Earthquakes and Structures, *Vol. 15, No. 2 (2018) 155-164*
DOI: https://doi.org/10.12989/eas.2018.15.2.155

A performance based strategy for design of steel moment frames under blast loading

Ghasem Dehghani Ashkezari*

Protective Structures and Materials Center, Passive Defense Department, Malek Ashtar University of Technology, Tehran, Iran

(Received October 26, 2016, Revised April 8, 2018, Accepted April 24, 2018)

Abstract. Design of structures subjected to blast loads are usually carried out through nonlinear inelastic dynamic analysis followed by imposing acceptance criteria specified in design codes. In addition to comprehensive aspects of inelastic dynamic analyses, particularly in analysis and design of structures subjected to transient loads, they inherently suffer from convergence and computational cost problems. In this research, a strategy is proposed for design of steel moment resisting frames under far range blast loads. This strategy is inspired from performance based seismic design concepts, which is here developed to blast design. For this purpose, an algorithm is presented to calculate the capacity modification factors of frame members in order to simplify design of these structures subjected to blast loading. The present method provides a simplified design procedure in which the linear dynamic analysis is preformed, instead of the time-consuming nonlinear dynamic analysis. Nonlinear and linear analyses are accomplished in order to establish this design procedure, and consequently the final design procedure is proposed as a strategy requiring only linear structural analysis, while acceptance criteria of nonlinear analysis is implicitly satisfied.

Keywords: steel moment resisting frames; capacity modification factor; performance based design; nonlinear dynamic analysis; linear dynamic analysis; blast loading

1. Introduction

Analysis and design of structures under blast loading is one the most complicated procedures in structural engineering. Because in addition to complex nonlinear analyses and material models, many details should be considered in order to have a desirable structural design. Main concepts existing in design of structure against blast loads are energy absorption, safety factors, load combination, strength properties, material properties and structural indeterminacy (Dusenberry 2010, Smith and Hetherington 1994). Two approaches are usually utilized in blast design of structures, including equivalent static and nonlinear dynamic analyses. Obviously, the nonlinear case is more accurate due to its ability for considering inelastic behavior, impact status, cyclic movements and strain rate properties.

In recent years, many attempts were carried out for design of earthquake-resistant structures which is led to deformation-based design rather than force-based one (Loulelis *et al*. 2012, Scawthorn and Chen 2002) Performance based design employs specific design criteria to reach a target structural behavior and performance. Also, performance based design plays a key role for structures subjected to explosives. Since steel material has identical capacity in tension and compression, a steel frame may have better behavior under rationally small explosives when a structure is not designed for blast. But concrete only sustains compression, whereas tension is endured by its reinforcements. Hence, since direction of the blast effects is unknown, concrete components performs better when a structure is designed for blast (Bangash 2006, Bangash 2009). Accordingly, there are many works dealing with assessment of steel structures subjected to explosive loading, among which some of them are explained here. Progressive collapse of steel building due to explosive attacks was studied by Elsanadedy *et al*. (2014). Guzas and Earls simulated effects of those loads for steel beam-column members (Guzas and Earls 2011). Liew and Chen utilized fiber elements for fire and explosion analysis of steel frames (Liew and Chen 2004). Mohamed and Louca fulfilled performance based design blast resistant offshore steel structures (Mohamed Ali and Louca 2008a, 2008b). Quiel *et al*. offered a performance based approach for quantifying structural resilience induced by blast damage (Quiel *et al*. 2015). Monir utilized unidirectional passive dampers to increase flexibility of blast resistant steel structures (Monir 2013). Fu carried out dynamic response evaluation of tall buildings due to blast loading (Fu 2013). Gerasimidis and Sideri proposed a new partially distributed damage technique to analyze progressive collapse of steel frames (Gerasimidis and Sideri 2016). Warn and Bruneau evaluated blast resistance of seismically designed steel plate shear walls (Warn and Bruneau 2009). Survivability of steel frames under fire and blast was studied by Liew (2008). Some studies accomplished dynamic analysis, performance based assessment and progressive collapse analysis of steel frame structures subjected to catastrophic events like earthquake, blast, impact and fire (Ferraioli 2016, Ferraioli *et al*. 2014, Ferraioli *et al*. 2018, Formisano *et al*. 2015, Formisano and Mazzolani 2010, 2012, Kaveh *et al*. 2015,

*Corresponding author, Ph.D.
E-mail: ghdeas@yahoo.com

ISSN: 2092-7614 (Print), 2092-7622 (Online)

Kaveh and Zakian 2016). Also, there are many studies focused on performance and damage assessment of other blast resistant structures (Bogosian *et al*. 1999, Echevarria *et al*. 2016, Ma *et al*. 2009, Mahmoud 2014, Stoddart *et al*. 2013).

This paper proposes a strategy for design of steel moment resisting frames under blast loads. In order to reduce computational efforts existing in inelastic dynamic analysis of the structures, the capacity modification factors of frame members are calculated to simplify the design of these structures subjected to blast loading such that only an elastic dynamic analysis is carried out in the proposed design procedure, and thus although acceptance criteria of inelastic analysis are indirectly considered, there is no need to perform any time-consuming inelastic dynamic analysis.

2. Brief backgrounds on performance based design of steel structures

2.1 Performance based seismic design

In recent decade, force-based design of a structure is modified to displacement-based design, which is in progressive state of development in performance based design concepts (ASCE 2014). Based on force–deformation relationships, action of a structural member can be categorized to force control and deformation control cases. Also, behavior of a member is assumed to be in three groups including ductile, semi-ductile and brittle. Linear and nonlinear analyses (static or dynamic) are utilized in performance seismic design and each kind of these analyses has its own acceptance criteria. Strength and stiffness of primary and secondary members are usually changed during a nonlinear analysis. Thus, nonlinear procedures give more realistic results with respect to linear ones.

In this subsection, linear dynamic procedure of performance based seismic design for steel structures is concisely discussed. This procedure considers the following load combinations (ASCE 2014)

$$Q_{UD} = Q_G + Q_E \tag{1}$$

And

$$Q_{UF} = Q_G + \frac{Q_E}{C_1 C_2 J} \tag{2}$$

In which Q_{UD} and Q_{UF} are actions caused by gravity and earthquake loads for deformation control and force control members, respectively. Q_G and Q_E are gravity and earthquake demands, respectively. J is the force-delivery reduction factor, varying between 1 to 2. For performance level of immediate occupancy, it is taken as unity. C_1 and C_2 are modification coefficients. Acceptance criteria for the actions obtained from linear analysis are described as follows

$$m\kappa Q_{CE} \geq Q_{UD} \tag{3}$$

And

$$\kappa Q_{CL} \geq Q_{UF} \tag{4}$$

where Q_{CE} and Q_{CL} are expected and lower bound strengths of a member, respectively. κ is a knowledge factor for implementing uncertainty effects. This factor is usually taken as 0.75, while it is 1 when a comprehensive study is fulfilled. m is component capacity modification factor to consider expected ductility associated with the action at the chosen structural performance level. Clearly, acceptance criteria for force control actions are handled by force constraints, whereas they are handled by deformation constraints for those of deformation control.

For a steel moment frames, yielding rotations of beams and columns, θ_y, are calculated by the following relations (ASCE 2014)

$$\theta_y = \frac{ZF_{ye} l_b}{6EI_b} \tag{5}$$

And

$$\theta_y = \frac{ZF_{ye} l_c}{6EI_c}\left(1 - \frac{P}{P_{ye}}\right) \tag{6}$$

in which l_b, l_c, $P_{ye}=A_g \times F_{ye}$ and P are length of beam span, length of column, expected yield force and axial force, respectively; Z and E are plastic and elastic modulus, respectively; I_c is moment of inertia for column; and I_b is moment of inertia for beam. Expected strengths of beams and columns are obtained from

$$Q_{CE} = M_{CE} = ZF_{ye} \tag{7}$$

And

$$Q_{CE} = M_{CE} = 1.18 ZF_{ye}\left(1 - \frac{P}{P_{ye}}\right) \leq ZF_{ye} \tag{8}$$

For bending actions in which axial compressive forces do not exceed 10 per cent of the lower bound axial compressive force, bending and shear actions are specified as deformation control. On the other hand, for a column with axial compressive forces which do not exceed 50 percent of the lower bound axial compressive force, P_{CL}, bending and compressive actions are specified as deformation control and force control, respectively. Hence, we have (ASCE 2014)

$$\begin{aligned} &for \quad 0.2 \leq \frac{P_{UF}}{P_{CL}} \leq 0.5 \\ &\frac{P_{UF}}{P_{CL}} + \frac{8}{9}\left[\frac{M_x}{m_x M_{CE_x}} + \frac{M_y}{m_y M_{CE_y}}\right] \leq 1 \end{aligned} \tag{9}$$

And

$$\begin{aligned} &for \quad \frac{P_{UF}}{P_{CL}} < 0.2 \\ &\frac{P_{UF}}{2P_{CL}} + \frac{M_x}{m_x M_{CE_x}} + \frac{M_y}{m_y M_{CEy}} \leq 1 \end{aligned} \tag{8}$$

But when this percentage exceed 50 percent, force control action governs for the column as expressed below

$$\frac{P_{UF}}{P_{CL}} + \frac{M_{UF_x}}{m_x M_{CL_x}} + \frac{M_{UF_y}}{m_y M_{CLy}} \leq 1, \tag{11}$$

2.2 Blast design

In early 20th century, past experiences of catastrophic events were used for blast design of structures. But these empirical methods had a main weakness due to their qualifying properties. After presentation of design codes for explosive loading of structures, initial versions of UFC-3-340-02 (DoD 2014) emphasized on reinforced concrete structures, while newer versions deal with other materials as well.

Provisions specified for design of protective structures depend on feature and location of explosion, sensitivity of a structural system, and physical properties and configuration of the structure. Based on UFC-3-340-02, four protection level are introduced (DoD 2014). In this research, the first level is taken into account. Also, blast loads are classified to unconfined and confined explosions, based on the charge confinement. Unconfined explosion is categorized to free air burst, air burst and surface burst, while the confined one is categorized to fully vented, partially confined and fully confined. Here, unconfined explosive of surface burst is loaded for the steel frames. These loads should be calculated form numerous graphs and relations written in UFC-3-340-02 where four load conditions are implemented for a cubic structure including front wall, roof, side wall and back wall conditions. Here, we study 2D frames requiring front wall, roof and back wall conditions for loading. For a single frame, apart from distributed loads, bay width (the distance between two adjacent frames at the same direction) should be used for computing the concentrated lateral loads acting on each floor. Therefore, lateral pressure obtained from the design code should be multiplied by column's width in order to calculate distributed lateral loads. Then, the lateral pressure should also be multiplied by bay and column widths to attain the concentrated forces. In conjunction with roof load, bay width is employed to find the distributed load of beams.

Acceptance criteria of nonlinear analysis of a steel frame structure subjected to explosives specify maximum allowable chord rotation of 2 degrees and maximum allowable drift of H/25 wherein H is story height (DoD 2014).

Strain rate effects of steel material is significant in blast analysis and design. Therefore, Johnson-Cook plasticity model is often utilized for impact phenomenon. Strain hardening, strain rate and thermal softening are the main features implemented by this model that is expressed below (Schwer 2007)

$$\begin{aligned} \sigma_Y &= \left[A + B(\varepsilon_{eff}^p)^N\right] \\ &(1 + C \ln \dot{\varepsilon})\left[1 - (T_H)^M\right] \end{aligned} \tag{12}$$

Wherein

$$\begin{aligned} \dot{\varepsilon} &= \frac{\dot{\varepsilon}_{eff}^p}{\dot{\varepsilon}_0}, \\ T_H &= \frac{T - T_R}{T_M - T_R}, \\ \Delta T &= \frac{1}{\rho C_p} \int \sigma d\varepsilon_{eff}^p. \end{aligned} \tag{13}$$

In which ε_{eff}^p is effective plastic strain; $\dot{\varepsilon}$ is strain rate; T_H, T_M and T_R are homogenous, melting and reference temperatures; ρ and C_p are mass density and specific heat capacity, respectively. The remaining parameters are constants determined by experiments for a selected material.

3. An algorithm for finding capacity modification factors

Here, an algorithm is presented to find capacity modification factors of steel frames by defining an inverse problem. Concepts of this algorithm are inspired from performance based seismic design codes (ASCE 2014). Hereafter, mB is defined as the capacity modification factor for blast-induced actions. Fundamental steps of calculating mB are summarized as follows:

1) Preform an initial design for a frame structure using usual design codes (e.g., ASCE7-10 (ASCE 2013)) without considering blast loads.
2) Compute blast loads for a specific detonation and distance from the structure, according to UFC-3-340-02 (DoD 2014).
3) Simulate the structure and perform a nonlinear dynamic analysis. Johnson-Cook strain rate dependent model is employed for material definitions. Also, the previously computed blast loads should be applied to the structure.
4) Check drift and chord rotation of the analyzed structure, according to UFC-3-340-02 specifications for steel frames. If deformations of the structure are acceptable, redesign the structure by changing cross-sectional areas and perform a nonlinear analysis again so that deformations reach to their acceptance values as near as possible. Otherwise, reinforce the structure by changing cross-sectional areas in order to reach acceptance values, but not further. Then, modify the design as same the previous stage to have minimal deference between existing deformations and allowable ones. It should be noted that final design cannot be weaker than the initial design.
5) After designing the structure in accordance with the previous step, accomplish linear dynamic analysis for that design. All details of the structure should be identical to finally designed structure at the previous step.
6) For the analyzed structure of step 5, find maximum moments for beams due to the loadings. Furthermore, find critical condition of axial force and moment interaction for columns. Consequently, calculate maximum actions of the members denoted by Q_{UD} in Eq. (1).
7) In this step, calculate Q_{CE} for all members of the structure in step 6, according to ASCE41-13 (ASCE 2014) as mentioned in Eqs. (7) and (8).
8) Based on acceptance criteria of linear dynamic analysis, all actions are supposed to be deformation control. Assume κ=1 and obtain m_B for each member of a steel moment frame as follows:

For beams

$$m_B = M_U / M_{CE} \quad (14)$$

For columns

$$for \quad 0.2 \le \frac{P_U}{P_{CL}} \le 0.5$$

$$m_B = \frac{8M_U}{9M_{CE}(1-\frac{P_U}{P_{CL}})} \quad (15)$$

and

$$for \quad \frac{P_U}{P_{CL}} < 0.2$$

$$m_B = \frac{M_U}{M_{CE}(1-\frac{P_U}{2P_{CL}})} \quad (16)$$

where M_U, P_U, P_{CL} and M_{CE} are bending moment, axial force, lower bound capacity of axial force and expected capacity of bending moment. If P_U/P_{CL} is more than 50 percent, then the column action is manifested as force control leading to m_B=1. Whenever value of mB is less than unity, the action is also manifested as force control one with m_B=1.

1) Repeat the procedure from step 2 to step 8 again for a new blast loading with another detonation and distance from the structure.
2) With responses obtained from numerous iterations, report suitable mB for beams and columns by computing response statistics.

The present algorithm can be improved and extended to different types of structural systems like braced frames and reinforced concrete frames. But it is herein developed to steel moment resisting frames. A flowchart of this algorithm is provided in Fig. 1.

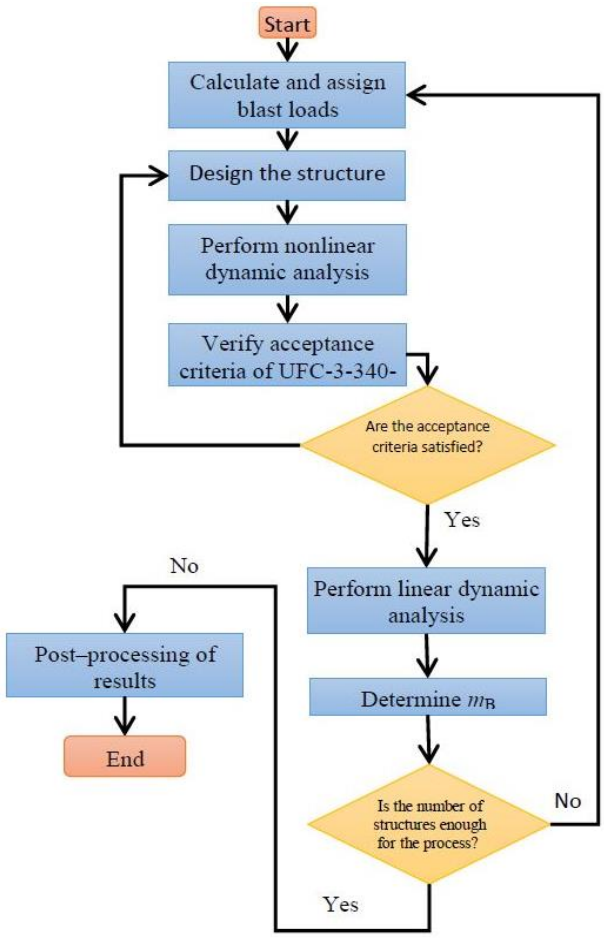


Fig. 1 Flowchart of the proposed algorithm for computing the capacity modification factor (m_B) of actions induced by blast loads

4. Structural models

Based on UFC 3-340-02 (structures to resist the effects of accidental explosions), it is true that blast pressure of each point on the structure depends on its distance from the source and the angle between wave propagation direction and the member surface. Therefore, the pressure is non-uniformly distributed at various points on the structure. However, when the structure is far from the source, uniform distribution of blast pressure on the structure can be assumed, because the distance becomes large and thus uniform pressure exist until higher level of the structure.

In the case of air blast, a better method is calculation of Mach wave index on the front side of the

Structure for which blast pressure has uniform distribution until a specific level. But blast pressure is usually assumed to be completely uniform for considering margin of safety and to avoid complexity.

In the case of surface blast having non-uniform pressure, conservatively, it is assumed that blast is far from the source so that semi-spherical Mach wave front is formed as vertical cylinder leading to uniform pressure distribution.

Consequently, as in this research surface blast in far range is studied, uniform pressure is defined for each side of the structure.

Blast loading on each side of a structure (side walls, roof and exposed sides of columns and beams) can be treated in two ways:

1) Covering walls (or infills) collapse and cannot sustain blast pressure.
2) Covering walls (or infills) can sustain blast pressure.

In the second case which is assumed here, lateral blast loads are transferred to rigid diaphragms and columns. The transferred loads are assigned to lateral resisting systems based on their stiffnesses, and are also applied as concentrated loads at the rigid diaphragms. In this case, the blast loads applied to the frames are larger than those of the first case. Therefore, large cross sections should be selected in order to satisfy design criteria. Also, it is assumed that side walls are only connected to rigid diaphragm and this assumption reduces blast loads acting on columns.

In order to consider load conditions of a frame due to blast loads, three load conditions are applied to a portal frame. Because reaching to acceptance values with minimal tolerance for all structural members at the same time is not an easy task, particularly when nonlinear analysis is under

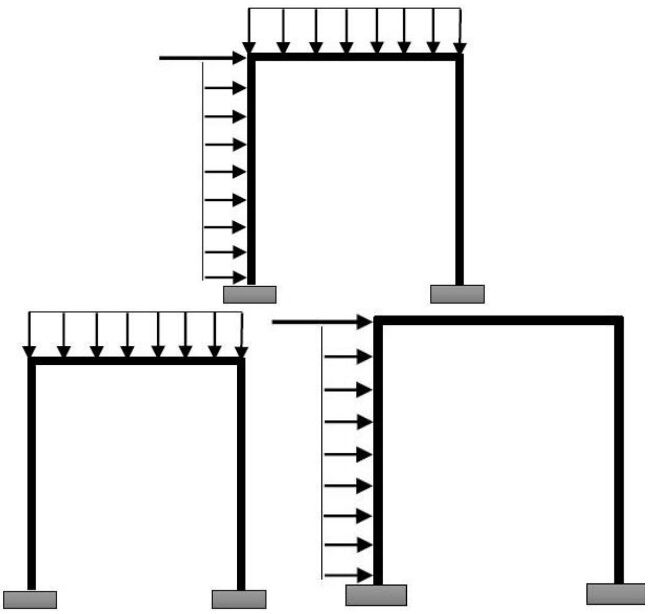


Fig. 2 Portal frames under three loading conditions: (a) the first deformation mode, (b) the second deformation mode, and (c) the third deformation mode

consideration.

Here, 2D frame is assumed and a portal frame with three loading conditions is considered for deformation assessment. On the other hands, there are two possibilities for curvature modes of a member, which include single curvature and double curvature modes. Clearly, damages due to double curvature mode are higher. In seismic loading (only lateral loads), the structure has double curvature mode, while in blast loading (vertical and lateral loads) both of the curvature modes exist. Furthermore, curvature modes of a 3D frame also include these two modes. Therefore, behavior of this portal frame can be generalized to multi - story frames. Because deformation of members can be simulated by a portal frame component having different loading conditions as represented in Fig. 2. These three cases provide fine performance assessment. Here, we utilize three detonations with two distances leading to six load magnitudes depicted in Fig. 3.

Therefore, 18 portal frame samples should be evaluated as listed in Table 1. Z index is larger than 3 for all the loads provided in Table 1, which shows that far range blast loading is considered. Material properties for linear and nonlinear stages are reported in Table 2 and Table 3, respectively, which the inelastic stage uses Johnson Cook model incorporating strain rate effects for the structure (Schwer 2007). The proposed algorithm is followed for these 18 samples. Lengths of columns and beam for the portal frame are 3.5 and 4 m, respectively. Finite element method is employed for the simulations, in which 14 elements are used for the beam and 16 elements are assigned to the columns. Load combination is written as follows

$$Comb = D + L + B \tag{17}$$

in which D, L and B denote dead, live and blast loads, respectively. Uniformly distributed load of 28 kN/m is

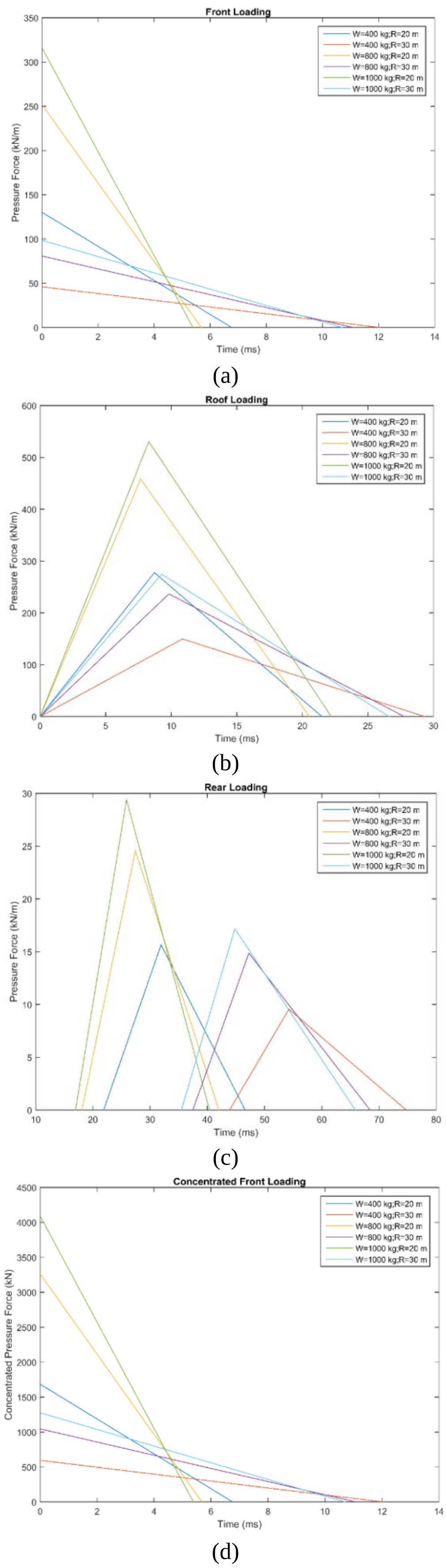


Fig. 3 Six types of blast time histories applied to the frames: (a) front wall load, (b) roof load, (c) rear wall load, and (d) concentrated front wall load

Table 1 Sample portal frames categorized according to applied loads

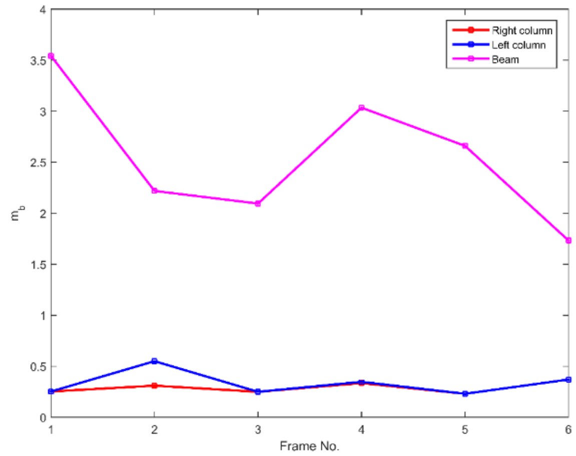
Sample No.	Loading Type	TNT Charge (kg)	Distance (m)	*Z* (ft/lb3)
1		400	20	6.8425
2		400	30	10.2638
3	1	800	20	5.4309
4		800	30	8.1464
5		1000	20	5.0416
6		1000	30	7.5624
7		400	20	6.8425
8		400	30	10.2638
9	2	800	20	5.4309
10		800	30	8.1464
11		1000	20	5.0416
12		1000	30	7.5624
13		400	20	6.8425
14		400	30	10.2638
15	3	800	20	5.4309
16		800	30	8.1464
17		1000	20	5.0416
18		1000	30	7.5624

Table 2 Material definitions for elastic analysis

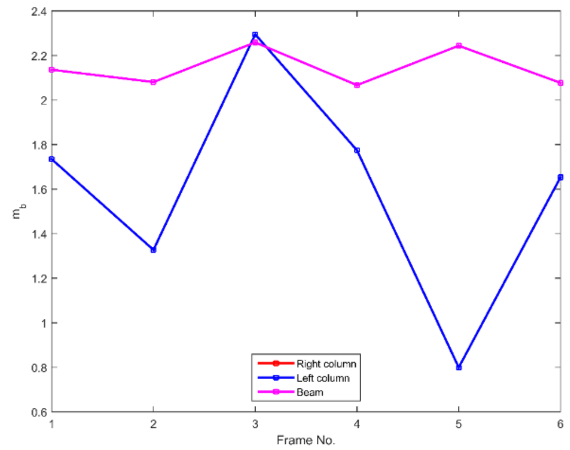
Parameters	Assigned value
Elastic modulus	2e8 kN/m^2
Poisson ratio	0.3
Density	7.849 kg/m^3
Damping ratio	0.05

Table 3 Material definitions for inelastic analysis

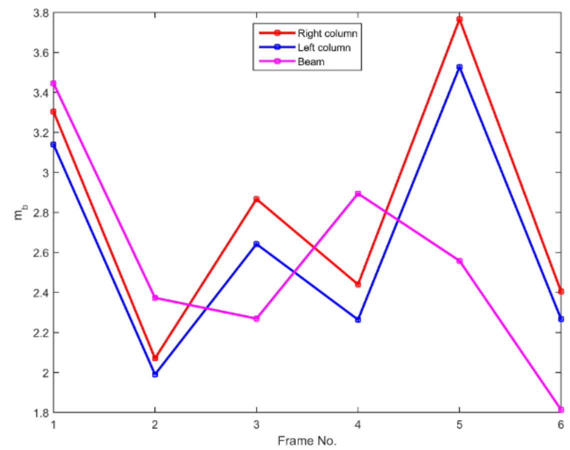
Parameters	Assigned value
Elastic modulus	2e8 kN/m^2
Poisson ratio	0.3
Density	7.849 kg/m^3
Damping ratio	0.05
Johnson Cook model	
A	286142 kN/m^2
B	500163 kN/m^2
C	0.017
N	0.228
M	0.917
ε_0	1
Tmelting	1773 K
Ttransition	293 K



(a)



(b)

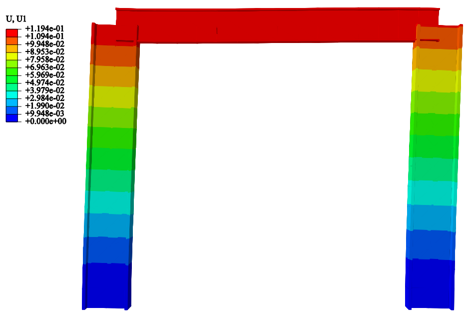


(c)

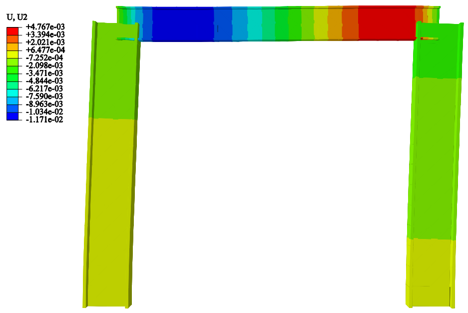
Fig. 4 Mean values of *mB* for the sample frames: (a) the frames with the first deformation mode, (b) the frames with the second deformation mode, and (c) the frames with the third deformation mode

applied to all beam members, due to dead and live loadings. As already mentioned, a multi-story frame with several spans can be decomposed to portal frame components with three loading conditions like those presented in Fig. 2. Cross-sections of members are selected from IPB and IPE types. Based on the UFC-3-340-02, allowable drift ratio in terms of story height and chord rotation for a steel frame are 0.04H and 2 degrees, respectively. As it is visible, only front wall and roof loads are employed, because there is no portal frame component for multi-span frame with front wall, roof and back wall loads at the same time. Moreover, front wall load always creates critical condition in comparison with the back wall case.

Analyses are terminated after 30 ms depending on the loading details. Chord rotation for two ends of beam and two ends of columns should be determined. Once final assignment and arrangement of an undertaken portal frame

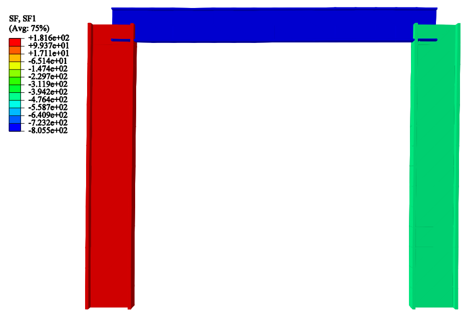


(a)

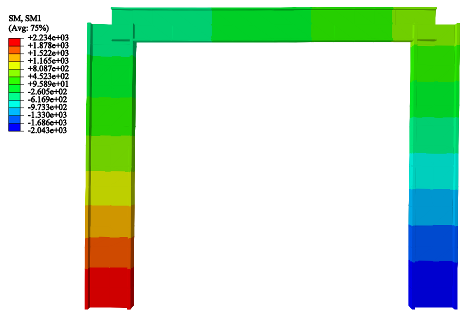


(b)

Fig. 5 Displacement field for portal frame of the first sample obtained by nonlinear dynamic analysis at the time corresponding to maximum values: (a) at horizontal direction, and (b) at vertical direction



(a)



(b)

Fig. 6 Action fields for portal frame of the first sample obtained by linear dynamic analysis at the time corresponding to maximum values: (a) axial force, and (b) bending moment

(i.e., one of 18 samples) is attained, this portal frame should be prepared for linear analysis. It should be note that the final cross-sections obtained for the frame may have

Table 4 Mean values of *mB* for the sample frames with the first deformation mode

Component	mB						Mean *mB*
	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Frame 6	
Right column	0.2531	0.3101	0.25	0.3352	0.2307	0.3701	0.2915
Left column	0.2530	0.5568	0.25	0.3476	0.2307	0.3701	0.3336
Beam	3.5417	2.2185	2.0947	3.0345	2.6601	1.7333	2.5471

Table 5 Mean values of *mB* for the sample frames with the second deformation mode

Component	*mB*						Mean *mB*
	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Frame 6	
Right column	1.7346	1.3274	2.2949	1.7745	0.7998	1.6534	1.5974
Left column	1.7346	1.3274	2.2949	1.7745	0.7998	1.6534	1.5974
Beam	2.1362	2.0808	2.2588	2.0670	2.2439	2.0773	2.1440

Table 6 Mean values of *mB* for the sample frames with the second deformation mode

Component	mB						Mean *mB*
	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Frame 6	
Right column	3.3044	2.0708	2.8671	2.4403	3.7664	2.4051	2.8090
Left column	3.1400	1.9903	2.6422	2.2641	3.5273	2.2676	2.6386
Beam	3.4455	2.3734	2.2695	2.8949	2.5584	1.8140	2.5593

unreasonable sizes in engineering aspect, but it is not important as we solve an inverse problem for finding *mB*, which is not a reasonable structural design necessarily. Once linear dynamic analysis is accomplished, one should find critical status of members in terms of mobilized forces and moments. For the columns, two critical conditions are evaluated for axial force and bending moment interaction. Firstly, the condition in which axial force is maximum. Secondly, the condition in which bending moment is maximum. mB is obtained for the both conditions and the condition with smaller *mB* is reported as capacity modification factor of that column. In the case where *mB* is smaller than 1 and/or P_U/P_{CL} is larger than 50 percent, *mB* is selected as unity and the member action is known as force control. After numerous analyses for each sample through the proposed algorithm, mean values of *mB* are listed by Tables 4-6 and Fig. 4. For example, displacement responses of nonlinear case, and axial force and bending moment of the linear case of the first sample are illustrated in Figs. 5 and 6.

5. Discussion on results and applications

Clearly, when *mB* is large for a member, ductility of that member is large as well. Thus, the member with large mB can sustain nonlinear actions, with having more capacity

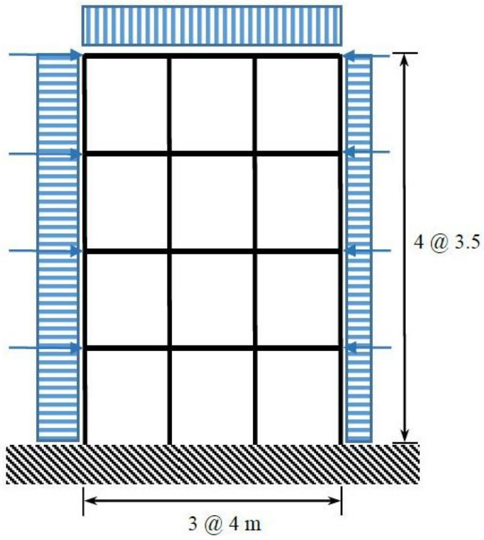


Fig. 7 Geometry of a 4 story frame and its applied blast loads

due to nonlinear deformations. Based on the results obtained in Tables 4-6 and Fig. 4, the following comments can be summarized for design of a multi-story steel frame, considering the margin of safety:

I. For beams of a frame with only lateral blast load or simultaneous lateral and downward loads, *mB* is taken as 2.5, whereas it is 2.1 when only downward blast load exists.

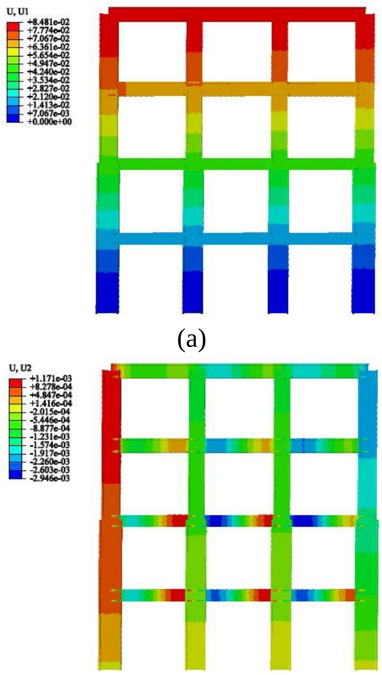
II. For external and internal columns of a frame, *mB* is chosen as unity when lateral and downward blast loads are imposed simultaneously.

III. For external and internal deformation control columns of a frame, *mB* is selected as 2.6 when only lateral blast load exists, while it is taken as 1.6 for the case where only downward blast force is loaded.

Now, after comparing mB values of beams for all the mentioned blast loading conditions with those of seismic loading in ASCE41-13 (ASCE 2014), one can understand that *mB* values of seismic and blast loads are slightly near together when only lateral blast loading is imposed, with respect to those of downward blast load. Because the first load condition of blast loading (lateral load) is similar to seismic loading of a structure for equivalent static analysis.

According to the suggested *mB* values mentioned above, one can merely utilize a linear dynamic analysis followed by the corresponding acceptance criteria to design a steel frame under blast loads without considering nonlinear analysis, while effects of nonlinear analysis are implicitly implemented by the obtained *mB* values. In other words, by using the proposed *mB* values, procedure of performance based seismic design for linear dynamic analyses can be followed for blast design of steel moment frames as concisely explained in section 2.

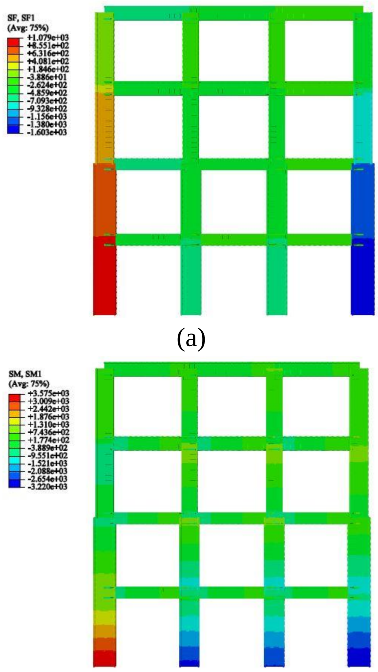
As an illustrative example, a 4 story frame (see Fig. 7) is designed by the attained *mB* values. Maximum displacements, axial forces and bending moments are indicated through contours in Figs. 8 and 9. Here, a linear dynamic analysis is accomplished and acceptance criteria of seismic design are verified by using *mB* values as illustrated



(a)

(b)

Fig. 8 Displacement field of the four story frame obtained by nonlinear dynamic analysis at the time corresponding to maximum values: (a) at horizontal direction, and (b) at vertical direction



(a)

(b)

Fig. 9 Action fields of the four story frame obtained by linear dynamic analysis at the time corresponding to maximum values: (a) axial force, and (b) bending moment

in Table 7. Available *IPE* and *IPB* cross-sections are selected for the members. Therefore, demand to capacity ratios of columns are slightly larger than 1 as mentioned in Table 7. Nevertheless, Tables 8 and 9 show that chord

Table 7 Maximum demands and applied *mB* values for design of 4 story frame using linear dynamic analysis

Section group	Assigned section	*Pu*	*Mu*	*mB*	Demand to capacity ratio
Cg1	IPB1000	1704.37	3866.03	1	1.2333
Cg2	IPB900	963.993	2981.12	1	1.0666
Cg3	IPB800	785.578	1807.11	1	0.8057
Cg4	IPB700	749.652	1263.95	1	0.9794
Bg1	IPE550	-	855.06	2.6	0.5147
Bg2	IPE600	-	890.932	2.5	0.4398

Table 8 Maximum chord rotation of 4 story frame and corresponding allowable value

Section group	Assigned section	Max. chord rot.	Allowable. chord rot.
Cg1	IPB1000	0.008	0.0349
Cg2	IPB900	0.004	
Cg3	IPB800	0.008	
Cg4	IPB700	0.004	
Bg1	IPE550	0.0032	
Bg2	IPE600	0.004	

Table 9 Maximum drift of 4 story frame and corresponding allowable value

Story No.	Drift	Allowable drift
1	0.0175	0.14
2	0.0278	
3	0.0237	
4	0.0135	

rotation and drift are still less than allowable values. As a result, the proposed method successfully designed the frame with suitable margin of safety. However, results confirm that *mB* may even be increased for further applications.

5. Conclusions

In this research, a simplified method is presented for design of steel moment resisting frames subjected to blast waves originating from ground level (far range surface burst). This method accelerates procedure of analysis and design through employing linear dynamic analysis followed by its simple acceptance criteria, instead of tedious and time-consuming nonlinear dynamic analyses. In order to achieve this scope, capacity modification factors are calculated for blast loads by using an algorithm with an inverse process such that effects of nonlinear dynamic analysis are indirectly taken into account. In this process, UFC-3-340-02 blast design criteria and ASCE41-13 seismic design criteria are incorporated for nonlinear and linear analyses, respectively. The present method determines mB for each member of a steel moment frame according to its location and loading positions. The proposed procedure utilizes linear dynamic analysis and neglects problems of nonlinear dynamic analyses like convergence, computational efforts and acceptance criteria assessments. Thus, it is expected that acceptance criteria of nonlinear analysis are satisfied when those of linear analysis are allowable. Moreover, a simple software may also be used for analysis and design of the steel frames under blast loads as there is no need to consider nonlinear and complex martial models.

Acknowledgements

The author is grateful to the constructive ideas and suggestions of anonymous reviewer and collaborator contributed in this research.

References

ASCE (2013), Minimum Design Loads for Buildings and Other Structures, ASCE 7-10: American Society of Civil Engineers.

ASCE (2014), Seismic Evaluation and Retrofit of Existing Buildings: ASCE 41-13, *American Society of Civil Engineers.*

Bangash, T. (2006), *Explosion-Resistant Buildings: Design, Analysis, and Case Studies*, Springer.

Bangash, Y.H. (2009), *Shock, Impact and Explosion: Structural Analysis and Design*, Springer.

Bogosian, D.D., Dunn, B.W. and Chrostowski, J.D. (1999), “Blast analysis of complex structures using physics-based fast-running models”, *Comput. Struct.*, **72**, 81-92.

DoD (2014), Structures to Resist the Effects of Accidental Explosions: UFC 3-340-02, Department of Defense, Unified Facilities Criteria 3-340-02, Washington, DC.

Dusenberry, D.O. (2010), *Handbook for Blast Resistant Design of Buildings*, John Wiley & Sons.

Echevarria, A., Zaghi, A.E., Christenson, R. and Accorsi, M. (2016), “CFFT bridge columns for multi hazard resilience”, *J. Struct. Eng.*, **142**, C4015002.

Elsanadedy, H.M., Almusallam, T.H., Alharbi, Y.R., Al-Salloum, Y.A. and Abbas, H. (2014), “Progressive collapse potential of a typical steel building due to blast attacks”, *J. Constr. Steel Res.*, **101**, 143-157.

Ferraioli, M. (2016), “Dynamic increase factor for pushdown analysis of seismically designed steel moment-resisting frames”, *Int. J. Steel Struct.*, **16**, 857-875.

Ferraioli, M., Avossa, A.M. and Mandara, A. (2014), “Assessment of progressive collapse capacity of earthquake-resistant steel moment frames using pushdown analysis”, *Open Constr. Build. Technol. J.*, **8**, 324-336.

Ferraioli, M., Lavino, A., Mandara, A., Donciglio, M. and Formisano, A. (2018), “Seismic and robustness design of steel frame buildings”, *Key Eng. Mater.*, **763**, 116-123.

Formisano, A. and Mazzolani, F. (2010), “On the catenary effect of steel buildings”, *COST ACTION C26: Urban Habitat Constructions under Catastrophic Events - Proceedings of the Final Conference*, 619-624.

Formisano, A. and Mazzolani, F. (2012), “Progressive collapse and robustness of steel framed structures”, *Civil-Comp Proceedings*, **99**, Stirlingshire, Scotland.

Formisano, A., Landolfo, R. and Mazzolani, F.M. (2015), “Robustness assessment approaches for steel framed structures under catastrophic events”, *Comput. Struct.*, **147**, 216- 228.

Fu, F. (2013), “Dynamic response and robustness of tall buildings under blast loading”, *J. Constr. Steel Res.*, **80**, 299-307.

Gerasimidis, S. and Sideri, J. (2016), “A new partial-distributed damage method for progressive collapse analysis of steel frames”, *J. Constr. Steel Res.*, **119**, 233-245.

Guzas, E.L. and Earls, C.J. (2011), “Simulating blast effects on steel beam-column members”, *Appl. Comput. Struct.*, **89**, 2149-2161.

Kaveh, A. and Zakian, P. (2016), “An efficient seismic analysis of regular skeletal structures via graph product rules and canonical forms”, *Earthq. Struct.*, **10**, 25-51.

Kaveh, A., Aghakouchak, A. and Zakian, P. (2015), “Reduced record method for efficient time history dynamic analysis and optimal design”, *Earthq. Struct.*, **8**, 639-663.

Liew, J.Y.R. (2008), “Survivability of steel frame structures subject to blast and fire”, *J. Constr. Steel Res.*, **64**, 854-866.

Liew, J.Y.R. and Chen, H. (2004), “Explosion and fire analysis of steel frames using fiber element approach”, *J. Struct. Eng.*, **130**, 991-1000.

Loulelis, D., Hatzigeorgiou, G. and Beskos, D. (2012), “Moment resisting steel frames under repeated earthquakes”, *Earthq. Struct.*, **3**, 231-248.

Ma, G., Huang, X. and Li, J. (2009), “Simplified damage assessment method for buried structures against external blast load”, *J. Struct. Eng.*, **136**, 603-612.

Mahmoud, S. (2014), “Blast load induced response and the associated damage of buildings considering SSI”, *Earthq. Struct.*, **7**, 349-365.

Mohamed Ali, R.M. and Louca, L.A. (2008a), “Performance-based design of blast resistant offshore topsides, Part II: Modelling and design”, *J. Constr. Steel Res.*, **64**, 1046-1058.

Mohamed Ali, R.M. and Louca, L.A. (2008b), “Performance based design of blast resistant offshore topsides, Part I: Philosophy”, *J. Constr. Steel Res.*, **64**, 1030-1045.

Monir, H.S. (2013), “Flexible blast resistant steel structures by using unidirectional passive dampers”, *J. Constr. Steel Res.*, **90**, 98-107.

Quiel, S., Marjanishvili, S. and Katz, B. (2015), “Performance-based framework for quantifying structural resilience to blast-induced damage”, *J. Struct. Eng.*, **142**, C4015004.

Scawthorn, C. and Chen, W.F. (2002), *Earthquake Engineering Handbook*, CRC press.

Schwer, L. (2007), Optional Strain-Rate Forms for the Johnson Cook Constitutive Model and the Role of the Parameter Epsilon_0. LS-DYNA, Anwenderforum, Frankenthal.

Smith, P.D. and Hetherington, J.G. (1994), *Blast and Ballistic Loading of Structures*, Utterworth Heinemann.

Stoddart, E.P., Byfield, M.P., Davison, J.B. and Tyas, A. (2013), “Strain rate dependent component based connection modelling for use in non-linear dynamic progressive collapse analysis”, *Eng. Struct.*, **55**, 35-43.

Warn, G. and Bruneau, M. (2009), “Blast resistance of steel plate shear walls designed for seismic loading”, *J. Struct. Eng.*, **135**, 1222-1230.

CC