are needed. It means that the proposed substructure method is similar in difficulty to the traditional alternate load path method. These analyses can be implemented by using commercial general software like ABAQUS and ANSYS.

There are some limitations on the proposed method. If the explosive charge is too close to the reinforced concrete slab, or even in case of contact explosion, the shape of the explosive charge cannot be ignored. The recommended value of r/r_0 is >3.0, where r is the distance between the slab and explosion center and r_0 is the radius of the explosive charge. In other words, the proposed substructure method cannot be used for the progressive collapse analysis in the case of r/r_0, which is smaller than 3.0. However, it should be noticed that if p_r is <0.040 N/mm^2 [27], the blast load effect on reinforced concrete slab can be ignored. The traditional alternative load path method, rather than the proposed substructure method, can be adopted by the structural engineers. For the steel column, the upper limit of p_r is greatly affected by column section and relative position between the column and explosive charge. The recommended value of p_r is 0.076 N/mm^2 [27], which is equal to that of reinforced concrete column.

In addition, it should be mentioned that the proposed substructure method is based on explicit dynamic solver rather than implicit static solver, which is more commonly used by structural engineers.

Progressive collapse is a global phenomenon and the studies about progressive collapse should investigate the response of the entire building. However, right now the studies about progressive collapse normally focus on only one storey substructure rather than the entire building structure. The past studies are based on column-removal scenario, which investigates the loading redistribution in the remaining structures after the column loss. In these studies, if the remaining structures cannot redistribute the vertical load and the structural components reach failure criteria, it is deemed that progressive collapse will occur. Therefore, it can be found that at current stage, the studies about progressive collapse are still based on substructure rather than the entire building structure. Thus, the substructure technique can still be used in the analysis about progressive collapse.

Acknowledgements

The authors gratefully acknowledge the financial support provided by the National Key Research and Development Program of China (2016YFC0701203), the National Natural Science Foundation of China (No. 51778086), the Fundamental and Frontier Research Project of Chongqing (cstc2015jcyjBX0024), and the Postdoctoral Research Project of Chongqing (Xm2017189).

References

[1] General Services Administration (GSA), Progressive collapse analysis and design guidelines for new federal office buildings and major modernisation projects, The US General Services Administration, 2013.
[2] Unified Facilities Criteria (UFC)-DoD, Design of buildings to resist progressive collapse, Department of Defense, 2013.
[3] J.R. McConnell, H. Brown, Evaluation of progressive collapse alternate load path analyses in designing for blast resistance of steel columns, Eng. Struct. 33 (10) (2011) 2899–2909.
[4] F. Feng, Dynamic response and robustness of tall buildings under blast loading, J. Constr. Steel Res. 80 (2013) 299–307.
[5] Y. Ding, X.R. Song, H.T. Zhu, Probabilistic progressive collapse analysis of steel frame structures against blast loads, Eng. Struct. 147 (2017) 679–691.
[6] K. Khandelwal, S. El-Tawil, Pushdown resistance as a measure of robustness in progressive collapse analysis, Eng. Struct. 33 (9) (2011) 2653–2661.
[7] S. Gerasimidis, J. Sideri, A new partial-distributed damage method for progressive collapse analysis of steel frames, J. Constr. Steel Res. 119 (2016) 233–245.
[8] M. Ettouney, R. Smilowitz, M. Tang, et al., Global system considerations for progressive collapse with extensions to other natural and man-made hazards, J. Perform. Constr. Facil. 20 (4) (2006) 403–417.
[9] J. Sideri, C.L. Mullen, S. Gerasimidis, et al., Distributed column damage effect on progressive collapse vulnerability in steel buildings exposed to an external blast event, J. Perform. Constr. Facil. 31 (5) (2017) 1–22.
[10] B.M. Luccioni, R.D. Ambrosini, R.F. Danesi, Analysis of building collapse under blast loads, Eng. Struct. 26 (1) (2004) 63–71.
[11] H. Hao, C.Q. Wu, Z.X. Li, et al., Numerical analysis of structural progressive collapse to blast loads, Trans. Tianjin Univ. 12 (s1) (2006) 31–34.
[12] Technical Manual (TM-5-855-1), Fundamentals of protective design for conventional weapons, US Department of the Army, Washington DC, 1986.
[13] Y.C. Shi, Z.X. Li, H. Hao, A new method for progressive collapse analysis of RC frames under blast loading, Eng. Struct. 32 (6) (2010) 1691–1703.
[14] T. Song, Lü Ly., Effects of explosive loads on progressive collapse performance of multi-storey steel frames, J. Southeast Univ. 41 (6) (2011) 1247–1252.
[15] LS-DYNA R9.0 Keyword User's Manual VOLUME I. Livermore Software Technology Corporation (LSTC)2016.
[16] LS-DYNA Theory Manual. , Livermore Software Technology Corporation (LSTC), 2016.
[17] K. Xu, Y. Lu, Numerical simulation study of spallation in reinforced concrete plates subjected to blast loading, Comput. Struct. 84 (2006) 431–438.
[18] M. Unosson, Numerical Simulations of Penetration and Perforation of High Performance Concrete with 75 Mm Steel Projectile. FOA-R-00-01634-311-SE. Technical Report. Tumba (Sweden), Defence Research Establishment Weapons and Protection Division, 2000.
[19] M. Unosson, L. Nilsson, Projectile penetration and perforation of high performance concrete : experimental results and macroscopic modeling, Int. J. Impact Eng. 32 (7) (2006) 1068–1085.

[20] F. Dinu, I. Marginean, D. Dubina, et al., Experimental testing and numerical modeling of steel frames under close-in detonations, 6th International Workshop on Performance, Protection & Strengthening of Structures under Extreme Loading, PROTECT2017, 11–12 December 2017, Guangzhou (Canton), China, vol. 210, 2017, pp. 377–385.
[21] G. Pijaudier-Cabot, J. Mazars, Damage models for concrete - handbook of materials behavior models - Section 6.13, Handbook of Materials Behavior Models 2001, pp. 500–512.
[22] G.R. Johnson, W.H. Cook, Fracture characteristics of three metals subjected to various strains, strain rates, temperatures and pressures, Eng. Fract. Mech. 21 (1) (1985) 31–48.
[23] C.Q. Wu, H. Hao, Modeling of simultaneous ground shock and airblast pressure on nearby structures from surface explosions, Int. J. Impact Eng. 31 (6) (2005) 699–717.
[24] J. Henrych, The Dynamics of Explosion and its Use, Elsevier Scientific Publishing Company, Amsterdam and New York, 1979.
[25] Eurocode 2: design of concrete structures, Civil Engineering, 2003.
[26] H. Chen, J.Y. Richard Liew, Explosion and fire analysis of steel frames using mixed element approach, J. Struct. Eng. 130 (7) (2004) 606–616.
[27] UFC 3-340-02, Unified Facilities Criteria: Structures to resist the effects of accidental explosions available from national institute of building sciences, U.S. Army Corps of Engineers, U.S. Naval Facilities Engineering Command (Preparing Activity), U.S. Air force Civil Engineer Support Agency, Washington DC, 2008.