Explicit and Combined Effects of Blast and Seismic Loadings on Tall Buildings with and without Shear Walls

Ali K. Kadhum¹ and Khattab S. Abdul-Razzaq²
¹University of AL-Mustansiriyyah, College of Engineering, Department of Water Resources, Baghdad, Iraq
²University of Diyala, College of Engineering, Department of Civil Engineering, Diyala, Iraq

Email: dr.khattabsaleem@yahoo.com

Abstract This paper aims at comparing seismic (EQ) and blast (TNT) loadings using different TNT weights on 12-storey reinforced concrete residential building with and without shear walls. These types of loading should be taken into considerations now in Iraq. The explosions besides earthquakes must be studied in Iraq to avoid significant losses in the lives of people and their public and private property. The same reinforced concrete multistory building was designed with three different TNT weights; 100, 350 and 750 kg in order to discuss drift, displacement and story shear. A commercial package ETABS2018 was used to analyze this 36-meter-high building. The building was designed according to ACI 318-14 and US Department of Defense TM5-1300 for blast load. As for the blast load, in case of shear wall existence, the maximum increase in drift was seen on the 11th and 12th stories about 1306-187% and 1232-175 %, respectively for each TNT weight compared with EQ. The maximum increase in drift was found on the 11th and 12th stories about 4236-618% and 2646-386% for each TNT weight. The maximum increase in story shear was seen on the 2nd and 3rd stories 5150-698% and 4625-618% for each TNT weight compared with EQ.

Key words: Blast load, Seismic load, ETABS, Tall building, TNT, drift, storey shear.

1. Introduction
Nowadays earthquake and terrorist attack explosion problems are very severe. To overcome such types of problems and to protect the structure from such disaster, this study is conducted. The blast explosion within or nearby structure is because of quarry blasting or vehicle bomb or pressure. These leads to catastrophic damage to the building structural frames, both externally and internally. The result will be critical life-safety systems shutting down, walls collapse, and blowing out of windows. Dams, bridges, buildings, pipelines systems, etc. are the lifeline structures which play a significant role in the country economy and that is why they should be safe in any condition. Reinforced concrete shear walls are
commonly designed as a lateral resisting system to withstand wind loads and earthquake. The shear walls
design to resist in-plane loads make them vulnerable to blast forces which usually generate out-of-plane
loads captured by the wall large surface area. Shear wall systems usually help in the building load bearing
action. Absence of such a component might cause a progressive collapse. That is why it is beneficial for
engineers to have the capability to estimate the blast resistance of such a structure so as to make the shear
wall design. There is a lot of research that dealt with the damage caused by seismic load or heat on
buildings [1-6]. However, studying the effect of shear walls on the damage caused by the blast load and
seismic load still needs to be more investigated [7,8].

1.1 Problem Statement
When the blast wave hits a rigid surface, the reflected pressure is more than the incident peak pressure
(P_{so}), Figure 1. This rise took place due to the propagation nature of the blast wave through the air.
Through the traveling of the wave, it transfers along the particles of the air that collide with the surface
upon reaching. In an ideal linear-elastic case, the particles must be freely able to bounce back causing a
reflected pressure equal to the incident pressure, therefore, the surface would under a double pressure. In
the case of a strong blast wave, like a shock wave is a non-linear event, the particles reflection is
obstructed by subsequent particles of the air that are transferred there, that causes much higher values of
the reflected pressure. In this situation the surface would suffer a much more acting pressure than the
incident one [9].

1.2 Characteristics of Blast Wave
Figure 2 illustrates the ideal appearance of time pressure for the free-blast wave issue, which comes a
point at a specific distance from the detonation. The pressure around the element is firstly equal to the
pressure around (P_o), and is subject to an immediate increase to the peak of the (P_{so}) pressure at the time of
arrival (t_A), when the shock front arrives to that point. The time required for compression to get its peak
value is very small and for the purposes of design, it is supposed to be zero. (P_{so}) peak pressure is also
recognized as lateral over-pressure or peak over-pressure. The value of the peak over-pressure as well as
the speed of propagation of the shock wave decreases with distance -from the detonation center-
increment. After its peak value, the pressure decreases with the exponential rate until it becomes close to
the surrounding pressure at t_A + t_o, where t_o is the period of the positive phase. After the pressure time
pattern positive phase, the pressure gets smaller (denoted by negative) than the surrounding value, and at

![Figure 1. Blast loading effect on the building](image-url)
last returns to it [9,10]. The negative phase is longer than the positive phase, and the minimum pressure value is referred to as \(P_{so}\) and the duration for that \(t_{o}\). Throughout this stage, the structures are exposed to suction forces, therefore sometimes fragments of glass loaded with explosions from faults of the facades are found outside of a building rather than inside [11,12].

![Typical blast wave pressure-time profile](image)

**Figures 2.** Typical blast wave pressure-time profile

### 2. Description and Modeling of Building

Figures 3 and 4 show the 12-storey reinforced concrete residential building which is 36 m height. It was analyzed using ETABS once under the blast load (with and without shear wall) and another under the seismic load (without shear wall).

#### 2.1 Properties

Concrete and steel characteristics were identified with the density properties of each type. Four models were studied with different burst weights as shown in the Table 1.
Table 1. Detailed properties

| Grade           | Model No. 1 | Model No. 2 | Model No. 3 | Model No. 4 |
|-----------------|-------------|-------------|-------------|-------------|
| Concrete        | M27         | M27         | M27         | M27         |
| Rebar           | Fe415       | Fe415       | Fe415       | Fe415       |
| Density of concrete (kN/m²) | 25          | 25          | 25          | 25          |
| Density of Steel (kN/m³)   | 78.5        | 78.5        | 78.5        | 78.5        |
| Poisson's ratio    | 0.2         | 0.2         | 0.2         | 0.2         |
| Type of Building   | Irregular   | Irregular   | Irregular   | Irregular   |

| Type of Model | Weight of TNT | Earthquake (Seismic Load) |
|---------------|---------------|---------------------------|
|               |               |                           |
| Code Concrete Design | ACI 318-14 |                             |
| Load Effect    | TM5-1300      | ASCE 07-10                 |
| Value for the Effect | 100          | 350                        | 750          |
| Standoff Distance (m) | 5            | 5                          | 5            |
| Live load (kN/m²) | 2.5          | 2.5                        | 2.5          |
| Floor finish load (kN/m²) | 1.5         | 1.5                        | 1.5          |
| Wall load (kN/m) | 15           | 15                         | 15           |

2.2 Properties of section

The dimensions of the columns, beams, walls and slabs were added, along with the dimensions of the building and height as shown in the Table 2.

Table 2. Model Description and Sectional Properties

| Plan               | 30 m × 36 m |
|--------------------|-------------|
| X- direction       | 6 spaces, 5m |
| Y- direction       | 6 spaces, 6m |
| The height of each story | 3 m        |
| Column Section     | 500mm × 250mm |
| Beam Section       | 600mm × 250mm |
| Shear wall         | 250 mm      |
| Slab Thickness     | 200 mm      |

3. Structural blast loads calculations

3.1 Determination of blast pressure

There are many approaches and relationships to calculate the incident pressure value at an exact distance from an explosion. All the suggested relationships require scaled distance computation, which relies on the actual distance from the spherical explosion center in addition to the explosive mass. Kinney [18] suggested a formulation that depends on chemical type explosions. It is defined by Equation (1) and has been widely used for the purposes of computer calculation:
where $Z$ (m/kg$^{1/3}$) is the scaled distance.

Po is the ambient pressure.

$P_{so}$ is the Peak Incident Pressure.

\[
P_{so} = P_0 \frac{808 \left[ 1 + \left( \frac{Z}{4 \cdot 5} \right)^2 \right]}{\left[ 1 + \left( \frac{Z}{0.048} \right)^2 \right] \left[ 1 + \left( \frac{Z}{0.32} \right)^2 \right] \left[ 1 + \left( \frac{Z}{1 \cdot 35} \right)^2 \right]^{0.5}}
\]

where $Z$ (m/kg$^{1/3}$) is the scaled distance.

For ground surface blast, Newmark [19] presented another computation way for peak pressure values without categorization based on detonation severity:

\[
P_{so} = 6784 \frac{w}{R^3} + 93 \sqrt[3]{\frac{w}{R^3}}
\]

where, $P_{so}$ is in bars, $W$ is the TNT charge mass in metric tons (=1000kg)

\[
R = \frac{R}{\sqrt{w}}
\]

where, $R$ is the distance from the point of interest to the detonation source [m] and $w$ is the weight (the mass) of the explosive, in kg.

For ground surface blast, Newmark [19] presented another computation way for peak pressure values without categorization based on detonation severity:

\[
P_{so} = \left\{ \begin{array}{ll}
6.7 \frac{6.7}{x} + 1 & , \text{For } p_{so} < 10 \text{ bar} \\
0.975 \frac{1.455}{x^2} + 5.85 \frac{5.85}{x^3} - 0.019 & , \text{For } 0.1 < p_{so} < 10 \text{ bar}
\end{array} \right.
\]

The most extensively accepted and used approach for the blast parameters determination is that suggested by Kingery-Bulmash [20]. Their paper includes formulations for both surface bursts (hemispherical pressure waves) and free air bursts (spherical) and present the values of reflected and incident pressures in addition to all other parameters. The suggested blast parameters are valid for distances 0.05-40 m as the diagrams contained in [20] are referred to 1kg of TNT. In order to make the comparison, Figure 4 presents the curves of the expressions above for peak incident overpressure versus scaled distance for both hemispherical waves (surface bursts) and spherical waves (free-air bursts). The Kingery-Bulmash study related curves are included for reference. It is seen that the plotted Equations (2), (4) and (5) significantly deviate from that of Kingery-Bulmash for small scaled distances. That can be attributed to the fact that these equations were derived not for conventional explosives, but principally for nuclear blasts. The curve plotted from Equation (1) of Kinney shows acceptable estimations over the whole scaled distance [13-15].
Figure 4. Curves comparison on peak incident overpressure versus scaled distance basis

Figure 5 shows the blast parameters diagrams for the blast wave positive phase for surface bursts. These diagrams are the demonstration of the curves aforesaid in references [10] and [11] in metric-units. They are generally more thorough, and the curves have been plotted related to scaled distances from Z=0.05 m/kg\(^{1/3}\) to Z=40 m/kg\(^{1/3}\). From these diagrams so as to get each parameter absolute value, its scaled value has to be multiplied by a factor W\(^{1/3}\) in order to take into consideration the charge actual size. Obviously, as stated above for the Hopkinson-Cranz scaling law, the quantities of velocity and pressure are not scaled. Figure 2 defines the symbols ran across in Figure 4, where the variation curve idealized pressure-time is shown. Other symbols are:

- \(U\) = speed of the shock wave (m/ms)
- \(L_w\) = blast wavelength (m).

\(L_w\) can be defined, at a given standoff distance point at a certain moment, for \(L_w +\) as the length under positive pressure (or, negative pressure for \(L_w -\)).

3.2 Positive phase parameters of TNT shock hemispherical wave

Table 3 shows the parameters of shock hemispherical wave positive phase, three different TNT weights from surface, each joint on side face of the 12-story [16,17].
Figure 5. Parameters of positive phase of shock hemispherical wave of TNT charges from surface bursts

Table 3. Parameters of blast load and pressure acting on side face for the building (From Figure 5)

| Value of $(Z)$ | Reflected impulse $Pr$ | Incident pressure $Pr_o$ | Incident impulse $Is$ | Reflected impulse $ Ir$ | Arrival time $Ta$ | Positive duration $To$ |
|---------------|------------------------|--------------------------|-----------------------|------------------------|-------------------|-----------------------|
| 1.3           | 5000                   | 1200                     | 230                   | 850                    | 0.45              | 1.9                   |
| 1.7           | 1600                   | 450                      | 170                   | 400                    | 1.8               | 2                     |
| 2             | 1000                   | 280                      | 150                   | 380                    | 1.9               | 2                     |
| 2.2           | 900                    | 250                      | 150                   | 320                    | 2                 | 2                     |
| 2.4           | 600                    | 180                      | 130                   | 300                    | 2.5               | 2.2                   |
| 2.6           | 500                    | 100                      | 100                   | 280                    | 3                 | 2.5                   |
| 2.8           | 400                    | 90                       | 95                    | 250                    | 3.2               | 2.8                   |
| 3             | 310                    | 95                       | 90                    | 220                    | 3.8               | 3                     |
| 3.5           | 250                    | 80                       | 80                    | 190                    | 4.5               | 3.4                   |
| 4             | 180                    | 60                       | 70                    | 180                    | 6                 | 3.5                   |
3.3 Value of the scaled distance

From Tables 4, 5 and 6, it can be seen the values for scaled distances explosion \((Z)\) for three different TNT weight models for each joint on side face of the 12-story.

**Table 4.** Value of \((Z)\) for standoff 5 m -TNT W 100 kg to all joints on side face building from the explosion

| Joint Story | 1   | 2   | 3   | 4   | 5   | 6   | 7   |
|-------------|-----|-----|-----|-----|-----|-----|-----|
| 1           | 3.46| 2.45| 2.45| 3.28| 2.45| 2.45| 3.46|
| 2           | 3.52| 2.54| 2.54| 3.35| 2.54| 2.54| 3.52|
| 3           | 3.58| 2.62| 2.62| 3.41| 2.62| 2.62| 3.58|
| 4           | 3.64| 2.7 | 2.7 | 3.47| 2.7 | 2.7 | 3.64|
| 5           | 3.7 | 2.78| 2.78| 3.54| 2.78| 2.78| 3.7 |
| 6           | 3.75| 2.86| 2.86| 3.6 | 2.86| 2.86| 3.75|
| 7           | 3.81| 2.93| 2.93| 3.66| 2.93| 2.93| 3.81|
| 8           | 3.87| 3   | 3   | 3.71| 3   | 3   | 3.87|
| 9           | 3.92| 3.07| 3.07| 3.77| 3.07| 3.07| 3.92|
| 10          | 3.98| 3.14| 3.14| 3.83| 3.14| 3.14| 3.98|
| 11          | 4.03| 3.21| 3.21| 3.89| 3.21| 3.21| 4.03|
| 12          | 4.08| 3.28| 3.28| 3.94| 3.28| 3.28| 4.08|

**Table 5.** Value of \((Z)\) for standoff 5 m -TNT W 350 kg to all joints on side face building from the explosion

| Joint Story | 1   | 2   | 3   | 4   | 5   | 6   | 7   |
|-------------|-----|-----|-----|-----|-----|-----|-----|
| 1           | 2.41| 1.71| 1.71| 2.28| 1.71| 1.71| 2.41|
| 2           | 2.45| 1.76| 1.76| 2.33| 1.76| 1.76| 2.45|
| 3           | 2.49| 1.82| 1.82| 2.37| 1.82| 1.82| 2.49|
| 4           | 2.53| 1.88| 1.88| 2.42| 1.88| 1.88| 2.53|
| 5           | 2.57| 1.93| 1.93| 2.46| 1.93| 1.93| 2.57|
| 6           | 2.61| 1.99| 1.99| 2.5 | 1.99| 1.99| 2.61|
| 7           | 2.65| 2.04| 2.04| 2.54| 2.04| 2.04| 2.65|
| 8           | 2.69| 2.09| 2.09| 2.59| 2.09| 2.09| 2.69|
| 9           | 2.73| 2.14| 2.14| 2.62| 2.14| 2.14| 2.73|
| 10          | 2.77| 2.19| 2.19| 2.66| 2.19| 2.19| 2.77|
| 11          | 2.81| 2.23| 2.23| 2.7 | 2.23| 2.23| 2.81|
| 12          | 2.84| 2.28| 2.28| 2.74| 2.28| 2.28| 2.84|
Table 6. Value of (Z) for standoff 5 m - TNT W 750 kg to all joints on side face building from the explosion

| Joint story | 1    | 2    | 3    | 4    | 5    | 6    | 7    |
|-------------|------|------|------|------|------|------|------|
| 1           | 1.78 | 1.26 | 1.26 | 1.69 | 1.26 | 1.26 | 1.78 |
| 2           | 1.81 | 1.3  | 1.3  | 1.72 | 1.3  | 1.3  | 1.81 |
| 3           | 1.84 | 1.35 | 1.35 | 1.75 | 1.35 | 1.35 | 1.84 |
| 4           | 1.87 | 1.39 | 1.39 | 1.79 | 1.39 | 1.39 | 1.87 |
| 5           | 1.9  | 1.43 | 1.43 | 1.82 | 1.43 | 1.43 | 1.9  |
| 6           | 1.93 | 1.47 | 1.47 | 1.85 | 1.47 | 1.47 | 1.93 |
| 7           | 1.96 | 1.51 | 1.51 | 1.88 | 1.51 | 1.51 | 1.96 |
| 8           | 1.99 | 1.54 | 1.54 | 1.91 | 1.54 | 1.54 | 1.99 |
| 9           | 2.02 | 1.58 | 1.58 | 1.94 | 1.58 | 1.58 | 2.02 |
| 10          | 2.05 | 1.62 | 1.62 | 1.97 | 1.62 | 1.62 | 2.05 |
| 11          | 2.07 | 1.65 | 1.65 | 2   | 1.65 | 1.65 | 2.07 |
| 12          | 2.1  | 1.68 | 1.68 | 2.03 | 1.68 | 1.68 | 2.1  |

3.4 Value of blast load effect on joint

From Fig 8 and Tables 7, 8 and 9, it can be seen the value for blast load applying on joint (side face) for 12-story, for three model different weights of TNT for each joint on side face of 12-story.
## Table 7. (p kN) blast load effect on joint side face for standoff 5m, 100kg

| Joint | Story | 1     | 2     | 3     | 4     | 5     | 6     | 7     |
|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| 1     | 1     | 730   | 1600  | 1600  | 1500  | 1600  | 1600  | 730   |
| 2     | 2     | 715   | 1450  | 1450  | 1482  | 1450  | 1450  | 715   |
| 3     | 3     | 700   | 880   | 880   | 1464  | 880   | 880   | 700   |
| 4     | 4     | 680   | 855   | 855   | 1430  | 855   | 855   | 680   |
| 5     | 5     | 665   | 825   | 825   | 1400  | 825   | 825   | 665   |
| 6     | 6     | 640   | 790   | 790   | 1380  | 790   | 790   | 640   |
| 7     | 7     | 620   | 780   | 780   | 1355  | 780   | 780   | 620   |
| 8     | 8     | 590   | 765   | 765   | 1320  | 765   | 765   | 590   |
| 9     | 9     | 575   | 758   | 758   | 1290  | 758   | 758   | 575   |
| 10    | 10    | 550   | 750   | 750   | 1260  | 750   | 750   | 550   |
| 11    | 11    | 543   | 742   | 742   | 1200  | 742   | 742   | 543   |
| 12    | 12    | 278   | 374   | 374   | 265   | 374   | 374   | 278   |

## Table 8. (p kN) blast load effect on joint side face for standoff 5m, 350kg

| Joint | Story | 1     | 2     | 3     | 4     | 5     | 6     | 7     |
|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| 1     | 1     | 1620  | 4000  | 4000  | 4420  | 4000  | 4000  | 1620  |
| 2     | 2     | 1535  | 3780  | 3780  | 3930  | 3780  | 3780  | 1535  |
| 3     | 3     | 1410  | 3550  | 3550  | 3450  | 3550  | 3550  | 1410  |
| 4     | 4     | 1290  | 3225  | 3225  | 3200  | 3225  | 3225  | 1290  |
| 5     | 5     | 1150  | 2955  | 2955  | 2885  | 2955  | 2955  | 1150  |
| 6     | 6     | 900   | 2530  | 2530  | 2650  | 2530  | 2530  | 900   |
| 7     | 7     | 870   | 2480  | 2480  | 2335  | 2480  | 2480  | 870   |
| 8     | 8     | 855   | 2440  | 2440  | 2440  | 2440  | 2440  | 855   |
| 9     | 9     | 830   | 2310  | 2310  | 2175  | 2310  | 2310  | 830   |
| 10    | 10    | 815   | 2280  | 2280  | 1742  | 2280  | 2280  | 815   |
| 11    | 11    | 810   | 1950  | 1950  | 1710  | 1950  | 1950  | 810   |
| 12    | 12    | 400   | 1120  | 1120  | 420   | 1120  | 1120  | 400   |

## Table 9. (p kN) blast load effect on joint side face for standoff 5m, 750kg

| Joint | Story | 1     | 2     | 3     | 4     | 5     | 6     | 7     |
|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| 1     | 1     | 3885  | 11500 | 11500 | 8000  | 11500 | 11500 | 3885  |
| 2     | 2     | 3690  | 10800 | 10800 | 7800  | 10800 | 10800 | 3690  |
| 3     | 3     | 3500  | 9800  | 9800  | 7660  | 9800  | 9800  | 3500  |
| 4     | 4     | 3330  | 9500  | 9500  | 7330  | 9500  | 9500  | 3330  |
| 5     | 5     | 3115  | 9100  | 9100  | 7000  | 9100  | 9100  | 3115  |
| 6     | 6     | 2890  | 8550  | 8550  | 6780  | 8550  | 8550  | 2890  |
| 7     | 7     | 2700  | 7950  | 7950  | 6460  | 7950  | 7950  | 2700  |
| 8     | 8     | 2530  | 7420  | 7420  | 6130  | 7420  | 7420  | 2530  |
| 9     | 9     | 2500  | 6850  | 6850  | 5700  | 6850  | 6850  | 2500  |
| 10    | 10    | 2475  | 6150  | 6150  | 5370  | 6150  | 6150  | 2475  |
| 11    | 11    | 2440  | 5500  | 5500  | 5040  | 5500  | 5500  | 2440  |
| 12    | 12    | 2185  | 5000  | 5000  | 5000  | 5000  | 5000  | 2185  |
3.5 ASCE 7-10 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern EQ+X according to ASCE 7-10, as calculated by ETABS.

3.5.1 Direction and Eccentricity
Direction = X + Y
Eccentricity Ratio = 5% for all diaphragms

3.5.2 Structural Period
Period calculation method = Program Calculations

- Coefficient, $C_t$, ASCE Table 12.8-2: $C_t = 0.028\text{ft}$
- Coefficient, $x$, ASCE Table 12.8-2: $x = 0.8$
- Structure Height Above Base, $h_n$: $h_n = 118.11\text{ft}$
- Long-Period Transition Period, $T_L$, ASCE 11.4.5: $T_L = 8\text{sec}$

3.5.3 Factors and Coefficients
- Response Modification Factor, $R$, ASCE Table 12.2-1: $R = 6.5$
- System Overstrength Factor, $\Omega_0$, ASCE Table 12.2-1: $\Omega_0 = 3$
- Deflection Amplification Factor, $C_d$, ASCE Table 12.2-1: $C_d = 5.5$
- Importance Factor, $I$, ASCE Table 11.5-1: $I = 1.5$

3.5.4 $S_s$ and $S_1$ Source = User Specified
- Mapped MCE Spectral Response Acceleration, $S_s$, ASCE 11.4.1: $S_s = 1g$
- Mapped MCE Spectral Response Acceleration, $S_1$, ASCE 11.4.1: $S_1 = 0.3g$

3.5.5 Seismic Response
- Site Class, ASCE Table 20.3-1 = D - Stiff Soil
- Site Coefficient, $F_a$, ASCE Table 11.4-1: $F_a = 1.1$
- Site Coefficient, $F_v$, ASCE Table 11.4-2: $F_v = 1.8$

MCE Spectral Response Acceleration, $S_{MS}$, ASCE 11.4.3, Eq. 11.4-1: $S_{MS} = F_a S_s$ $S_{MS} = 1.1g$

MCE Spectral Response Acceleration, $S_{M1}$, ASCE 11.4.3, Eq. 11.4-2: $S_{M1} = F_v S_1$ $S_{M1} = 0.54g$

Design Spectral Response Acceleration, $S_{DS}$, ASCE 11.4.4, Eq. 11.4-3: $S_{DS} = \frac{2}{3} S_{MS}$ $S_{DS} = 0.733333g$

Design Spectral Response Acceleration, $S_{D1}$, ASCE 11.4.4, Eq. 11.4-4: $S_{D1} = \frac{2}{3} S_{M1}$ $S_{D1} = 0.36g$
3.5.6 Equivalent Lateral Forces

Seismic Response Coefficient, $C_s$, ASCE 12.8.1.1, Eq. 12.8-2

$$C_s = \frac{S_{DS}}{\left( \frac{R}{T} \right)}$$

ASCE 12.8.1.1, Eq. 12.8-3

$$C_{s,\text{max}} = \frac{S_{DL}}{\left( \frac{R}{T} \right)}$$

ASCE 12.8.1.1, Eq. 12.8-5

$$C_{s,\text{min}} = \max(0.044S_{DS}, 0.01) = 0.0484$$

ASCE 12.8.1.1, Eq. 12.8-6

$$C_{s,\text{min}} = 0.5 \frac{S_1}{\left( \frac{R}{T} \right)}$$

$$C_{s,\text{min}} \leq C_s \leq C_{s,\text{max}}$$

4. RESULTS

4.1 DRIFT: -

According to the results, Figure 7 (A, B and C) shows the effect of shear wall existence. Applying TNT load, it is seen that the increase in drift in the building without shear wall is more than that with shear wall or also than EQ load. Taking into considerations the results of the stories 12th, 11th, 7th, 6th, 3rd and 2nd, drift increased by 132%, 265%, 829%, 998%, 1899% and 2659% due to 100kg blast load. While drift increased by about 132%, 253%, 813%, 958% and 2709% due to 350kg blast load. Finally, drift increased by about 123%, 242%, 819%, 1000%, 1957% and 2702% due to 750kg blast load. The maximum increase in drift was observed on the 2nd and 3rd stories by about 1958-1899% and 2709-2659%, respectively for each TNT weight.

Furthermore, applying EQ with TNT to building that has shear wall, it can be seen that the drift increases in case of with shear wall more than the case of EQ. Contemplating the results of the stories 12th, 11th, 7th, 6th, 3rd and 2nd, drift increase was by about 618%, 386%, 194%, 177%, 125% and 102% due to 100kg blast load. While drift increased by about 1469%, 918%, 461%, 425%, 314% and 259% due to 350 kg blast load. Finally, the drift increase was about 4236%, 2646%, 1344%, 1243%, 911% and 745% due to 750 kg blast load. The maximum increase in drift was observed on the 12th and 11th stories about 4236-618% and 2646-386%, respectively for each TNT weight compared with EQ.
4.2 Displacements

Figure 8 shows the difference between with and without shear wall building under blast load in addition to seismic load without shear wall. It is seen that the increase in displacement in building without shear wall was more than that with shear wall, and also, more than that under-EQ load. Taking the results of the stories 2nd, 3rd, 6th, 7th, 11th and 12th, displacement increased by about 1043%, 1127%, 1685%, 1904%, 2460% and 2685% due to 100kg blast load. While displacement increased by about 1063%, 1146%, 1590%, 1749%, 2437% and 2707% due to 350kg blast load. Finally, displacement increase was about 1062%, 1145%, 1588%, 1742%, 2437% and 2692% due to 750 kg blast load. The maximum increase in displacement was seen on the 2nd and 3rd stories about 2460-2430% and 2707-2685%, respectively for each TNT weight.

Also, applying EQ with TNT with shear wall building, it can be seen that the displacement increase in case of with shear wall is more than EQ. Taking the results of the stories 12th, 11th, 7th, 6th, 3rd and 2nd, displacement increase was about 187%, 175%, 142%, 135%, 172% and 245% of 100 kg blast load. While displacement increase was about 451%, 425%, 351%, 334%, 448% and 626% due to 350 kg blast load. Finally, displacement increase was about 1306%, 1232%, 1022%, 976%, 1287% and 1795% due to 750 kg blast load. The maximum increase displacement was seen on the 12th and 11th storeys about 1306-187% and 1232-175%, respectively for each TNT weight.
Figure 8-A

Figure 8-B

Figure 8-C

**Figure 8.** Max Displacement (mm) with different four models

### 4.3 Story Shear

Figure 9 shows the difference between with and without shear wall building under blast load in addition to seismic load without shear wall. From Figure 9, it can be seen that the increase in story shear under different TNT weights and under EQ. Contemplating the results of the stories 2nd, 3rd, 6th, 7th, 11th and 12th, story shear increase was about 257%, 258%, 234%, 231%, 222% and 246% due to 100-350kg of TNT. While story shear increased by about 286%, 289%, 298%, 293%, 272% and 245% due to 350-750kg of
TNT. Finally, story shear increased by about 737%, 747%, 699%, 678%, 606% and 602% due to 100-750 kg of TNT.
The maximum story shear increase was seen on the 2nd and 3rd stories about 257% and 258% under TNT weight of 100-350 kg. The maximum story shear increase was seen on the 6th and 7th stories about 699% and 678% due to TNT weight of 350-750 kg. The maximum story shear increase was seen on the 2nd and 3rd stories about 737% and 747% due to TNT weight of 100-750 kg.

Comparing EQ with different weights of TNT, it can be seen that the story shear increase in building due to TNT is more than that due to EQ. Contemplating the results of the stories 12th, 11th, 7th, 6th, 3rd and 2nd, the story shear increased by about 195%, 301%, 453%, 487%, 618% and 698% due to 100 kg TNT compared with that due to EQ. The story shear increased by about 480%, 669%, 1047%, 1143%, 1596% and 1799% due to 350 kg TNT compared with EQ.

Finally, the story shear increased by about 1177%, 1825%, 3076%, 3412%, 4625% and 5150% due to 750 kg TNT compared with EQ. The maximum story shear increase was seen on the 2nd and 3rd storeys about 5150-698% and 4625-618%, respectively for each TNT weight compared with EQ. Table 10 may be so useful because it shows the summery in detail.

![Figure 9. Max story shear (kN) with different Models: No.1, 2, 3 and 4 For 12 Story](image-url)
Table 10. The Summary of the results

| Member Type | Total number of Members | TNT With shear wall | TNT Without shear wall | EQ Without shear wall |
|-------------|-------------------------|---------------------|-----------------------|----------------------|
|             |                         | 5m Standoff 100 kg  | 5m Standoff 350 kg    | 5m Standoff 750 kg   |
| Columns     | 492                     | 27 (F)              | 260 (F)               | 422 (F)              |
| Beams       | 768                     | 138 (F)             | 436 (F)               | 650 (F)              |
| Total Fail Member | 1260            | 165                 | 692                   | 1072                 |

*F* = Failure

4.4. Comparing Cost and weight

Table 11 shows the cost and weight increase due to different weights of TNT in the cases of with and without shear wall. It is seen that the cost increase becomes more in case of shear wall absence. The last column of Table 11 shows the cost increase due to earthquake in the case of shear wall existence. It is worth noting that, in general, the increase in cost is the same as the increase in weight.

Table 11. Cost and Weight Summary

| TNT without shear wall | % 100 kg | % 350 kg | % 750 kg | % EQ |
|-----------------------|----------|----------|----------|------|
| TNT with shear wall   |          |          |          |      |
| % 100 kg             | 646      | 703      | 749      | 110  |
| % 350 kg             | 154      | 167      | 178      | 461  |
| % 750 kg             | 99       | 108      | 115      | 714  |

5. Conclusion

The applied blast load is considered an effective lateral force on the building side. This effect is obvious when three weights of TNT 100, 350 and 750 kg are compared with earth quake seismic load. Column-beam joints have been strengthened by additional reinforcement to withstand the lateral forces of the blast load. The following conclusions were reached:

1. In case of shear wall existence, taking into considerations the results of the stories 12th, 11th, 7th, 6th, 3rd and 2nd, the displacement increase due to blast load was more than that due to EQ. The displacement increase due to 100kg blast load was about 187%, 175%, 142%, 135%, 172% and 245% in comparison with EQ. While in case of 350kg blast load, the displacement development was about 451%, 425%, 351%, 334%, 448% and 626%. Finally, in case of 750kg blast load, displacement increase became about 1306%-1232%-1022%, 976%, 1287% and 1795%. From other hand, the maximum increase in displacement took place on the 12th and 11th stories about 1306-187% and 1232-175%, respectively for each TNT weight in comparison with EQ.
In case of shear wall existence, taking the results of the stories 12th, 11th, 7th, 6th, 3rd and 2nd into considerations, the increase in drift due to blast load was more than that due to EQ. Due to 100kg blast load, drift increased by about 618%, 386%, 194%, 177%, 125% and 102% in comparison with EQ. While in case of 350kg, drift increase became about 4236%, 2646%, 1344%, 1243%, 911% and 745%. Finally, in case of 750kg, drift increase became about 1469%, 918%, 461%, 425%, 314% and 259%. From other hand, the maximum increase in drift was seen on the 12th and 11th stories by about 4236-618% and 2646-386%, respectively for each weight TNT in comparison with EQ.

When increasing the TNT weight from 100 to 350 and then to 750kg, the displacement, drift and story shear increased rapidly, which accelerated the failure of the building (column and beam). In order to reduce the effect of TNT weights, there was an increase in the concrete section dimensions and the related steel reinforcement which increased cost and weight of the building. More specifically, resisting TNT without shear wall increased cost and weight of the building by about 646%, 167% and 115% due to 100, 350 and 750kg of TNT, respectively in comparison with EQ. TNT with shear wall increased cost and weight of the building by about 110%, 461% and 714% due to 100, 350 and 750kg, respectively in comparison with EQ.

References
[1] Bhatt, A C, Mevada, S V and Patel, S B 2016 Comparative Study of Response of Structures Subjected To Blast and Earthquake Loading Aditya C. Bhatt. et al. Int. Journal of Engineering Research and Applications, ISSN, pp.2248-9622.
[2] Bayoumey, A A and Attia, W A 2016 Assessment of existing structures under the action of gravity, earthquake and blast loads International Journal of Engineering Science and Innovative Technology (IJESIT) Vol. 5 pp.37-47.
[3] Kadhum, A K and Abdul-Razzaq, K S 2020 March. Effect of seismic load on steel frame multistory building from economical point of view. In AIP Conference Proceedings Vol. 2213 No. 1 pp. 020114). AIP Publishing LLC.
[4] Kadhum, A K and Abdul-Razzaq, K S 2018 Effect of Seismic Load on Reinforced Concrete Multistory Building from Economical Point of View International Journal of Civil Engineering and Technology (IJCIET) Vol.9 No.11 pp.588-598. http://www.iaeme.com/IJCIET/issues.asp?JType=IJCIET&VType=9&IType=11
[5] Abdul-Razzaq, K S 2015 Effect of Heating on Simply Supported Reinforced Concrete Deep Beams Diyala Journal of Engineering Sciences Vol. 8 No. 2 pp.116-133.
[6] Abdul-Razzaq, K S 2015 Effect of Heating on Shear Strength In Waste Plastic Lightweight Concrete by Using A New Test Specimen Diyala Journal of Engineering Sciences Vol. 8 No. 4 pp.189-210.
[7] Draganić, H. and Sigmund, V 2012 Blast loading on structures. Technical Gazette, Vol. 19 No. 3 pp.643-652.
[8] Koccaz, Z Sutcu, F and Torunbalci, N 2008 October Architectural and structural design for blast resistant buildings In The 14th World Conference on Earthquake Engineering. Beijing.
[9] Karlos, V and Solomos, G 2013 Calculation of blast loads for application to structural components. Luxembourg Publications Office of the European Union.
[10] Dusenberry, D O ed 2010 Handbook for blast resistant design of buildings. John Wiley & Sons.
[11] Shallan, O Eraky, A Sakr, T and Emad, S 2014 Response of building structures to blast effects. International journal of engineering and innovative technology Vol. 4 No. 2 pp.167-175.
[12] Ratna, S 2016. K. Analysis of RCC and Simcon Buildings Subjected To Blast Effects International Journal of Civil Engineering and Technology (IJCIET) Vol. 7 No. 4. http://www.iaeme.com/IJCIET/issues.asp?JType=IJCIET&VType=7&IType=4.
[13] TM5 – 1300 (1990). Design of Structures to resist the effect of accidental explosions Washington D C. U. S. Department of Army.
[14] UFC 2008 Unified Facilities Criteria 3-340-02: Structures to resist the effects of accidental explosions, Dept. of the Army, the NAVY and the Air Force, Washington DC, USA.
[15] Smith P D and Hetherington J G 1994 Blast and ballistic loading of structures Butterworth Heinemann.
[16] Ali Kifah Kadhum, Lina K. kadhum 2020 Architectural and Structural Design to Compare the Effect of Blast Load on Irregular Buildings From View Point Steel Braces International Journal of Psychosocial Rehabilitation Vol. 23 No. 06.
[17] Lina K. Kadhum, Ali Kifah kadhum 2020 Behavior of Architectural and Structural for Steel Fram Tall Building Subjected to Blast Loads Xi'an Jianzhu Keji Daxue Xuebao/Journal of Xi'an University of Architecture & Technology Vol. 12 No. 4 pp.5762-5773.
[18] Kinney G F and Graham K J 1985 Explosive Shocks in Air, Springer, Berlin.
[19] Newmark N M and Hansen R J 1961 Design of blast resistant structures, Shock and Vibration Handbook Vol.3 Eds. Harris & Crede, McGraw-Hill, New York.
[20] Kingery C N and Bulmash G 1984 Technical report ARBRL-TR-02555: Air blast parameters from TNT spherical air burst and hemispherical burst, AD-B082 713, U.S. Army Ballistic Research Laboratory, Aberdeen Proving Ground, MD.