In this paper, the response of regular concrete space frames subjected to gravity and blast loads for a six-storey building of 18 m high for a charge weight of 100 kg TNT at a 40 m range is studied. The type of blast chosen is a surface blast. Five different types of frames, skeleton frame (SFR), skeleton frame with the stiffness of slab (SFRS), skeleton frame with the stiffness of slab and 230 mm thick infill walls (SFRSWs 1), skeleton frame with the stiffness of slab and 150 mm thick infill walls (SFRSWs 2), and skeleton frame with the stiffness of slab and 115 mm thick infill walls (SFRSWs 3) were modelled and analyzed using STAAD Pro by converting the peak reflected blast pressure into equivalent static pressure by using the dynamic load factor. As the storey level increases the lateral displacements, shear force, and bending moment decrease due to a decrease in the impact of the blast at higher stories. The incorporation of infill walls in SFRSW1, SFRSW2, and SFRSW3 type frames shows a significant reduction in the lateral displacements due to the increase of stiffness when compared with SFR type frames.

1. Introduction

The rise in terrorist attacks in recent years has demonstrated that the impact of blast loads on structures is a serious issue that should be considered during the design phase. Even though terrorist attacks are very rare, the blast loads are dynamic loads which should be considered in the design process such as wind and earthquake loads. Due to the reason, the load generated due to blast is very high in magnitude, residential construction engineers are not considering it for design purposes, which leads to an increase in the construction cost of the project. An explosion of a bomb within or close to a structure can result in catastrophic damage to the internal and external structure of a building, the collapse of walls which results in the loss of life of humans, and many injuries to the occupants residing in the nearby vicinity of the blast.

In recent times, the increase in terrorist attacks and explosions of bombs in the vicinity of buildings has become a serious concern for researchers and designers to study the impact of the dynamic and impulsive blast effects caused by the explosion on structures. The rapid release of energy from a nuclear or a chemical source for a short interval of time with a huge impact is termed a “blast.” This impact created by the blast significantly affects the structures, which can be resisted by studying the progressive collapse analysis of a structure during the design phase or by the reduction of projectiles due to fragmentation. Steel and RCC elements in the structure increase the ductility by absorbing some amount of energy, and the brittle materials such as masonry, glass, and timber will fail suddenly due to the impact created at the time of the blast [1]. In the early 1960s, the analysis of blast loading on structures was used by the US Department of Army and released the code “structures to resist the effects of accidental explosions” in 1959, and the latest edition TM 5–1300 [2] that came into existence in 1990 is used all over the world by design engineers, military, and defence organizations. Many researchers [3–5] have developed analytical, numerical, and experimental methods to predict the blast response of structures.
semiempirical, and numerical approaches to estimate the blast loads coming onto the structures. The loads generated from the explosions such as gas, nuclear materials, plastic explosives, and the interaction of the blast with the building elements can be predicted in terms of TNT charge weight and the blast scaling laws. When an explosion occurs, it is classified as either confined or unconfined, depending on the confinement. Explosions in the air or on the surface are classified as confined, whereas explosions inside a building are classified as unconfined [5]. Control of deflection, crack width, vibration, and other serviceability-related criteria are not normally deemed essential. Explosions result in large dynamic loads, greater than the original design loads, for which the structures are analysed and designed. Analysis and design of blast loads require detailed knowledge of blast and its phenomena. Meganadh [6] investigated the response of blast load on a G + 5 multistoried building by considering the standoff distance of 40 m for surface burst and concluded that the increase of stiffness of RC elements gives better results and also resists the uplift forces when compared to the elements with less stiffness. Effects of blast loads can be reduced by providing a shear wall which provides the lateral resistance, and the effect of lateral loads can be controlled, which in turn reduces the damage to the buildings. Failure of the members of the structure is mainly due to front face pressure at ground floor level, and it decreases with an increase in the storey height [3]. In urban terrains, the prediction of blast loads and their effects on adjacent buildings has been investigated using empirical relations and considers that there should not be any obstacle between the target and charge weight [7]. Ngo has concluded that the prevention of progressive collapse criteria should be included in the codal provisions, and also, the ductility requirements should be enhanced to study its behaviour under severe load conditions [8]. Existing studies show that from the empirical methods, there is a large variation of peak positive incident pressure for free air and surface burst conditions for small standoff distances. To study the exact behaviour of an explosion, it is necessary to study the blast wave parameters at the centre of the explosion [1]. Blast load is a dynamic load, and the response of a structure subjected to seismic load is different from the blast load in terms of storey drift, structural and nonstructural safety aspects, and volume of concrete required for blast-resistant buildings [9]. Different charges such as TNT, RDX, and POLONIT-V are used as explosives for the blast, and theoretically, the assumption that blast waves propagate as hemispherical or spherical wavefronts is not true in the reality and it follows an irregular trend [7]. Harinadh [4] studied the linear response analysis of RC bare frames subjected to blast loads by considering four bare frames, namely, single storey-single bay, three storey-one bay, five storey-one bay, and ten storey-three bay and concluded that the response of the structure to blast load depends on the time of arrival and peak positive incident pressure. Hrvoje Draganic [5] explains the process of determination of blast load and suggests a numerical solution how to compute the positive and negative pressures that are acting on the facades of the building in the form of pressure time history analysis using SAP2000. Ullah reviewed the empirical and analytical estimations for the incident peak blast pressure suggested by researchers and concluded that if the scaled distance is more, blast parameters can be predicted more accurately using the existing expressions and if it is less, the accuracy may be reduced due to the limited test availability of data [10]. Using the applied element method, the collapse analysis of buildings subjected to blast loads was studied, and the parameters such as the severity of flying debris and the strength of undisturbed buildings were addressed [11]. Krishna studied the effect of storey drift and nonlinear time history analysis on a high-rise building in SAP2000 for 100 kg TNT charge weight at a 30m standoff distance, and the formation of hinges at different locations of beams and columns was addressed. TM 5-1300 [2] American Code for Structures to Resist the Effects of Accidental Explosions and IS 4991-1968 [12] Indian Code for Blast Resistant Design of Structures for Explosions above Ground are used to find the shock loading produced during the blast.

2. Preliminary Design and Plan

Configuration of Frames

The study is conducted by modelling six-storey RC regular space frames with plan dimensions of $30 \times 10 \text{ m}$, $30 \times 15 \text{ m}$, $30 \times 20 \text{ m}$, and $30 \times 30 \text{ m}$. The plan configurations are chosen based on the change in the aspect ratio of the building by keeping one dimension constant, and in the other direction, the number of bays of each frame varies. Each of the space frames is analysed for the following five cases as shown in Table 1.

The plan of regular frames used in the analysis is having 6 bays in the x direction and in the z direction, and it changes as per the plan configuration. A six-storey RC building with a height of each storey as 3 m, length and width of each bay 5 m, is considered in this study. Dimension of beams is considered as 300 mm $\times$ 450 mm and columns 300 mm $\times$ 600 mm. As per the codal provisions, the dead load of 13.8 kN/m, the live load of 2 kN/m², and the load combination 1.5(DL + LL) are considered for the analysis. The 3-D rendered view of skeleton frames (SFRs), skeleton frames with slab elements (SFRS), and skeleton frames with slab elements and infill walls (SFRSWs1) which are developed in the analysis software STAAD for the purpose of analysis as shown in Figures 1–3, respectively.

3. Estimation of Blast Load

Blast load is an extreme dynamic load which creates a huge impact in a short interval of time. The blast wave reaches the target point in a short interval of time $t_A$ after the blast and reaches the maximum positive pressure known as peak incident pressure $P_{SO}$ and then rapidly reduces to atmospheric pressure $P_{SO}$ and the downward trend continues in the negative direction until it reaches the maximum negative pressure. The blast wave propagates further in the negative phase and was taken as insignificant. There are many models developed by researchers for the peak positive incident pressure for surface and free air burst criteria. In the current
study, the estimation of blast load calculation is performed by considering the surface burst criteria and the negative pressure is ignored. The representation of the positive phase, negative phase, and the impulse produced by the blast is illustrated in Figure 4. As per IS 4991–1968, the standoff distance was taken as 40 m for surface burst, and the blast pressure at different levels of a structure for a charge weight of 100 kg TNT was determined. The peak-reflected blast pressure calculated using TM 5-1300 is converted into equivalent static pressure by using a dynamic load factor. The dynamic load factor depends on the ratio of the natural time period and peak pressure duration, which is calculated as per ASTM F2247-equivalent static load method. The calculated static pressure at different stories of a building is defined as a time history function in STAAD.Pro as a uniform pressure on the side walls, and after postprocessing, the comparison of displacements, bending moments, and shear force among all the considered regular RC space frames was studied.

3.1. Blast Load Estimation and Calculation of Load at 100 TNT Charge Weight. The peak positive incident pressure and time of arrival calculations are performed at different stories of the building at a standoff distance of 40 m as follows: Figures 5 and 6 represent the impact of surface burst criteria at ground level and 3 m level.

3.1.1. At Ground Level
Step 1:
Charge weight = 100 kg = 100 \times 2.204 = 220.41 lbs
Range, \( R_G = 40 \text{ m} = 40 \times 3.28 = 131.2 \text{ ft} \)
Step 2:
\( W = 1.2 \times 220.4 = 264.48 \text{ lbs} \)
Step 3:
Calculation of free-blast wave parameters \( P_{so}, t_A, \) and \( i_s \) from Figures 2–15 of TM 5-1300
Scaled ground distance, \( Z_G = R_G/W^{1/3} = 131.2/264.48^{1/3} = 20.439 \text{ ft}/\text{lb}^{1/3} \)
Peak positive incident pressure, \( P_{so} = 2.893 \text{ psi} \)
Time of arrival of blast load, \( t_A/W^{1/3} = 12.645, t_A = 81.226 \text{ ms} \)

3.1.2. At 3 m Level
Step 1:
Figure 4: Representation of the effect of the blasting phenomenon.

Figure 5: Blast effect at ground level.

Figure 6: Blast effect at 3 m height of the building.

Figure 7: Reflected pressure distribution for 100 kg TNT.

Figure 8: Lateral displacements vs storey level for 30 × 10 m frames.

Figure 9: Lateral displacements vs storey level for 30 × 15 m frames.
Charge weight = 100 kg = 100 × 2.204 = 220.41 lbs
Range, \( R_G = (40^2 + 3^2)^{1/2} = 44.112 \times 3.28 = 131.568 \text{ ft} \)

Step 2:
\( W = 1.2 \times 220.4 = 264.48 \text{ lbs} \)

Step 3:
Calculation of free-blast wave parameters \( P_{so}, t_A, \) and \( i_s \) from Figures 2–15 of TM 5-1300 scaled ground distance,
\( Z_G = R_G/W^{1/3} = 131.568/264.481/3 = 20.497 \text{ ft lb}^{1/3} \)

Peak positive incident pressure, \( P_{so} = 2.88 \text{ psi} \)

Time of arrival of blast load, \( t_A/W^{1/3} = 12.702, t_A = 81.534 \text{ ms} \)

Unit positive incident impulse, \( i_s/W^{1/3} = 4.213, i_s = 27.043 \text{ psi-ms} \)

Fictitious positive phase pressure duration, \( t_{of} = 2 \times i_s/P_s = 2 \times 27.043/2.881 = 18.774 \text{ ms} \)

Step 4:
Angle of incidence, \( \alpha = \tan^{-1}(3/40) = 4^\circ \)

Determine the reflected pressure coefficient \( C_{ra} \) from Figure 2-193 of TM 5-130
\( C_{ra} = 2.129 \)

Peak-reflected pressure at an angle of incidence, \( P_{ra} = C_{ra} \times P_{so} = 2.129 \times 2.881 = 6.042 \text{ Mpa} \)

3.1.3. At 18 m Level

Figure 10: Lateral displacements vs storey level for 30 × 20 m frames.

Figure 11: Lateral displacements vs storey level for 30 × 30 m frames.

Figure 12: Shear force vs storey level for 30 × 10 m frames.

Figure 13: Shear force vs storey level for 30 × 15 m frames.
Step 1:
Charge weight = 100 kg = 100 × 2.204 = 220.41 lbs
Range, \( R_G = (40^2 + 18^2)^{1/2} = 43.86 \times 3.28 = 143.872 \) ft
Step 2:
\( W = 1.2 \times 220.4 = 264.48 \) lbs
Step 3:
Calculation of free-blast wave parameters \( P_{so} \), \( t_A \), and \( i_s \) from Figures 2–15 of TM 5-1300 scaled ground distance, \( Z_G = R_G/W^{1/3} = 143.872/264.481/3 = 22.413 \text{ ft/lb}^{1/3} \)
Peak positive incident pressure, \( P_{so} = 2.532 \) psi
Time of arrival of blast load, \( t_A/W^{1/3} = 14.290 \), \( t_A = 91.728 \) ms
Unit positive incident impulse, \( i_s/W^{1/3} = 3.868 \), \( i_s = 24.829 \) psi-ms

Step 4:
\[ \alpha = \tan^{-1} \left( \frac{18}{40} \right) = 24^\circ \]
Determine the reflected pressure coefficient \( C_r \) from Figure 2-193 of TM 5-130
\[ C_r = 2.118 \]
Peak-reflected pressure at an angle of incidence,
\[ P_{ra} = C_r \times P_{so} = 2.118 \times 2.532 \text{psi} = 0.037 \text{ Mpa} \]

Fictitious positive phase pressure duration,
\[ t_A + t_{of} = 2 \times i_s/P_{so} = 2 \times 24.829/2.532 = 19.612 \text{ ms} \]

Table 2: Pressure, time of arrivals, and time of fictitious positive phase pressure duration at different storey levels of a structure for 100 kg TNT.

| Height (m) | Pressure (Mpa) | \( t_A \) (ms) | \( t_{of} \) (ms) | \( t_A + t_{of} \) (ms) |
|-----------|----------------|----------------|-----------------|----------------------|
| 0         | 0.043          | 81.226         | 18.749          | 99.975               |
| 3         | 0.042          | 81.534         | 18.774          | 100.308              |
| 6         | 0.042          | 82.446         | 18.856          | 101.301              |
| 9         | 0.041          | 83.941         | 18.986          | 102.927              |
| 12        | 0.040          | 85.629         | 19.155          | 104.785              |
| 15        | 0.038          | 88.614         | 19.372          | 107.986              |
| 18        | 0.037          | 91.728         | 19.612          | 111.339              |

The values of peak positive incident pressure are tabulated in Table 2, and the variation of pressure along the height is shown in Figure 7. As the height of the structure increases, there is a decrease in the amount of pressure, and also, the time of fictitious positive pressure distribution increases.

4. Results and Discussion

4.1. Lateral Displacements at Different Storey Levels for 30 × 10 m, 30 × 15 m, 30 × 20 m, and 30 × 30 m Frames.

The calculation of lateral displacements is carried out for each dimension of frame considered in analysis software STAAD, results are analysed, and also, a comparison of lateral displacements against storey levels is also performed. The variation of lateral displacements at different storey levels is shown in Figures 8–11.

From Figure 8, it was observed that there is a percentage of reduction in the displacement of 20.16, 97.26, 96.94, and 96.69 for SFRS, SFRSW 1, SFRSW 2, and SFRSW 3, respectively, when compared to SFR at storey 6 for 30 × 10 m frames. From Figure 9, it was observed that there was a percentage of reduction in the displacement of 3.08, 97.30, 96.98, and 96.74 for SFRS, SFRSW 1, SFRSW 2, and SFRSW 3, respectively, when compared to SFR at storey 6 for 30 × 15 m frames. From Figure 10, it was observed that there was a percentage of reduction in the displacement of 24.86, 97.38, 97.08, and 96.84 for SFRS, SFRSW 1, SFRSW 2, and SFRSW 3, respectively, when compared to SFR at storey 6 for 30 × 20 m frames. From Figure 11, it was observed that there was a percentage of reduction in the displacement of 27.12, 97.43, 97.12, and 96.89 for SFRS, SFRSW 1, SFRSW 2, and SFRSW 3, respectively, when compared to SFR at storey 6 for 30 × 30 m frames. It was inferred from the displacement results that the lateral displacement values reduce significantly from SFRS to SFRSW1, SFRSW2, and SFRSW3 due to the presence of...
Figure 16: Bending moment vs storey level for 30×10 m frames.

| Storey 1 | Storey 2 | Storey 3 | Storey 4 | Storey 5 | Storey 6 |
|----------|----------|----------|----------|----------|----------|
| Bottom Node | Top Node | Bottom Node | Top Node | Bottom Node | Top Node | Bottom Node | Top Node | Bottom Node | Top Node | Bottom Node | Top Node |
| SFR | 950.38 | 587.81 | 348.79 | 576.34 | 184.47 | 481.03 | 55.1 | 359.58 | -49.23 | -253.95 | -161.76 | 18.34 |
| SFRS | 695.43 | 560.45 | 435.58 | 578.28 | 252.35 | 465.97 | 124.11 | 339.28 | 12.79 | -225.96 | -101.61 | 0.47 |
| SFRSW 1 | -113.92 | 71.37 | -36.67 | 23.85 | -13.66 | 9.1 | -7 | 4.8 | -5.27 | 3.86 | -9.75 | 12.1 |
| SFRSW 2 | -158.07 | 98.01 | -47.66 | 30.75 | -17.34 | 11.37 | -8.98 | 5.99 | -6.97 | 4.89 | -12.88 | 15 |
| SFRSW 3 | -191.07 | 112.5 | -48.86 | 30.79 | -17.41 | 11.28 | -9.82 | 6.56 | -8.32 | 5.78 | -15.43 | 17.59 |

Figure 17: Bending moment vs storey level for 30×15 m frames.

| Storey 1 | Storey 2 | Storey 3 | Storey 4 | Storey 5 | Storey 6 |
|----------|----------|----------|----------|----------|----------|
| Bottom Node | Top Node | Bottom Node | Top Node | Bottom Node | Top Node | Bottom Node | Top Node | Bottom Node | Top Node | Bottom Node | Top Node |
| SFR | 1069.97 | -648.79 | 408.21 | -653.33 | 218.13 | -551.01 | 70.61 | -414.7 | -49.77 | -292.68 | -167.68 | 2.96 |
| SFRS | 761.44 | -667.45 | 483.6 | -647.22 | 295.04 | -515.68 | 152.3 | -372.61 | 26.87 | -244.37 | -101.14 | -3.22 |
| SFRSW 1 | -143.24 | 86.65 | -42.9 | 28.12 | -16.26 | 11.04 | -8.5 | 5.94 | -6.3 | 4.77 | -10.56 | 12.93 |
| SFRSW 2 | -189.17 | 114.87 | -54.66 | 35.29 | -19.86 | 13.13 | -10.34 | 6.96 | -8 | 5.78 | -13.83 | 16.01 |
| SFRSW 3 | -210.14 | 123.63 | -54.37 | 33.72 | -18.45 | 11.59 | -9.93 | 6.3 | -8.34 | 5.63 | -15.41 | 17.39 |
Figure 18: Bending moment vs storey level for 30 × 20 m frames.

Figure 19: Bending moment vs storey level for 30 × 30 m frames.
infill walls, which increases the stiffness of the structure and also due to the reduction in the positive pressure along the height.

4.2. Shear Force at Different Storey Levels for 30 × 10 m, 30 × 15 m, 30 × 20 m, and 30 × 30 m Frames. The variation of shear force at storey levels for different plan configurations is shown in Figures 12–15, respectively.

From Figure 12, it was observed that there was a change of shear force from 512.73 kN to −61.76 kN for a 30 × 10 m frame, 572.92 kN to −76.63 kN for a 30 × 15 m frame, 625.25 kN to −88.64 kN for a 30 × 20 m frame, and 665.19 kN to −94.01 kN for a 30 × 30 m frame while comparing skeleton frames (SFRs) with skeleton frames with the stiffness of slab and 230 mm infill walls (SFRSWs 1). From Figures 13–15, it was also observed that as the storey level increases, there is a reduction in the amount of shear force not only due to the decrease in the reduction of positive pressure due to blast load but also due to the amount of the total dead and live load acting at the top floors.

4.3. Bending Moment at Different Storey Levels for 30 × 10 m, 30 × 15 m, 30 × 20 m, and 30 × 30 m Frames. The variation of bending moment at the bottom node and top node at each storey for the different plan configurations is shown in Figures 16–19.

From Figure 16, it was observed that there was a decrease in the bending moment from 950.38 kNm to 695.43 kNm for SFR to SFRS, and there is a change in sign of the bending moment for SFRSW1, SFRSW2, and SFRSW3, having intensities of the bending moment −113.92 kNm, −158.07 kNm, and −191.07 kNm at the bottom node for 1st storey of 30 × 10 m frames. From Figures 17–19, it was inferred that the bending moment reduces from the bottom storey to the top storey due to the reduction in the peak positive incident pressure along the height. Also, from SFR to SFRS frame types, there is a reduction in the bending moment due to the consideration of the stiffness of the slab element. From SFRSW1 to SFRSW2 and SFRSW3 frames, there is an increase in the bending moment values due to a reduction in the thickness of infill walls from 230 mm to 150 mm and 115 mm, respectively. In all different plan configurations, the bending moment is maximum at the bottom storey and it is increased by 12.5% for 30 × 15 m frames, 23.4% for 30 × 20 m frames, and 32.2% for 30 × 30 m, respectively, when compared with 30 × 10 m plan configuration for SFR type frames.

For the 30 × 30 m plan configuration, the lateral displacements, shear force, and bending moment values are more when compared to 30 × 10 m, 30 × 15 m, and 30 × 20 m due to changes in the base dimensions of the plan and also due to the distribution of load across all the structural elements present. For the skeleton frame, the increase in the percentage of the maximum lateral displacement was found to be 35.06% for 30 × 30 m, 24.43% for 30 × 20 m, and 13.29% for 30 × 15 m when compared with 30 × 10 m plan configuration. For the skeleton frame with the stiffness of slab and with infill walls, also, the 30 × 30 m frame has more displacement when compared with other configurations due to change in the number of bays.

5. Conclusions

We studied the response of regular RC space frames subjected to blast load for a six-storey structure with plan dimensions 30 × 10 m, 30 × 15 m, 30 × 20 m, and 30 × 30 m, and the following are the conclusions drawn from the study:

1. As the storey level increases, the amount of the peak positive incident pressure reduces, and the shock velocity also reduces with the increase of the standoff distance.

2. Maximum lateral displacement values at storey 6 for all the plan configurations (30 × 10 m, 30 × 15 m, 30 × 20 m, and 30 × 30 m) reduce gradually from SFR to SFRS, SFRSW3, SFRSW2, and SFRSW1 due to the increase of stiffness in the frames by proving the different thickness of infill walls.

3. Shear force and bending moment values also reduce with the increase of storey height due to change in the amount of the peak positive incident pressure at different stories.

4. By the incorporation of infill walls (which practically exists), instead of considering only the skeleton frame, the displacement, shear force, and bending moment values reduce considerably due to the effect of blast load.

Data Availability

The data utilized to support the findings of the study are included within the article.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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