Innovative Infrastructure Solutions (2021) 6:223
https://doi.org/10.1007/s41062-021-00585-y

TECHNICAL PAPER

Dynamic performance of multi-storey buildings under surface blast: A case study

Venkata Sudha Ambavaram[1] · Anupama Muddarangappagari[1] · Anoosha Mekala[1] · Rajaram Chenna[1]

Received: 23 January 2021 / Accepted: 1 July 2021

Abstract

Blast loads create massive damage and reduce the structure's strength and stability, leading to internal and external damage. Due to this damage under extreme loads, efforts have been made to improve methods of structural analysis and design to resist blast loads. The research mainly focuses on the effect of surface blast on low-rise and high-rise buildings. In this study, five buildings with the same plan configuration are considered and designed according to Indian Standards. These buildings are modelled in SAP2000 software. The displacement responses are represented in terms of nonlinear time histories for different charge weights and standoff distances. Strengthening measures can be done to minimize the dynamic effect of the building due to blast loading. The responses significantly change at lower standoff distances for high-rise buildings and the same for standoff distances greater than 10 m. No significant effect is observed on the response of building between single and double bracings.

Keywords Blast load · Standoff distance · Charge weight · Displacement response

Introduction

Nowadays, many countries are facing a severe problem, i.e. terrorism. These threats are becoming more common and are very difficult to identify. These are man-made disasters or tragic accidents, and they can range from household gas explosion to nuclear explosion. Man-made hazards are different from natural hazards such as earthquakes, floods, and hurricanes. The major terrorist attacks are with the explosion, or impact or both. A bomb explosion within or around a building can lead to severe building damages and depends on building layout, the material used, charge weight, and standoff distance. The damage of the structure can be reduced if adequately designed for the extreme dynamic loads. On the other hand, retrofitting the buildings can be done to ensure the safety of existing buildings against such events. The main part of the casualties in the explosions is due to structural damage.

After the progressive collapse of the Ronan Point apartment, England, in 1968, structural engineering research departments worldwide forced to direct their research towards the progressive failure of high-rise buildings under abnormal loads. So many guidelines, standards, and criteria were developed by different government and private agencies in the world [1–4]. These guidelines and criteria would not have been efficient to prevent the progressive collapse in recent terrorist attacks if the target buildings were designed and built for the above guidelines and standards. It is highly expensive to design each element of high-rise buildings to withstand an unexpected abnormal load. These guidelines and criteria may help to prevent progressive collapse for expected, possible and probable threats on structures. The structures designed for regular design codes can withstand to some extent of extreme loads and then lead to collapse. Over 2000 fatalities happened due to bomb blasts in the last two decades in India. From Fig. 1, more casualties are observed due to notable bomb blasts that occurred in the 2002 Akshardham temple (Gujarat) blast, 2006 Mumbai train blast, and 2008 Hyderabad blast.

A numerical study was done on the internal explosion of a reinforced concrete (RC) building [6]. A new method was proposed to evaluate the blast response of the building. However, the proposed method was time-consuming to

✉ Rajaram Chenna
drchenna78@gmail.com

1 Department of Civil Engineering, Rajeev Gandhi Memorial College of Engineering and Technology, Nandyal 518 501, India

Published online: 24 July 2021

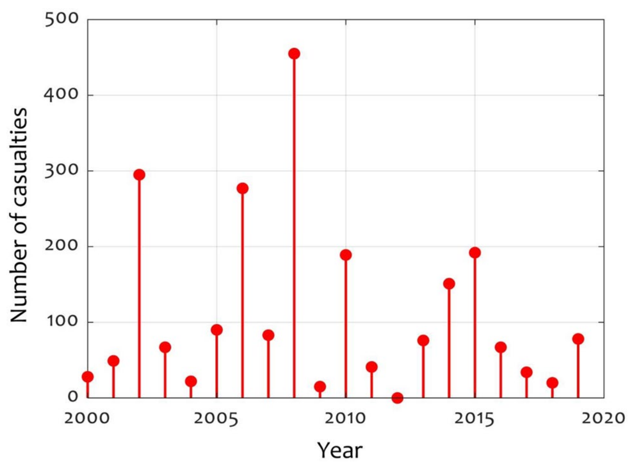


Fig. 1 Statistics of casualties in India due to bomb blasts since 2000 (Source: National Consortium for the Study of Terrorism and Responses to Terrorism, Global Terrorism Database, Univ. of Maryland [5]) *Note: The foundation is assumed to be fixed at ground surface level*

calculate the response of the multi-storeyed building. Ductile detailing of framed members reduces displacement response due to extreme loadings such as blast and progressive collapse [7]. Interaction diagrams were developed between fire and explosion for steel-framed structures. The study was not concentrated on concrete-encased steel columns, as these columns are generally used in multi-storeyed structures [8]. A two-step approach was presented to the building to identify local damage elements due to blast loading [9]. A two-dimensional RC frame was analysed using nonlinear dynamics analysis software LS-DYNA to study the response due to blast loading over the building's height. It was concluded that the blast pressure would negatively affect the lower floors of the building proportionate to the charge weight and standoff distance [10].

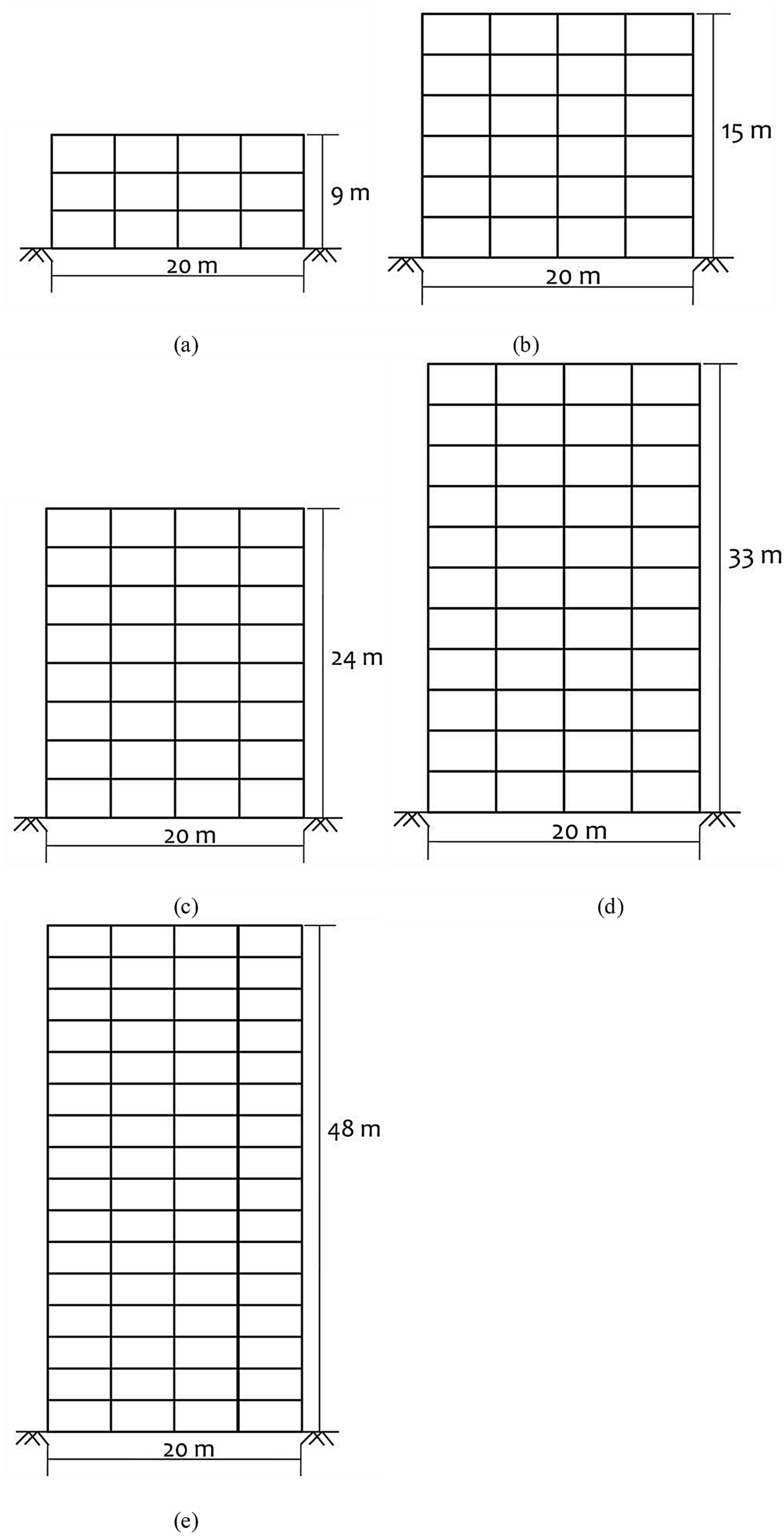
Nonlinear dynamic analysis was done on a steel building. The study investigated that modelling parameters affect the progressive collapse of the building [11]. A new method was developed considering both the nonzero initial condition and existing damage in structural members for the progressive collapse of RC frames [12]. A study was conducted on the structural collapse behaviour using the applied element method (AEM). Different structures have been analysed for earthquake ground motion and blast loading [13]. Various empirical equations were reviewed to compute the blast wave parameter proposed by several researchers [14]. The nature of blast load is different from seismic loads. A comparative study has been done between earthquake loads and blast loads on a ten-storeyed building. It was concluded that the inter-storey drifts resulted from blast loads were significantly higher than earthquake loads [15]. A building subjected to blast load was studied due to the removal of columns and the position of shear walls [16]. A methodology was improved to determine the probabilistic damage of steel columns due to blast loads. It was found that fixed supports have severe damage compared to pinned supports, considering flexural mode [17]. The effect of masonry infill walls on the response of RC frames was studied under progressive scenario [18]. A parametric study was done to find out the blast responses of different buildings. It was concluded that the blast responses are governed by the air pressure effect and ground shock effect for low-rise buildings and high-rise buildings, respectively [19]. Retrofitting techniques using FRP and polyurea have been proved efficient in blast mitigation [20]. A study has been done on structural wall of structure to mitigate blast effect. It was observed that increasing the thickness of the walls decreases the impulse applied on four sides of the wall. Reinforcing the walls has an insignificant effect on the deflection of the walls [21].

Very few studies have been carried out on the effect of a surface blast on structures. The dynamic effects of an RC building due to various blast loads have not been addressed for retrofitting buildings, which can minimize the damage of the building. The present study focuses on the above problem and considers $G+2$, $G+5$, $G+7$, $G+10$, and $G+15$ storeyed building subjected to surface blasting with a charge weight of 10 kg, 20 kg, and 30 kg, where 'G' stands for ground level, and 'number' denotes the number of storeys above ground level. These charge weights are placed at a

Table 1 Building properties

Parameter	G + 2 building	G + 5 building	G + 7 building	G + 10 building	G + 15 building
Material properties					
Grade of concrete	M_{25}	M_{25}	M_{25}	M_{25}	M_{25}
Grade of steel	Fe_{415}	Fe_{415}	Fe_{415}	Fe_{415}	Fe_{415}
Poisson's ratio (υ)	0.30	0.30	0.30	0.30	0.30
Geometry properties					
Size of beam (mm)	250 × 400	250 × 400	250 × 400	250 × 400	250 × 400
Size of column (mm)	400 × 400	400 × 400	400 × 400	400 × 400	400 × 400
Thickness of slab (mm)	150	150	150	150	150
Dynamic properties					
Fundamental period (s)	0.40	0.82	1.10	1.54	2.27

Fig. 2 Elevation of G + 2, G + 5, G + 7, G + 10, and G + 15 storeyed buildings used in the analysis. Note: The foundation is assumed to be fixed at ground surface level



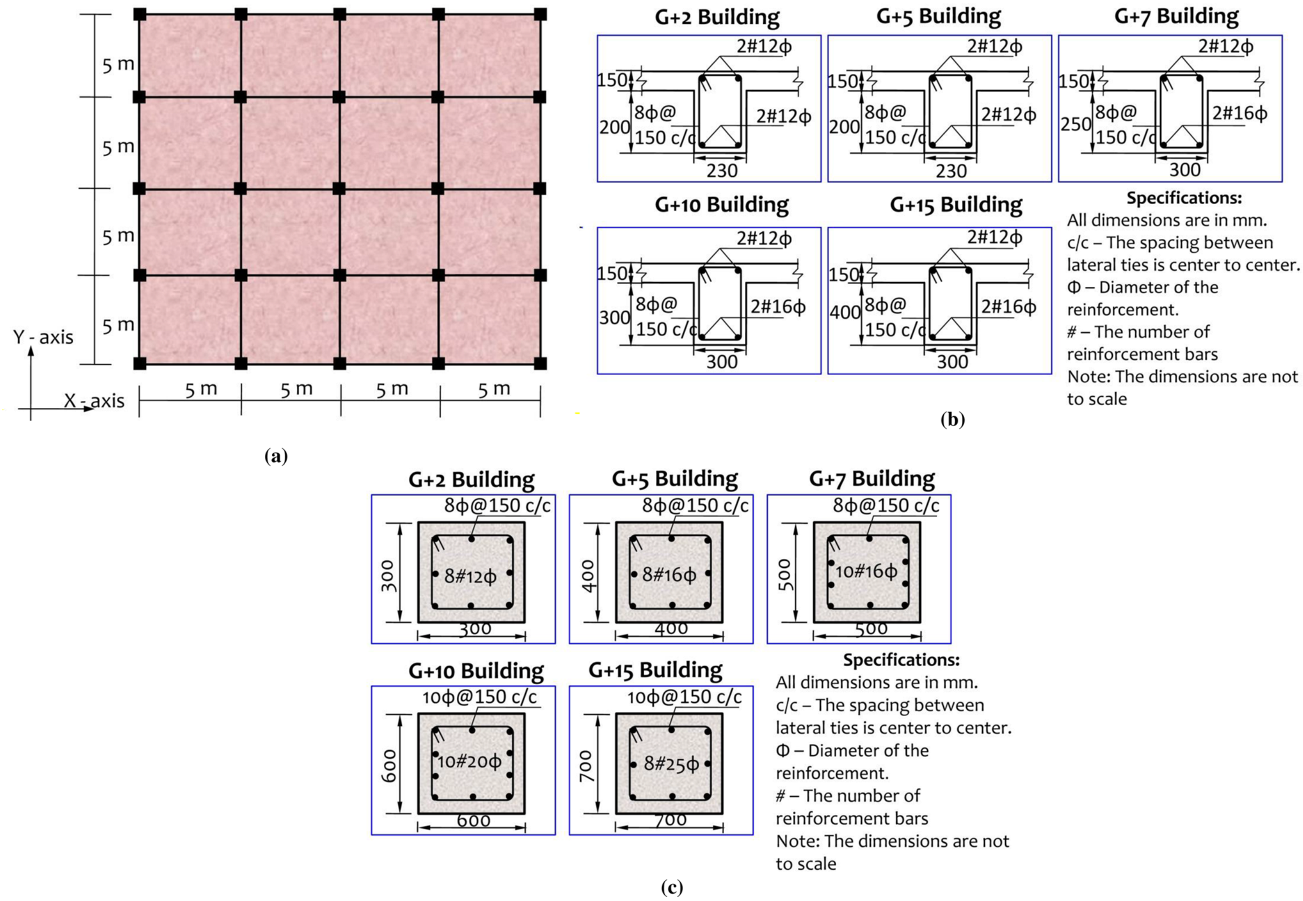


Fig. 3 Building details considered in the analysis **a** plan, **b** column reinforcement detailing, and **c** beam reinforcement detailing

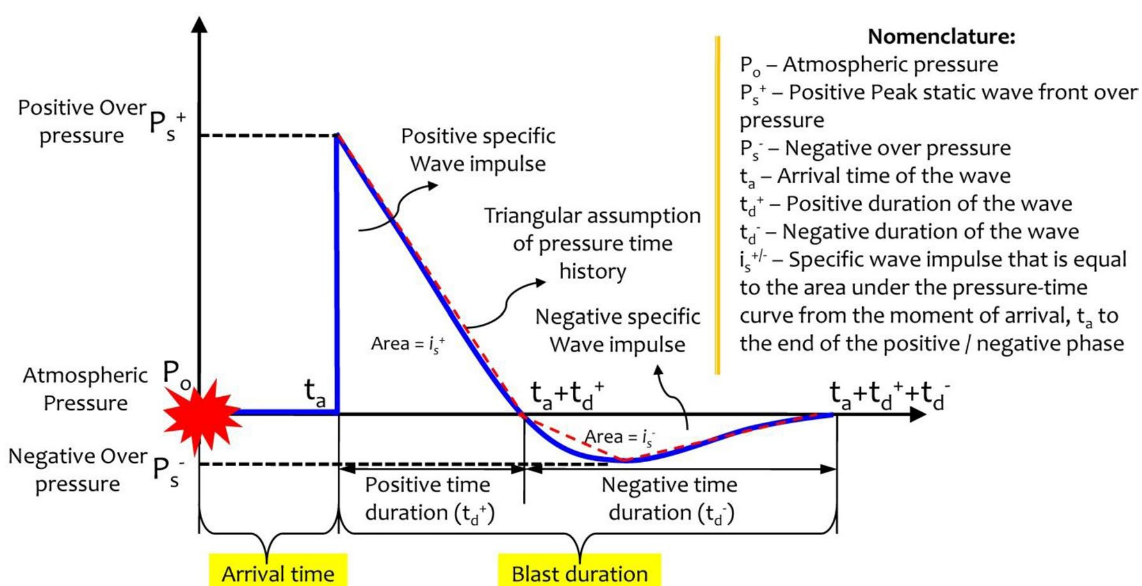


Fig. 4 Blast characteristics: overpressure time history with critical blast parameters

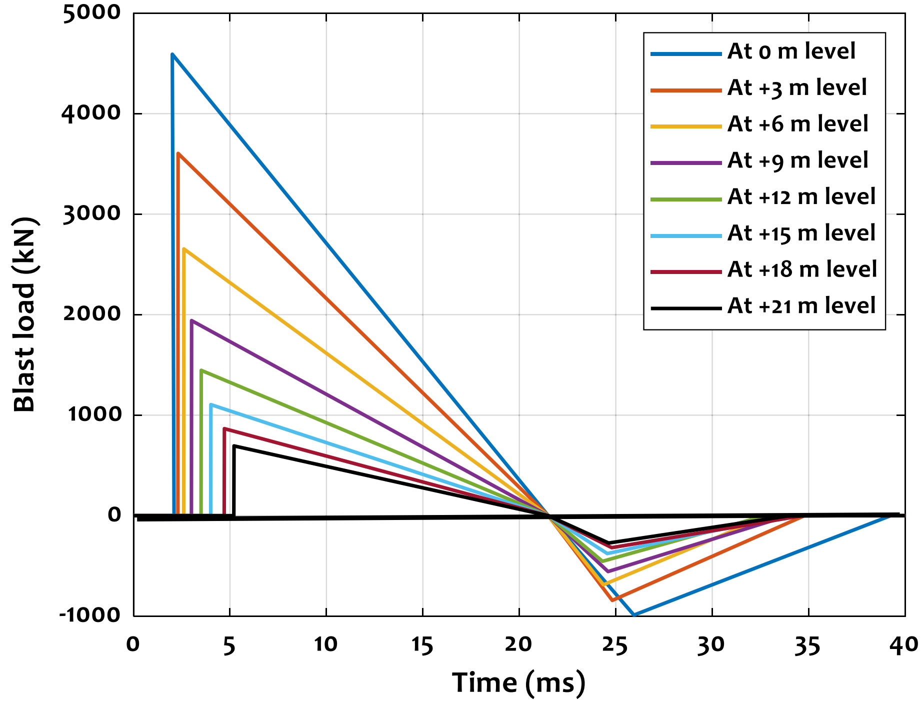


Fig. 5 Blast force vs time plots for a typical G+7 storeyed building located at 10 m standoff distance subjected to a charge weight of 10 kg

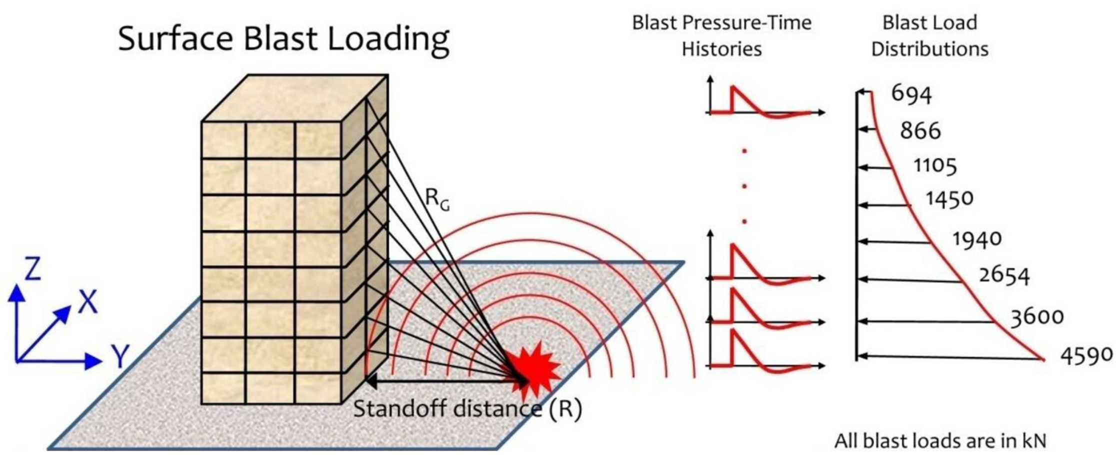


Fig. 6 Blast loads that act on each storey of G+7 storey building for various charge weights and standoff distances

standoff distance ranges from 10 to 50 m with an interval of 10 m. The effect of standoff distance and charge weight over the height of building cases is expressed in maximum displacements, inter-storey drifts, and base shear. Later, the same analysis is carried out for a *G*+7 storeyed building with a charge weight of 20 kg placed at a distance of 20 m using strengthening techniques such as jacketing of columns and bracings.

Numerical modelling of building

Buildings considered in this study are subjected to three different charge weights ranging from 10 to 30 kg placed at different standoff distances ranging from 10 to 50 m. A 20 m×20 m building plan dimension with four bays of equal dimension and a storey height of 3 m is considered for analysis. The live load acting on the building is assumed to be

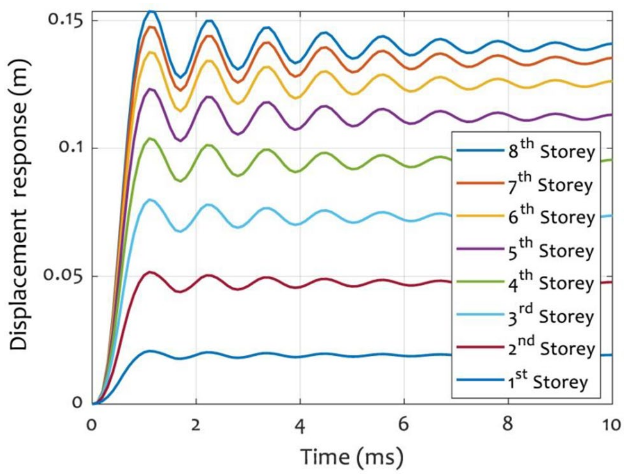


Fig. 7 Displacement time histories of G+7 storey building along Y direction subjected to blast of 10 kg and located at 50 m standoff distance

3 kN/m^2 and designed as per Indian Standards [22]. The geometry, material, and dynamic properties of buildings are shown in Table 1. The buildings are modelled in the finite element method (FEM)-based software SAP2000 [23]. The building's geometry details and reinforcement details are shown in Figs. 2 and 3. The foundation reinforcement details are not considered and are assumed to be fixed at ground surface level. A step-by-step procedure for estimating surface blast loading that acts on a regular building is summarized in the next section.

Estimation of surface blast loading

A building of width w, breadth B, and height H is located at standoff distance R measured from the centre of charge to the object, and it is subjected to charge weight W (more precisely mass of explosion expressed in kg); the scaled

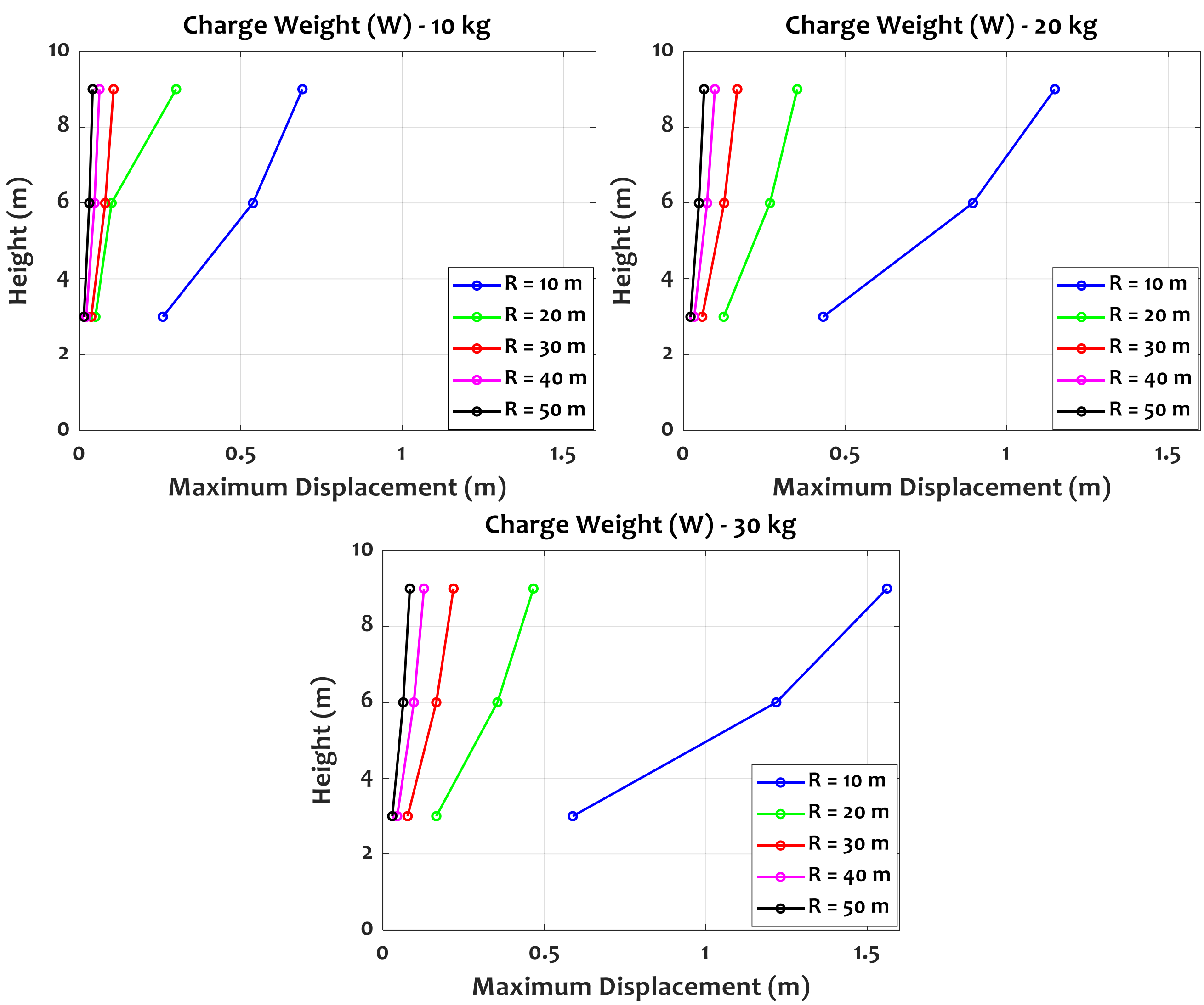


Fig. 8 Maximum displacements at each storey level of G+2 storey building under different charge weights and standoff distances along Y direction

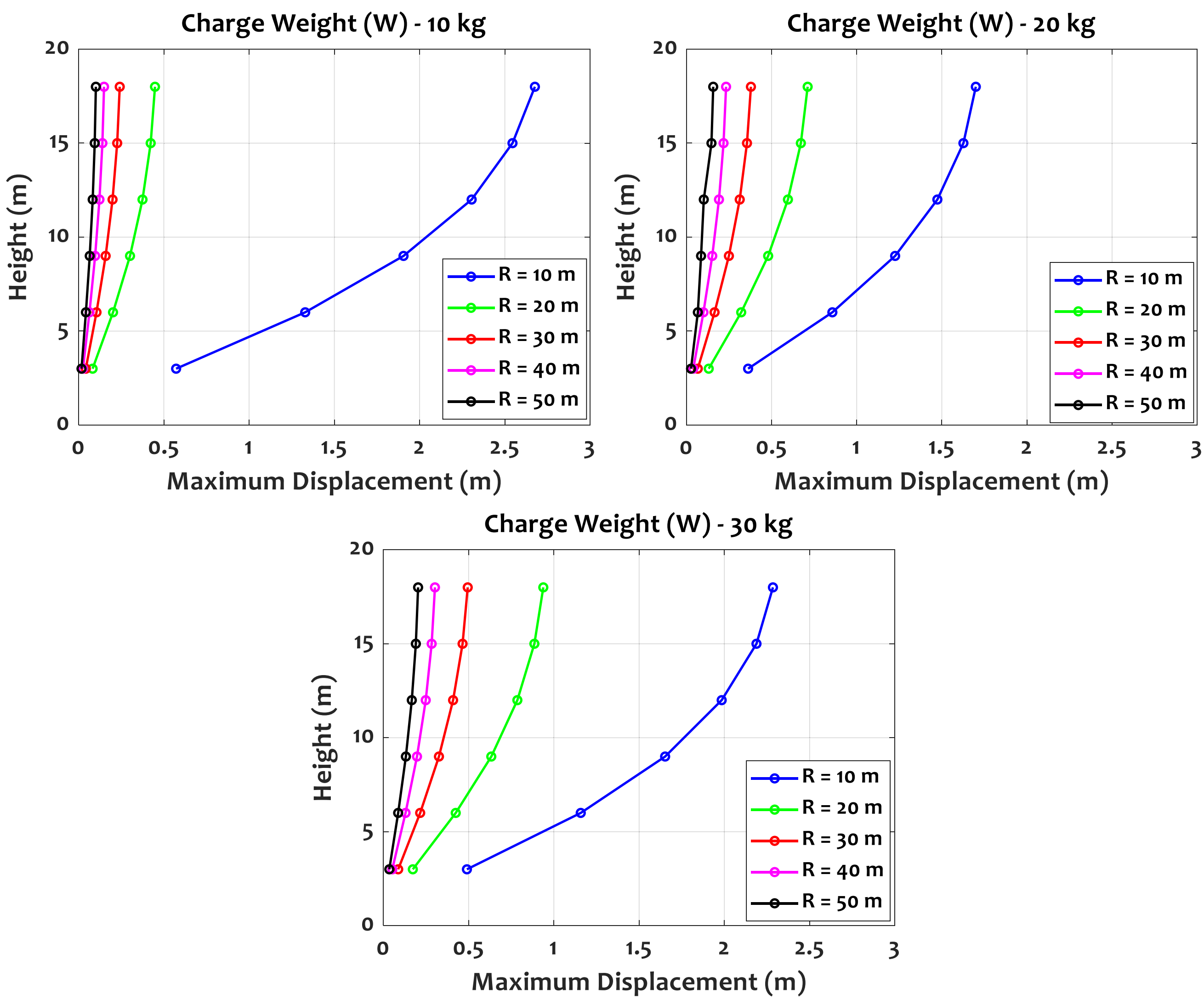


Fig. 9 Maximum displacements at each storey level of G + 5 storey building under different charge weights and standoff distances along Y direction

distance Z is calculated from Eq. 1 and is expressed in m/kg$^{0.33}$ as [24],

$$Z = \frac{R}{W^{(0.333)}} \tag{1}$$

The charge weights are taken as 10 kg, 20 kg, and 30 kg in this analysis. For surface blasts, the equation for the peak overpressure P_s^+, initially proposed by Newmark and Hansen in 1961 [25]. Later Kadid et al., 2012; Vijayaraghavan et al., 2012 modified the analysis [26, 27]. The P_s^+ is calculated as,

$$P_s^+ = 0.6784\frac{W}{R^3} + 0.294\sqrt{\frac{W}{R^3}} \tag{2}$$

The peak overpressure has the unit of megapascals (MPa). On the other hand, the P_s^+ can obtain from unified facilities

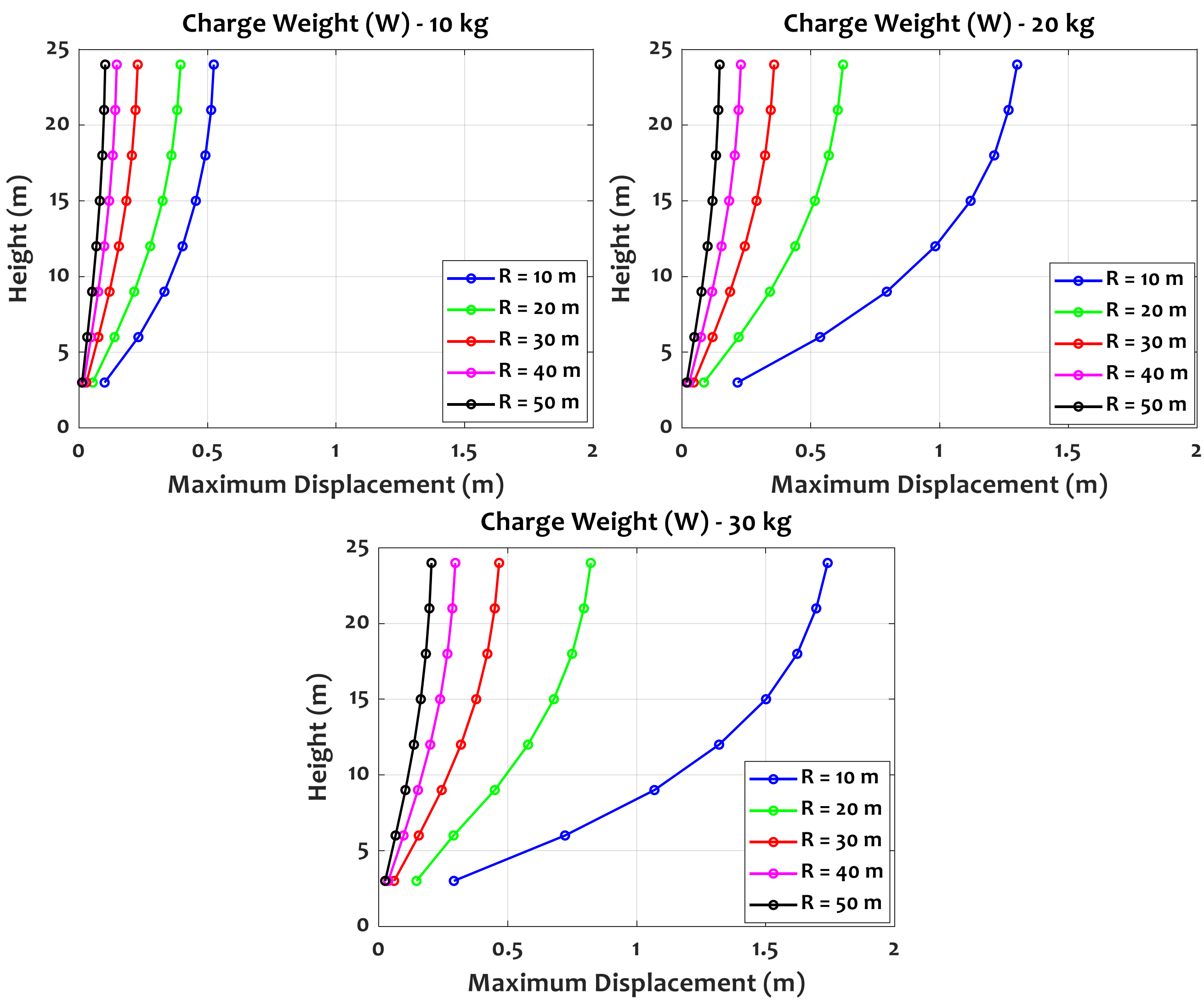


Fig. 10 Maximum displacements at each storey level of G+7 storey building under different charge weights and standoff distances along Y direction

criteria [28]. The coefficient of reflected overpressure (C_r), reflected overpressure (P_r), and arrival time of the wave (t_a) are calculated as follows [29–31].

$$C_r = 0.3P_s^{+(0.25)} \tag{3}$$

$$P_r = C_r P_s^+ \tag{4}$$

$$t_a = \frac{8.534}{a_o} Z^{-0.996} \tag{5}$$

The units for P_s^+, P_r are in MPa. For a surface burst, t_a is expressed in milliseconds (ms). The parameter a_o is the speed of sound in air 340 m/sec.

The negative pressure P_s^- (in kPa) located below atmospheric pressure in the negative phase of the blast is given by,

$$P_s^- = -\frac{35}{Z}; Z > 1.6 \tag{6}$$

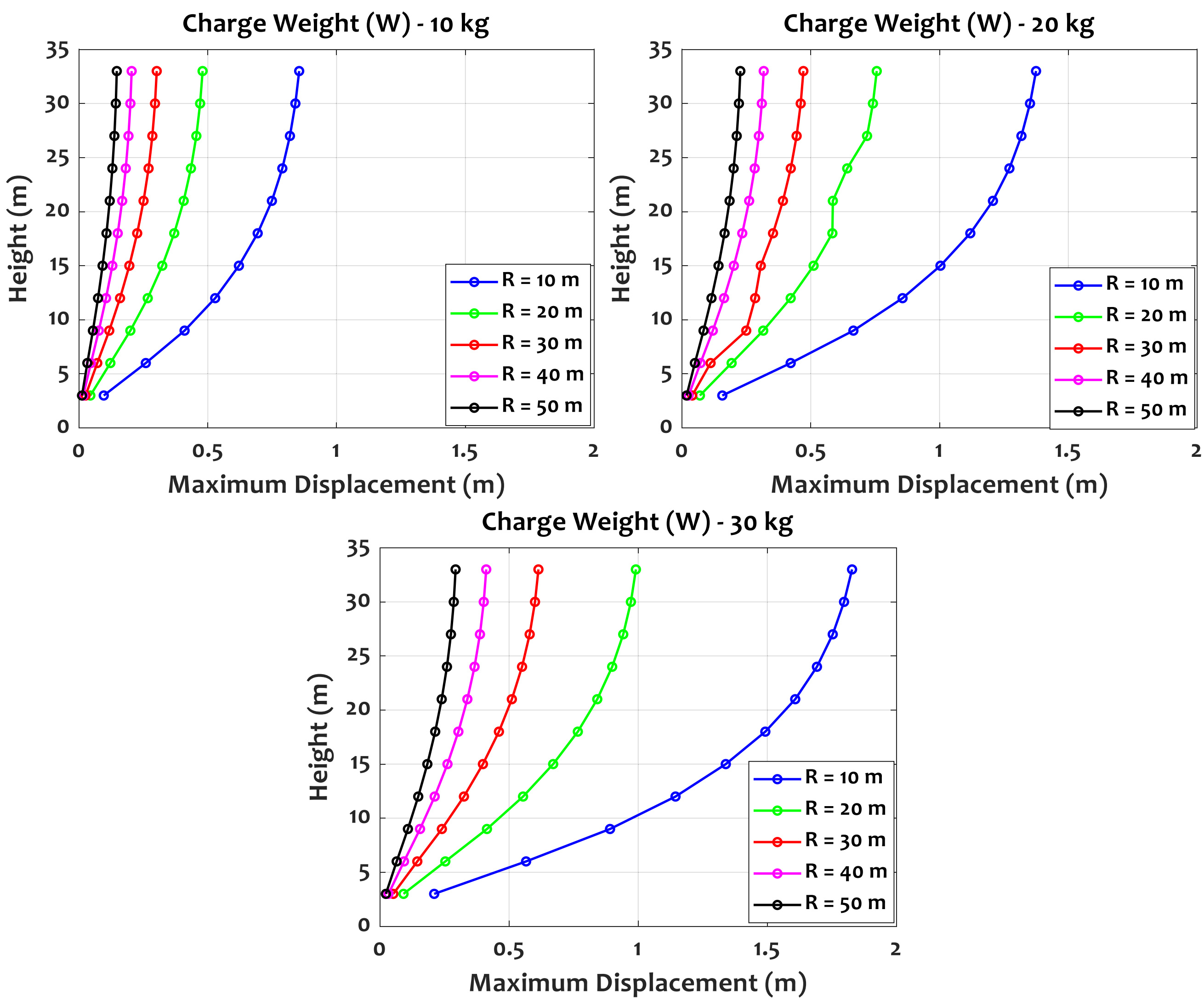


Fig. 11 Maximum displacements at each storey level of G + 10 storey building under different charge weights and standoff distances along Y direction

The initial pressure reduction can be assumed as a triangular pressure impulse in this analysis. The positive and negative phase duration for the surface blast is expressed as [25],

$$t_d^+ = 10W^{(1/3)}; t_d^- = \frac{2i_s^-}{P_s^-} \tag{7}$$

where i_s^- is the total negative impulse that is equal to the area under the negative pressure–time curve. The total positive and negative phase duration of blast overpressure is expressed in milliseconds (ms). The negative peak pressure is considered as 0.25 times of t_d^-. A schematic diagram representing blast parameters is shown in Fig. 4.

The parameters P_s^+, P_s^-, P_r, t_a, t_d^+, and t_d^- are calculated from either curve suggested by unified facilities criteria or from Eqs. 2, 3, 4, 5, 6, and 7. The above parameters are calculated for all cases considered in this analysis are shown in

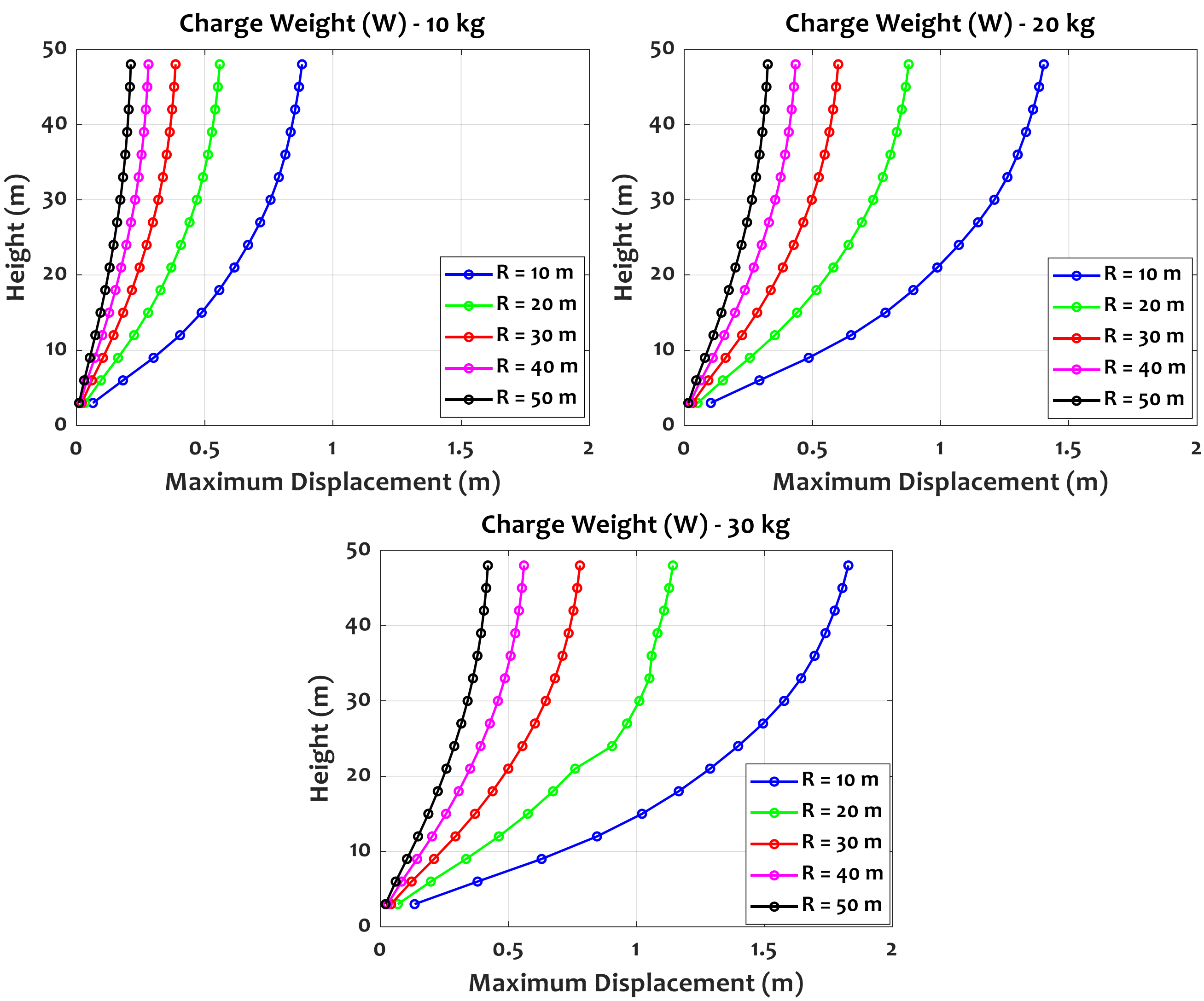


Fig. 12 Maximum displacements at each storey level of G+15 storey building under different charge weights and standoff distances along Y direction

Appendix. The positive and negative pressure time histories are plotted against time. The blast load is calculated as the product of blast pressure and a portion of the area where the blast is subjected to. It is applied at each storey level to get the displacement response of the building. The blast load time histories against time for a $G+7$ storey building as shown in Fig. 5, and peak values are plotted as shown in Fig. 6.

Discussion

A nonlinear dynamic analysis is done for the buildings mentioned above with the variation of standoff distances and charge weights. The results are discussed in terms of maximum displacement response, inter-storey drift, and base shear of each building case.

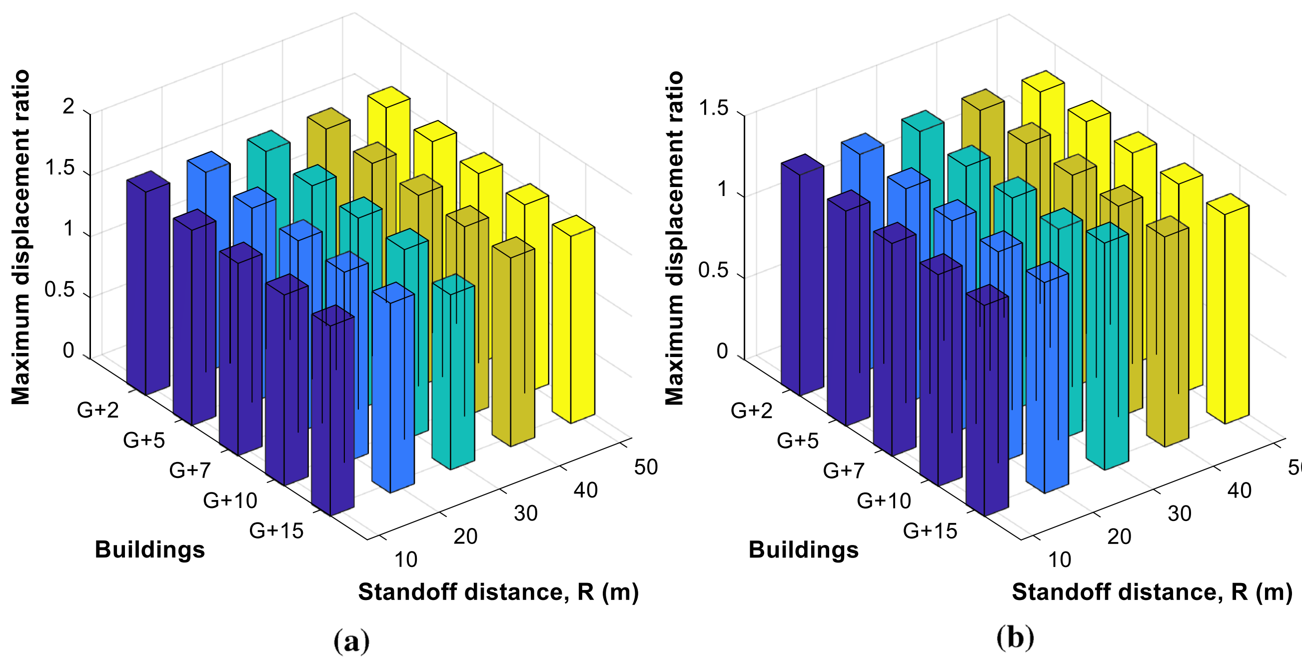


Fig. 13 Maximum displacement ratios of each building case with respect to charge weight 10 kg. **a** Charge weight W—20 kg and (b) charge weight W—30 kg

Effect of charge weight

Figure 7 represents the displacement response of $G+7$ storey building along Y direction subjected to blast of 10 kg and located at 50 m standoff distance. A plot of maximum displacement response of buildings with a variation of standoff distance due to different charge weights can be seen from Figs. 8, 9, 10, 11, and 12. It is observed that the pattern of maximum displacement responses is similar to that of charge weight. As the charge weight of blast increases, the displacement responses of the building increase. On the other hand, it decreases with standoff distance. A three-dimensional plot is drawn between maximum displacement ratio and standoff distance shown in Fig. 13. It is defined as the ratio between maximum displacement of building subjected to charge weight W—20 kg or W—30 kg and W—10 kg. At lower standoff distance, the maximum displacement ratio amplifies 1.5 and 1.523 times that of 20 kg and 30 kg charge weights, respectively. At higher standoff distance, the displacement ratio amplifies 1.5 times that of 20 kg and 30 kg charge wei ghts.

Similarly, the responses significantly reduce as the standoff distance increases. The maximum displacement at the top floor of a $G+7$ building subjected to 10 kg charge weight of blast is around 1.5, and 1.52 times that of 20 kg, and 30 kg charge weights of the same building at lower standoff distance. Figures 14, 15, 16, 17, and 18 show the inter-storey drift of all buildings subjected to blast load with various standoff distances. Figure 19 presents the base shear of all buildings subjected to different blast loads and standoff distances. For $G+2$, $G+5$, and $G+7$ storey buildings located at $R=10$ m, the base shear is increased to 1.7 times and 2.4 times with charge weight from 10 to 20 kg and 30 kg, respectively. For the same buildings, it is increased to 1.4 times and 2.0 times at $R=50$ m. A linear decrease in base shear is observed with an increase in charge weight.

Effect of standoff distance

Figure 13 represents the maximum displacement ratios of each building case considering charge weight 10 kg. The

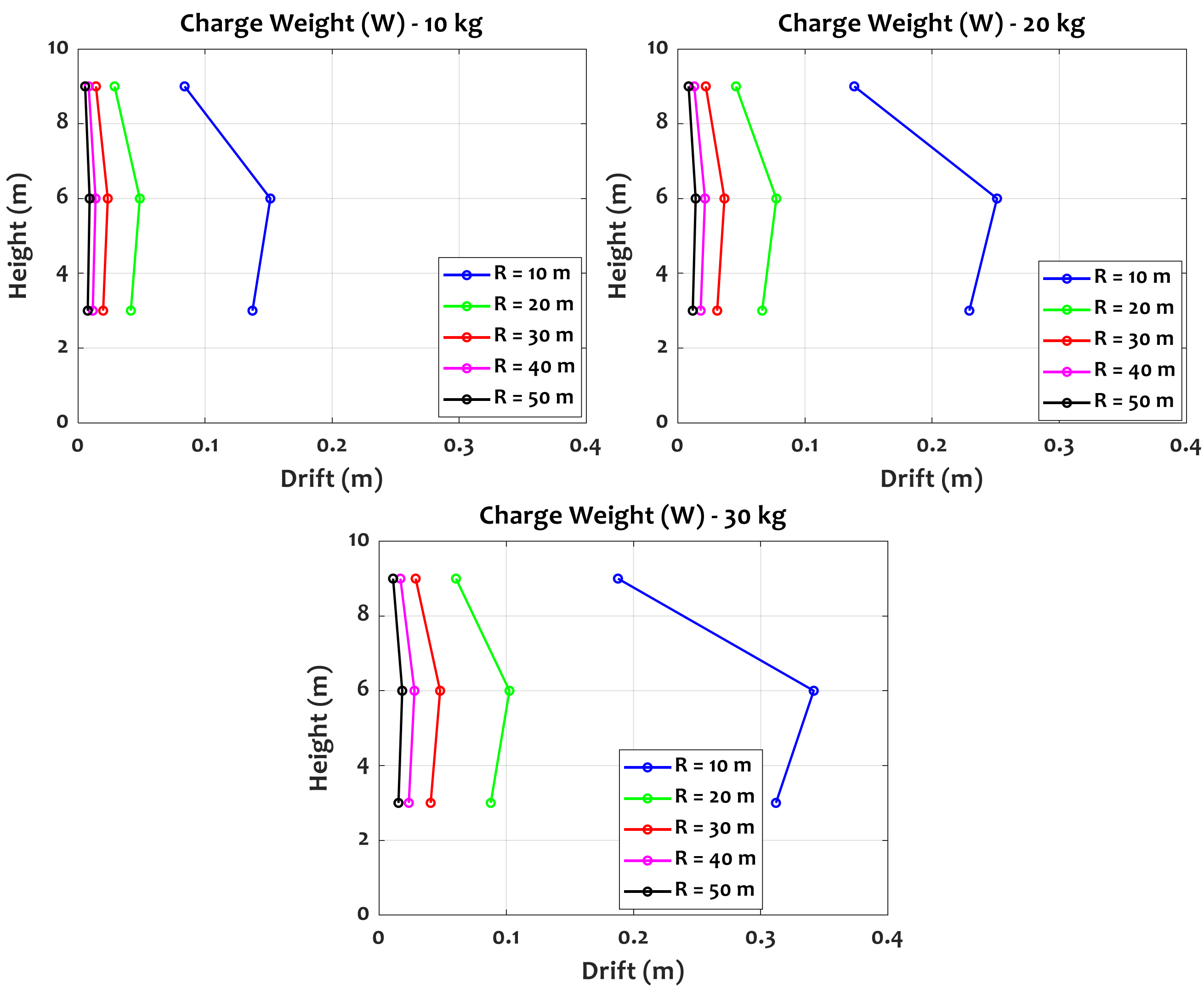


Fig. 14 Inter-storey drifts of G + 2 storey building under different charge weights and standoff distances

maximum displacement at the top floor of any building case subjected to 10 kg charge weight of blast is around 1.5, and 2.5 times that of 20 kg, and 30 kg charge weights of the same building at 10 m as standoff distance. A sharp decrease in maximum displacement response is observed with the standoff distance in the vicinity of the building. After that, a slight reduction in response is observed with an increase in standoff distance. The inter-storey drift also follows a similar trend as that of maximum displacement response. As the building height increases, the top displacement of the building at $R = 10$ m is twice at $R = 20$ m. The top displacement is significantly more at the proximity of blast, and it reduces as the standoff distance increases. The base shear also follows a similar trend as that of maximum displacement with standoff distance.

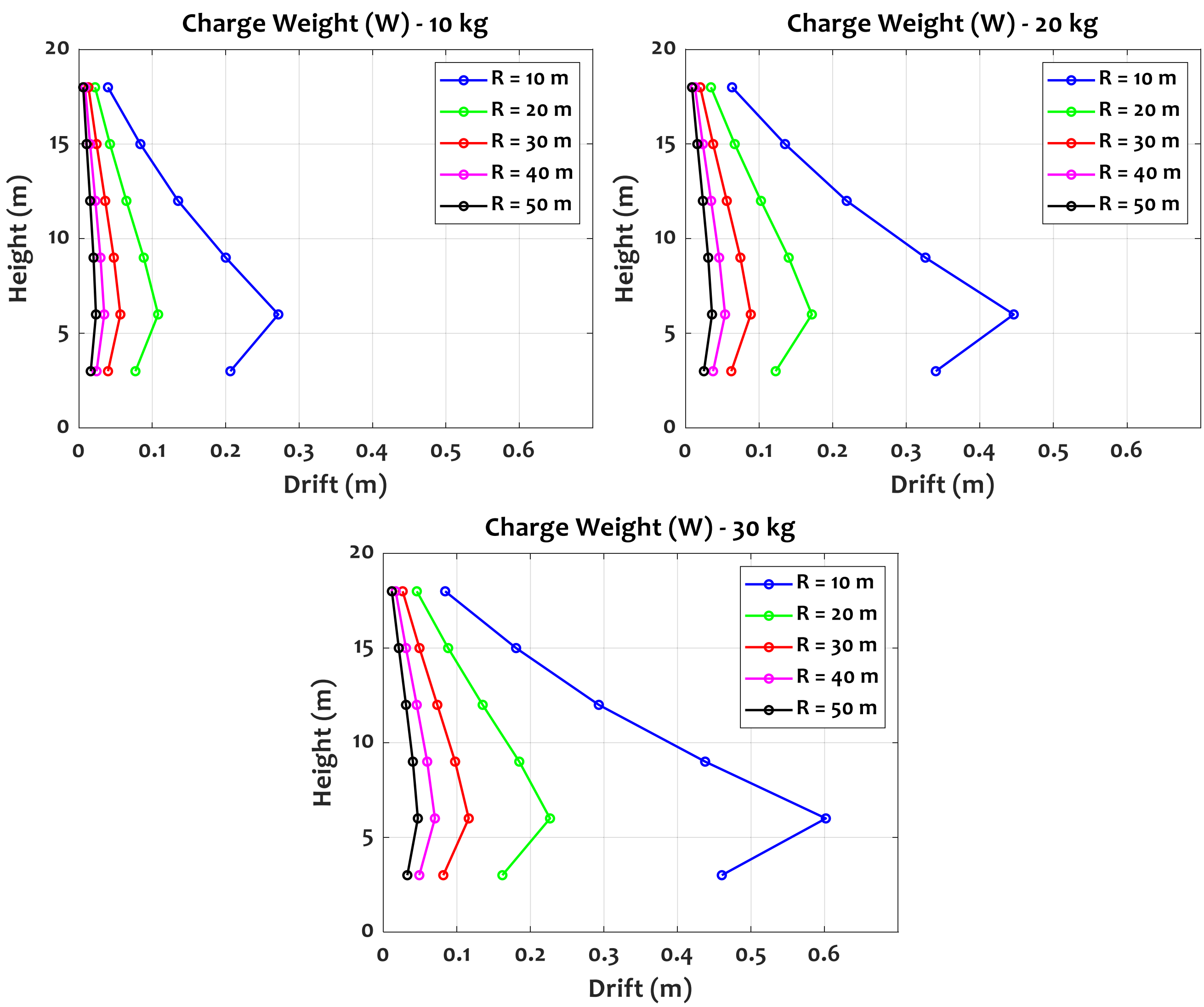


Fig. 15 Inter-storey drifts of G + 5 storey building under different charge weights and standoff distances

Effect of positive and negative wave pressure

A plot has been drawn on the maximum displacement response between positive wave pressure and negative wave pressure for a $G+7$ storey building in Fig. 20. It is observed that negative wave pressure has a minimum effect on the peak response at lower storey levels. A significant effect is observed at higher storey levels when compared with positive wave pressure. Hence, it is suggested to include negative wave pressure to calculate blast load for high-rise buildings.

Validation

The displacement time history responses of a building are validated with AEM. The objective of the subsection is to check the correctness of displacement time history results of the building from FEM and AEM due to blast loads. For this purpose, a single bay single-storey frame is selected subjected to 500 kg TNT charge weight and located at 10 m standoff distance. The building is modelled in FEM-based software SAP2000. Harinath Babu modelled the building

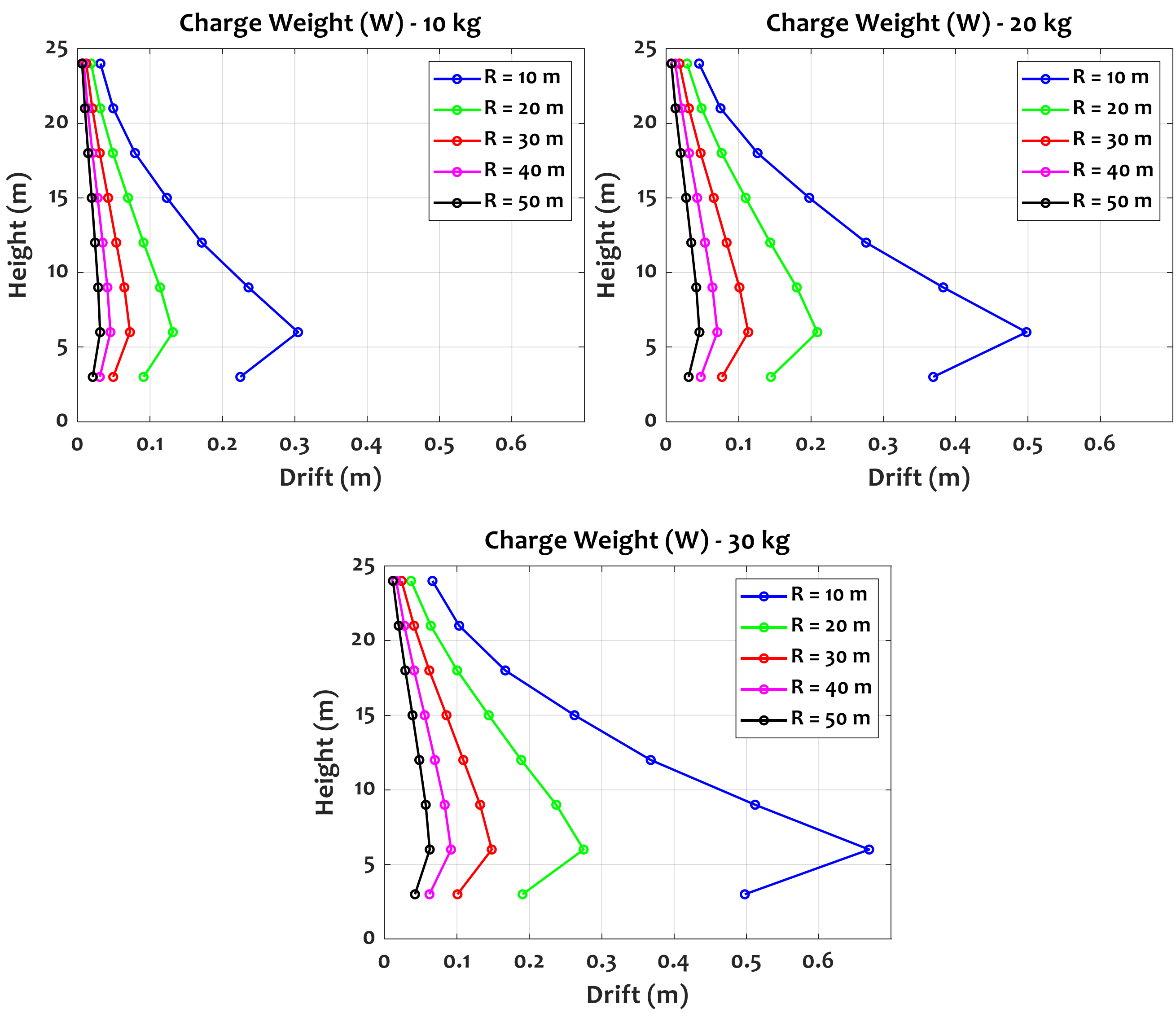


Fig. 16 Inter-storey drifts of G + 7 storey building under different charge weights and standoff distances

frame in AEM to understand the behaviour of frame subjected to blast load [13]. A summary of AEM is described and is as follows. AEM is a numerical method and capable of following structural behaviour starting from linear static analysis, crack propagation in nonlinear analysis, separation of elements in contact analysis, and even collapse of structure [32]. In this method, the adjacent elements are connected with normal and shear springs representing internal deformations, stresses, and strains. It is assumed that the motion of elements is like a rigid body motion.

The blast loads are calculated according to Technical Manual 5–1300 by United States of America, Department of Army [33]. The blast loads obtained from calculations are 916 kN and 683 kN that act on the ground floor and at + 3 m levels. A comparison is made between displacement time histories of the frame using FEM and AEM. The displacement time histories from both analyses are shown in Fig. 21.

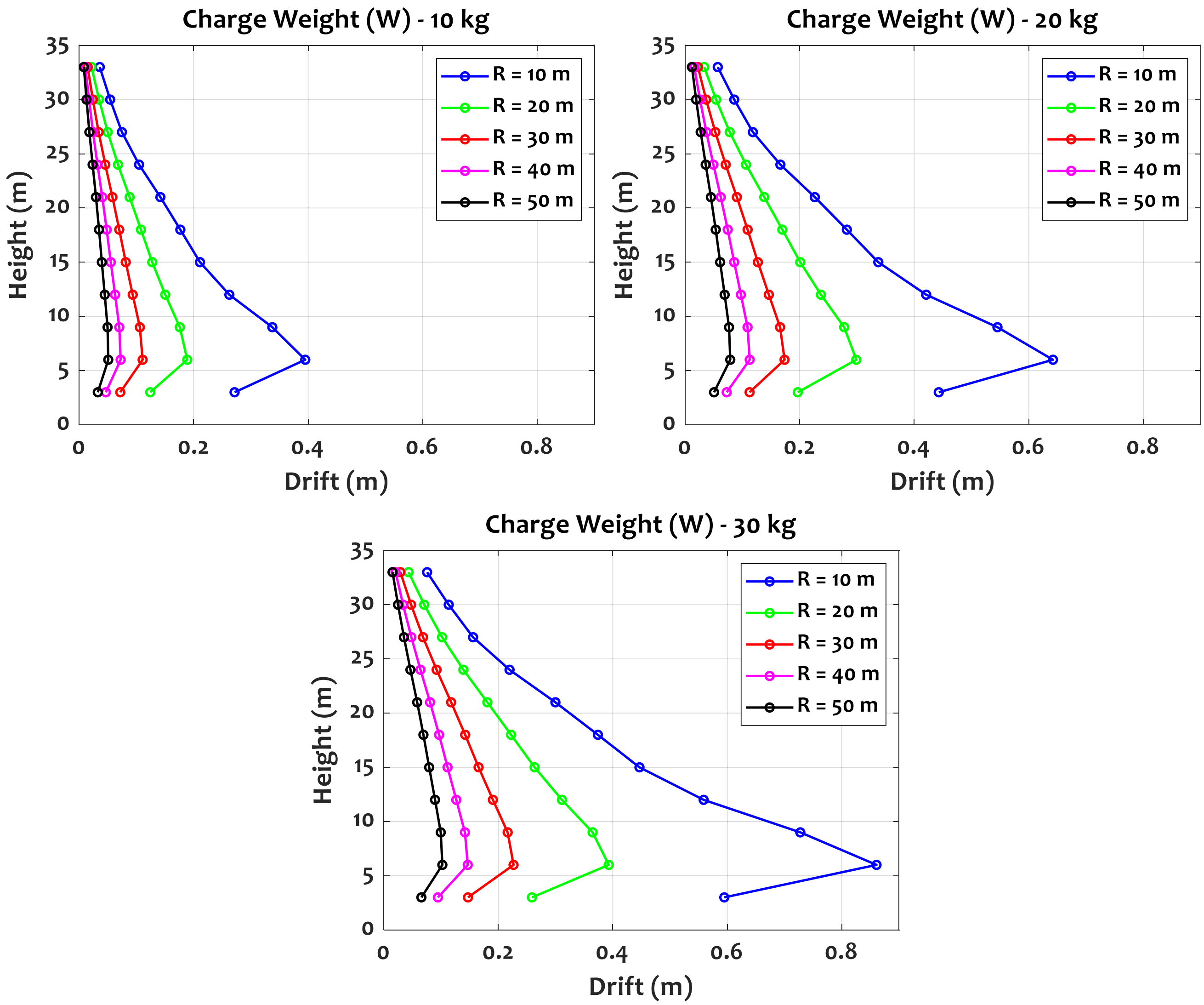


Fig. 17 Inter-storey drifts of G + 10 storey building under different charge weights and standoff distances

It is observed that the maximum displacement of the building obtained from FEM and AEM is 0.173 m and 0.142 m, respectively. The nonlinear material models used in FEM and AEM are the Takeda model and Okamura–Maekawa model [34]. The same displacement response is observed from both analyses till 1.53 s. After that, the displacement response of frame changes due to different material models used from both analyses.

Strengthening of building

Strengthening of buildings can be done to reduce the displacement response of building due to blast loads. It is defined as reconstructing any part of the building to achieve the higher strength of an existing building. Strengthening can be done in two ways, i.e. local and global. The current analysis considers two techniques to improve lateral resistance capacity of the building, namely, i) jacketing and ii) providing bracings [35]. These bracings are provided along

Y-axis as shown in Fig. 22. In this analysis, a $G+7$ storey building located at 20 m standoff distance subjected to 20 kg charge weight. The effect of jacketing and bracings on the overall response of building due to blast load is described below.

Effect of jacketing

Jacketing of columns improves axial and shear strength of column, and it is done through an increase in column cross section. The analysis considers various cross sections ranging from 0.4 m×0.4 m to 0.6 m×0.6 m with an interval of 0.1 m×0.1 m. These cross sections are denoted as 0.4LJ, 0.5LJ, and 0.6LJ, and LJ stands for Local Jacketing. The blast loads are applied along the Y-axis (perpendicular to the wall direction), as shown in Fig. 22. Maximum top displacement of 1.235 m, 0.877 m, 0.747 m are observed with an increase in column cross section. The top displacements of 0.4LJ are reduced to 40.8% and 65.3% for 0.5LJ and 0.6LJ.

Fig. 19 Base shear of building cases: **a** G+2 storey, **b** G+5 storey, **c** G+7 storey, **d** G+10 storey, and **e** G+15 storey under different charge weights and standoff distances

Effect of addition of bracings

The analysis considers single and double diagonal bracings at the bottom storey of a $G+7$ storeyed building. Various cross sections of bracings are used, ranging from 0.1 m×0.1 m to 0.3 m×0.3 m at an interval of 0.1 m. The displacement responses are calculated at each floor, and the responses are compared with the open ground storey. A maximum top displacement of 1.297 m is observed for open storey building and building with bracings, as shown

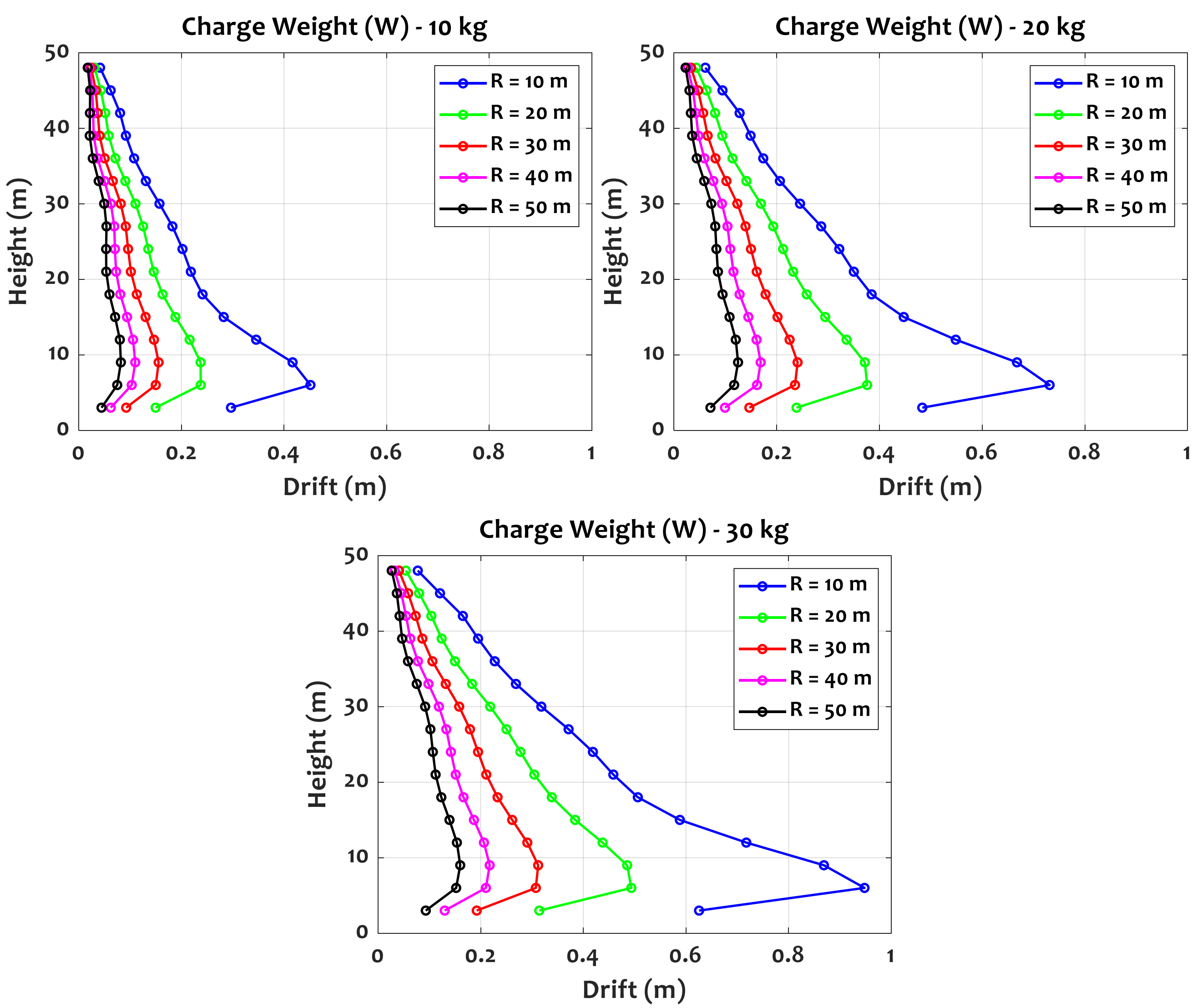
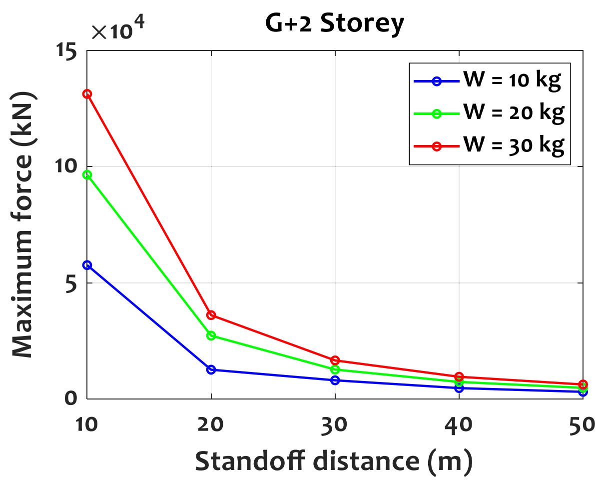
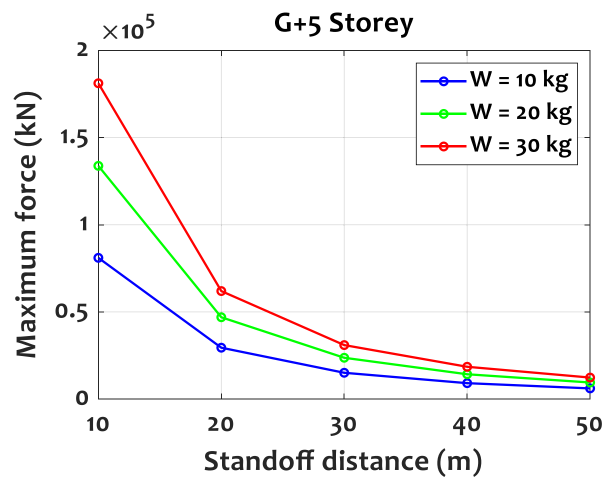


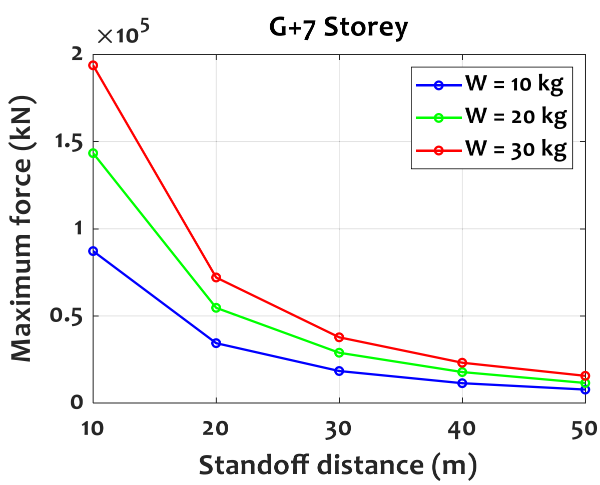
Fig. 18 Inter-storey drifts of G+15 storey building under different charge weights and standoff distances


G+2 Storey
×10^4
Maximum force (kN)
Standoff distance (m)
W = 10 kg
W = 20 kg
W = 30 kg
0
5
10
15
10
20
30
40
50

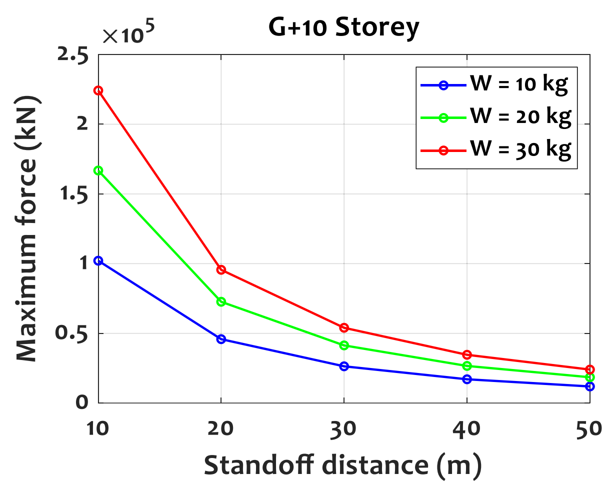
(a)


G+5 Storey
×10^5
Maximum force (kN)
Standoff distance (m)
W = 10 kg
W = 20 kg
W = 30 kg
0
0.5
1
1.5
2
10
20
30
40
50

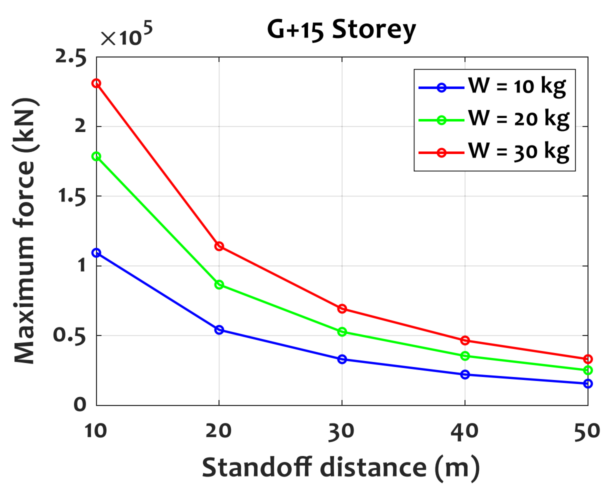
(b)


G+7 Storey
×10^5
Maximum force (kN)
Standoff distance (m)
W = 10 kg
W = 20 kg
W = 30 kg
0
0.5
1
1.5
2
10
20
30
40
50

(c)


G+10 Storey
×10^5
Maximum force (kN)
Standoff distance (m)
W = 10 kg
W = 20 kg
W = 30 kg
0
0.5
1
1.5
2
2.5
10
20
30
40
50

(d)


G+15 Storey
×10^5
Maximum force (kN)
Standoff distance (m)
W = 10 kg
W = 20 kg
W = 30 kg
0
0.5
1
1.5
2
2.5
10
20
30
40
50

(e)

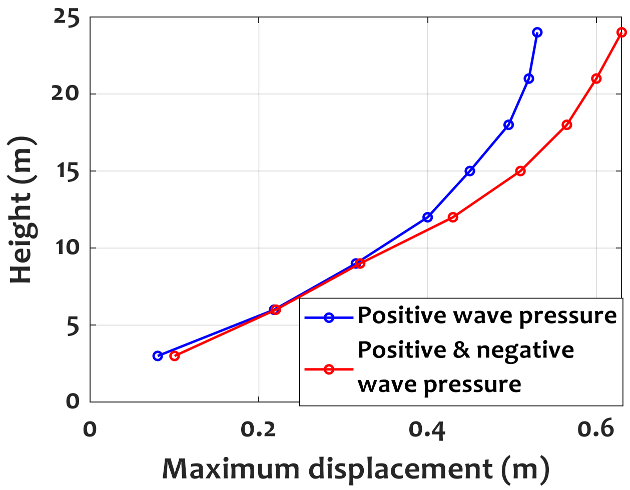


Fig. 20 Effect of positive and negative wave pressure on the maximum displacement response of G + 7 storey building

in Fig. 22. No significant change in displacement response is observed with the addition of bracings and cross section of bracings.

Conclusions

A study has been done on RC buildings of different heights subjected to various surface blast loads and standoff distances. The analysis considered five RC buildings with the same plan configuration and various heights. The selected buildings were modelled in standard FEM SAP2000 software and located at standoff distances ranging from 10 to 50 m subjected to charge weight ranging from 10 to 30 kg. The results were discussed in terms of maximum displacements, storey drifts, and base shear. Further, the analysis was carried out to mitigate the effect of blast response using strengthening techniques, i.e. jacketing of columns and addition of bracings with various dimensions.

- The maximum storey displacements, inter-storey drifts, and base shears were significantly varied at R < 30 m and significant variation was observed beyond 30 m for low-rise buildings. On the other hand, the displacement responses were significantly changed at a lower standoff distance ($R = 10$ m) for high-rise buildings, and the responses were the same for standoff distance greater than 10 m.
- The base shear was increased to 1.7–2.5 times with an increase in charge weight at $R = 10$ m for low-rise buildings.
- No significant effect was observed on the response of building between single and double bracings. The behaviour of buildings under air blast was not considered in this study and will be considered for the future work.

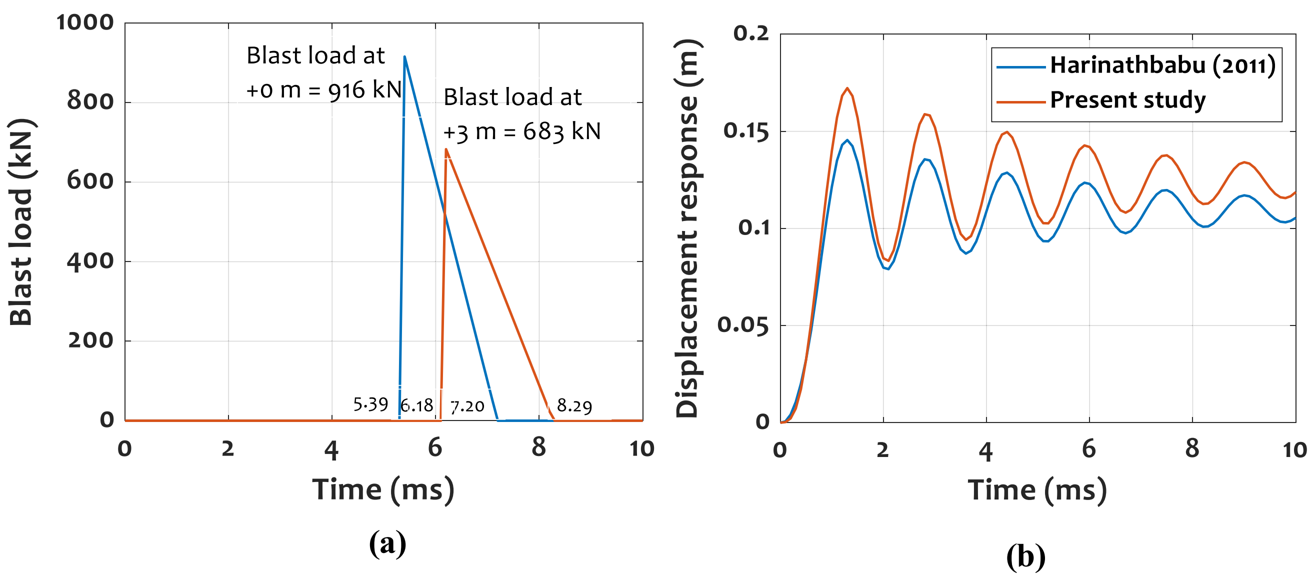


Fig. 21 Comparison of displacement time histories between AEM and FEM due to blast loads: **a** blast loads and **b** displacement time history

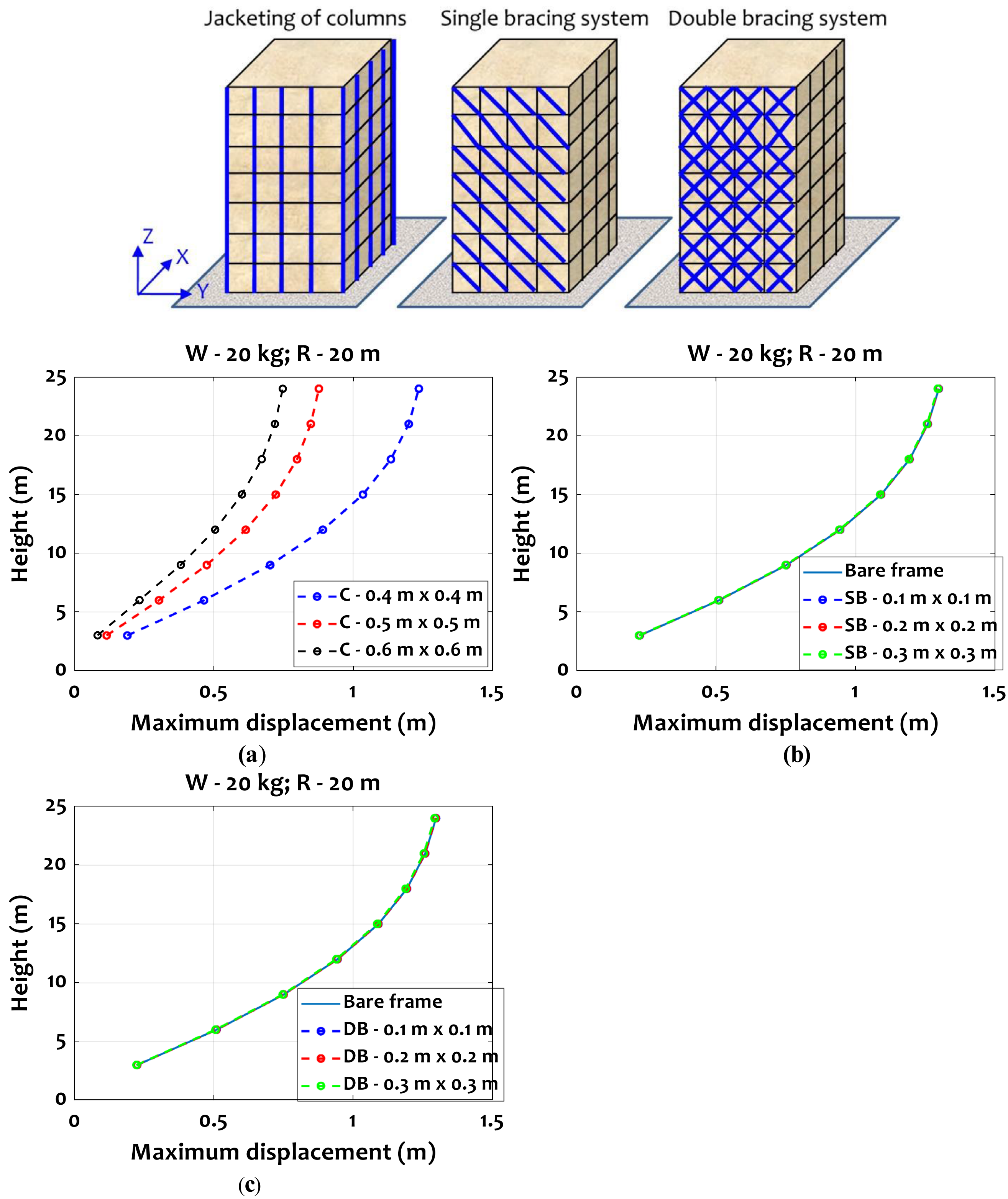


Fig. 22 Effect of maximum displacements of G+7 storey building due to **a** jacketing, **b** single bracing, and **c** double bracing, C—columns, SB—bracing single, and DB—bracing double

Declarations

Conflict of interest The author(s) declare that there is no conflict of interest regarding research, authorship, and/or publication of this article.

References

1. DoD (Department of Defense) (2013) Design of buildings to resist progressive collapse. UFC 4–023–03, Washington, DC
2. General Services Administration (GSA) (2003) Progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects. GSA, Washington, D.C.
3. ASCE (2005) Minimum design loads for buildings and other structures. SEI/ASCE 7-05 ASCE, Reston, Virginia
4. ACI 318 (2005) Building Code Requirements for Structural Concrete and Commentary (ACI 318R-05). ACI Committee 318, American Concrete Institute, Farmington Hills, USA
5. National Consortium for the Study of Terrorism and Responses to Terrorism, Global Terrorism Database, Univ. of Maryland. https://www.start.umd.edu/gtd/
6. Ambrosinia D, Luccionia B, Jacintoa A, Danesia R (2005) Location and mass of explosive from structural damage. J Eng Struct 27:167–176
7. Agnew E, Marjanishvili S, Gallant S (2007) Concrete detailing for blast. Struct Mag 5:26–28
8. Liew J (2008) Survivability of steel frame structures subject to blast and fire. J Constr Steel Res 64:854–866
9. Jun-Xiang X, Xi-La L (2009) A two-step approach to progressive collapse analysis of building structures under blast loading. J Shanghai Jiaotong Univ (Sci) 14:393–397
10. Jayasooriya R, Thambiratnam DP, Perera NJ (2014) Blast response and safety evaluation of a composite column for use as key element in structural systems. Eng Struct 61:31–43
11. Kwasniewski L (2010) Nonlinear dynamic simulations of progressive collapse for a multi-storey building. J Eng Struct 32:1223–1235
12. Shi Y, Zhong X, Hong H (2010) A new method for progressive collapse analysis of RC frames under blast loading. J Eng Struct 32:1691–1703
13. Harinathbabu R (2011) Numerical simulation of progressive collapse of RC buildings subjected to earthquake and blast loads. Dissertation, Andhra University, India
14. Goel MD, Matsagar VA, Gupta AK, Marburg S (2012) An abridged review of blast wave parameters. Def Sci J 62:300–306. https://doi.org/10.14429/dsj.62.1149
15. Nourzadeh D, Humar J, Braimah A (2017) Comparison of response of building structures to blast loading and seismic excitations. Proced Eng 210:320–325
16. Helmy H, Salem H, Mourad S (2012) Progressive collapse assessment of framed reinforced concrete structures according to UFC guidelines for alternative path method. Eng Struct 42:127–141. https://doi.org/10.1016/j.engstruct.2012.03.058
17. Hadianfard MA, Malekpour S, Momeni M (2018) Reliability analysis of H-section steel columns under blast loading. Struct Saf 75:45–56
18. Qian K, Li B (2017) Effects of masonry infill wall on the performance of RC frames to resist progressive collapse. J Struct Eng ASCE 143:1–17. https://doi.org/10.1061/(ASCE)ST.1943-541X.0001860
19. Sunita T, Bharti SD, Shrimali MK, Datta TK (2020) Effect of Surface Blast on Multistory Buildings. J Perform Constr Facil 34:1–15
20. Badshah E, Amjad N, Muhammad A, Feroz S, Kareem A (2017) Review of blast loading models, masonry response, and mitigation. Shock Vib. https://doi.org/10.1155/2017/6708341
21. Ahmed K, Taha MS, Zahran ZG (2021) Mitigation of the blast load effects on a building structure using newly composite structural configurations. Def Technol 17:75–84
22. ____Indian standard code of practice for plain and reinforced concrete. IS: 456–2000. Bureau of Indian Standards, New Delhi.
23. SAP2000 (2016) Integrated finite element analysis and design of structures computers and structures. CA, USA, Berkeley
24. Aleem U, Furqan A, Heung-Woon J, Sung-Wook K, Jung-Wuk H (2016) Review of analytical and empirical estimations for incident blast pressure. KSCE J Civ Eng 21:2211–2225
25. Newmark NM, Hansen RJ (1961) Design of blast resistant structures. In: Harris CM, Crede CE (eds) Shock and vibration handbook. McGraw-Hill, New York USA
26. Kadid A, Nezzar B, Yahiaoui D (2012) Nonlinear dynamic analysis of reinforced concrete slabs subjected to blast loading. Asian J Civ Eng 13:617–634
27. Vijayaraghavan C, Thirumalaivasan D, Venkatesan R (2012) A study on nuclear blast overpressure on buildings and other infrastructures using geospatial technology. J Comput Sci 8:1520–1530
28. Unified Facilities Criteria (2008) UFC 3–340–02 Structures to resist the effects of accidental explosions. U.S. Army Corps of Engineers, Naval Facilities Engineering Command, Air Force Civil Engineer Support Agency
29. Nelson L, Priyan M, Tuan N (2004) Response Spectrum Solutions for Blast Loading. Electron J Struct Eng (eJSE) 4:28–44
30. Ahmad S, Elahi A, Pervaiz H, Rahman AGA, Barbhuiya S (2014) Experimental study of masonry wall exposed to blast loading. Mater de Constr 64:1–11
31. Wu C, Hao H (2007) Numerical simulation of structural response and damage to simultaneous ground shock and air blast loads. Int J Impact Eng 34:556–572
32. Hatem TD, Kimiro M (1998) Applied element simulation for collapse analysis of Structures. Bull Earthq Resist Struct, IIS, Univ Tokyo 32:113–123
33. U.S. Department of the Army (1990) Structures to Resist the Effects of Accidental Explosions. Technical Manual 5–1300 (TM5–1300), Washington DC, USA
34. Okamura H, Maekawa K (1991) Nonlinear analysis constitutive models of reinforced concrete. Gihodo Co., Ltd., Tokyo
35. Murty CVR (2007) IITK-GSDMA guidelines on measures to mitigate effects of terrorist attacks on buildings. NICEE. Kanpur, India