Vulnerability Assessment Of Reinforced Concrete Columns Subjected To Blast Attacks

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Abstract: Blast attacks cause severe damage to exterior columns of a building and may lead to a progressive collapse of the entire building. When an exterior column is highly damaged due to blast loading, and cannot transfer its vertical loads safely to the adjacent columns, the whole structure may experience a progressive collapse. This represents one of the most common terrorist threats nowadays that cause a high number of injuries and fatalities. In the current study, the behavior of 12-stories reinforced concrete structure subjected to blast loading has been investigated. Structural analysis was carried out in three-dimensional nonlinear dynamic scheme using the Applied Element Method (AEM), where a focus was paid to the exterior columns of the building and their connecting structural members. The studied column was subjected to a charge weights of 500 lb. and 1000 lb. located at standoff distance 2, 4 and 6 meters away from the column. Structural response to the impulsive blast pressure was investigated, where eventually, an estimation to the minimum sufficient standoff distance required to avoid structural progressive collapse is obtained.

Index Terms: Progressive collapse, Blast loading, Column, Applied Element Method (AEM), Standoff distance, UFC 3-340-02.

1. INTRODUCTION
The protection of structures from potential terrorist attacks has become one of most severity issues for structural engineering. The awareness of designing the civilian facilities against the potential threats has raised due to the terrorist attacks happened last decades. There is an essential need for structural engineers to have a deeper understanding of the behavior of the structures subjected to blast loading. Since the terrorist attack of the Murrah federal office building in Oklahoma City, April 1995, attention to structural vulnerability assessment due to blast loading has been raised. Murrah federal building was partially collapsed due to terrorist attack by a bombed truck located at distance of 14 ft. On June 1996, a truck bomb attacked the eight-stories housing structure of the United States Air Force members in Khobar city, Saudi Arabia resulted in a partial collapse of the building. While the most famous progressive collapse event was the failure of the World Trade Centre towers on September 11, 2001. As a result of those terrorist attacks, many Agencies have adopted blast-resistant design structures guidelines, such as UFC 3-340-02 guidelines “Structures to resist the effect of accidental explosions” [1]. “Single degree of freedom structural response limit for antiterrorism design”[2].

2. RESEARCH OBJECTIVE
The objective of this study is to assess the progressive collapse vulnerability of mid-rise reinforced concrete framed structure, where its exterior columns are subjected to blast attack by estimating the minimum standoff distance required to prevent structural collapse. The column behavior and its damage pattern under blast loading is investigated.

3. PROPERTIES OF IDEAL BLAST WAVE [1], [3], [4], [5]
Explosion is a very rapid chemical reaction last for only milliseconds, causing a gigantic releasing of hot gases and energy and it results in an environment of a very high temperature and pressure. This released hot gases expands and propagates spherically with speed faster than the sound speed causing the front shock blast wave which contains a huge energy, this type of explosions is considered a free-air burst. When a detonation charge is located on, or at height very near to the ground surface, it is considered a surface burst explosion type. Where an initial incident shock wave is propagating away from the charge location, while this incident wave is reflected and reinforced by the ground surface resulting in a reflected wave. The reflected wave merges with the incident wave at the point of detonation to form a single hemispherical wave as illustrated in Fig. 1. The ideal relation between pressure and time for case of free-air burst is shown in Fig. 2. After the detonation took place at time t=0, the blast wave is formed and propagates away from the detonation source till it reaches a specific point at time t=tA. At the arrival time tA, the pressure will experience a sudden increase from ambient pressure value Po to the peak overpressure value Pso. As blast wave continue its propagation, the overpressure value Pso will decay in an exponential rate with time until reaching the ambient pressure Po after duration to, this duration is defined as the positive phase duration. The pressure values at the positive phase can be obtained at any time as proposed in (1) [6]. In addition, UFC 3-340-02 provides several charts to calculate all of the blast parameters.
\[ P(t) = P_{so}(1 - \frac{t}{t_0})e^{-\frac{t}{b}} \]  
(1)

Where,
- \( P_{so} \) is the peak overpressure,
- \( t_0 \) is the positive phase duration,
- \( b \) is a decay coefficient from a nonlinear fitting of experimental pressure time curve.
- \( t \) is the time elapsed, measured from the instant of blast arrival.

This phase is followed by a negative phase in which, the pressure values decrease to be less than the ambient pressure and record a minimum pressure value of \( P_{so}^- \). Finally, the pressure value increase until reaching the ambient pressure again after duration of \( t_{o-} \). It is desirable to mention that, most of the structural damage occurs during the positive phase due to the high pressure values compared to the negative phase. For that reason, the structural designers usually take the positive phase into account while ignoring the negative phase.

The distance between the detonation point and the studied structure is one of the most significant factors in the blast analysis, which is defined as the standoff distance \( R \). Where the standoff distance increases the peak pressure value of blast wave decreases and consequently the structural damage as shown in Fig. 3. If two different detonation charge weights \( (W_E) \) of the same explosive material and geometry where placed at the same scaled distance \( Z \) from the structure of interest, similar blast wave can be obtained at the point of interest. This is known as the scaling law as stated in (2) [1].

\[ Z = \frac{R}{\sqrt{W_E}} \]  
(2)

Where,
- \( Z \) = Scaled distance
- \( R \) = is the distance from the detonation source to the point of interest.
- \( W_E \) = Effective charge weight.

There are different types of the explosive materials; each explosive material has its blast pressure, heat of detonation \( (H_d) \) and output energy. In order to take into account the differences between these materials, a trinitrotoluene (TNT) Equivalency concept \( (W_E) \) is adopted. Heat of detonation \( (H_d) \) of some commonly used explosive materials are presented in Table 1. The relation between effective charge weight \( (W_E) \) and heat of detonation \( (H_d) \) of different explosives is presented in (3) [1].

\[ W_E = W_{EXP} \frac{H_d^{EXP}}{H_d^{TNT}} \]  
(3)

Where,
- \( W_E \) = effective charge weight
- \( W_{EXP} \) = weight of the explosive in question
- \( H_d^{EXP} \) = heat of detonation of explosive in question
- \( H_d^{TNT} \) = heat of detonation of TNT

4. REFLECTION OF THE BLAST WAVE

When the blast wave with incident peak pressure \( P_{so} \) contacts with obstacle surface (wall of building), it reflects producing a peak reflected pressure value \( P_r \) that is higher than the incident value. This can be explained as the shock wave propagates; it rapidly moves the air particles that impinging the surface, where the reflection of these particles is obstructed by the subsequent transferring air particles, resulting in a much higher values of reflected pressure than the incident one [4]. As shown in Fig. 4 the reflected wave is following the same behavior of the incident wave but with higher-pressure values. That is clarify that reflected peak pressure value is the governing pressure value in the blast design. The peak reflected pressure \( P_r \) can be calculated as an amplifying ratio of the incident pressure \( P_{so} \) using (4).

\[ P_r = C_r P_{so} \]  
(4)

Where,
- \( P_{so} \) is the peak incident pressure
- \( C_r \) is the reflection coefficient (depends on the angle of incidence \( \alpha \))

Fig. 1. Surface burst blast environment [1]
Fig. 2. Shock wave from detonation [4]

Fig. 3. Relation between standoff distance and peak pressure values [4]

Table 1. Heat of detonation of explosive materials [1]

<table>
<thead>
<tr>
<th>Name of explosive</th>
<th>Heat of detonation [MJ/kg]</th>
</tr>
</thead>
<tbody>
<tr>
<td>TNT</td>
<td>4.10-4.55</td>
</tr>
<tr>
<td>C4</td>
<td>5.86</td>
</tr>
<tr>
<td>RDX</td>
<td>5.13-6.19</td>
</tr>
<tr>
<td>PETN</td>
<td>6.69</td>
</tr>
<tr>
<td>PENTOLITE 50/50</td>
<td>5.86</td>
</tr>
<tr>
<td>NITROGLYCERIN</td>
<td>6.30</td>
</tr>
<tr>
<td>NITROMETHANE</td>
<td>6.40</td>
</tr>
<tr>
<td>NITROCELLULOSE</td>
<td>10.60</td>
</tr>
<tr>
<td>AMON./NIT. (AN)</td>
<td>1.59</td>
</tr>
</tbody>
</table>

Fig. 4. Incident and reflected pressure time histories [4]

5. APPLIED ELEMENT METHOD (AEM)

The Applied Element Method (AEM) is an advanced modelling technique following the approach of discrete cracking [7]. In AEM, structural components are divided virtually into small three-dimensional elements as shown in Fig. 5. The elements are connected by groups of shear and normal springs. Those springs are responsible for the transfer of shear and normal stresses between adjoining elements. Once the stresses in the connecting springs exceed their capacity, they fail and the adjoining elements eventually separate.

AEM is adopting nonlinear cyclic constitutive models as shown in Fig. 6. For concrete in compression, it follows elasto-plastic and fracture model of [8]. While a linear stress-strain relationship is assumed for concrete in tension. The model adopted for modeling the reinforcing bars is the model of Pinto to represent an envelope behavior [9].

The AEM is a stiffness-based method, in which an overall stiffness matrix is formulated and the equilibrium equations
are nonlinearly solved in order to calculate the displacement and deformation of the elements. The nonlinear equations are solved flowing step by step integrations [10], [11].

In this research, the Advanced analytical software Extreme Loading for Structures (ELS), which is following the AEM, is used is [12]. ELS has been verified to be a high fidelity nonlinear analytical software that can track the structural behavior for severe case of loading like earthquake, progressive collapse and blast loading. It can predict the structural behavior through all stages of the applied loads, starting at the elastic stage, followed by the stage of initiation and propagation of cracks, reinforcement yielding, then separation of the element, finally, collision of the element, collision with the ground and with neighboring structures [11].

For these reasons, researchers usually model only a portion of the structure of interest to reduce the analysis time and size [16].

The structure dimensions are (36x36 m), six equal bays in both direction each bay is six m span. The structure total height is 37 m above the ground, where the ground floor is 4 m height while the upper typical floors are 3 m height. The structure was subjected to typical office building loads [15]. All of the building stories have 2 KN/m² flooring load and 2.5 KN/m² as equivalent wall loads in addition to its own weight while the live load is 2.5 KN/m², except the roof floor is subjected to 3 KN/m² as live loads. The structural system of the building is solid slab structure with projected beams. The compressive strength of the concrete used is \( f'c = 30 \text{ N/mm}^2 \) while the main reinforcing steel grade is 36/52 for main steel and 24/35 for the stirrups.

It is commonly known, that the blast analysis is very complicated and requires experienced engineers in that field to achieve accurate simulation of the structural behavior. In addition, to perform a nonlinear blast analysis requires costly computer.

6. CASE STUDY

The structure under investigation is a reinforced concrete office building. The structure consists of twelve typical stories as shown in Fig. 7. It was designed to withstand the gravity and seismic loads in accordance with ACI 318-14 [14]. The structure was subjected to typical office building loads [15]. All of the building stories have 2 KN/m² flooring load and 2.5 KN/m² as equivalent wall loads in addition to its own weight while the live load is 2.5 KN/m², except the roof floor is subjected to 3 KN/m² as live loads. The structural system of the building is solid slab structure with projected beams. The compressive strength of the concrete used is \( f'c = 30 \text{ N/mm}^2 \) while the main reinforcing steel grade is 36/52 for main steel and 24/35 for the stirrups.

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The model contains only a single bay of the structure spanned from the centerline of right bay to the centerline of the left bay. Conservatively, only the three lower stories were modeled, as the number of the upper stories increases, the effect of the Virendeel action increases too; resulting in more structural stability [17]. The service loads of the removed stories were added as a lumped mass of 2150 KN on the exterior column C1 and 4200 KN on the interior column C2.

Along the bounding edge in Y-direction, the partial model is assumed to be restricted from movement about X (\( U_x = 0 \)) and rotation about Y (\( U_y = 0 \)) as this edge represents the centerline of the slab. The bounding edge along the X-direction is assumed to be restricted from movement about Y (\( U_y = 0 \)) and rotation about X (\( U_x = 0 \)). This assumption is justified by the existence of large part of the structure located in this area.
The column under investigation is the exterior column on the front façade of the building (C1) which is subjected to blast attack from bombed vehicle. Based on design of gravity and seismic loads, the exterior column dimension is 300×1000 mm 16 φ 18 longitudinal reinforcement (reinforcement ratio ≈ 1.5%) and φ 8@142mm stirrups.

7. ASSUMPTIONS
The following assumptions are followed in the current study:

- The studied structure is assumed to be subjected to blast attack from bombed vehicles with two different charge weights of trinitrotoluene (TNT) explosive material, 500 lb. (226.8 kg) and 1000 lb. (453.6 kg). The charge weight of 500 lb. is representing the capacity of a bombed car, while the 1000 lb. charge is the capacity of a bombed van [18].

- For civilian structures, which is commonly not to be designed against blast loading, the distance between the exterior column under investigation and the detonation source (bombed vehicle) is typically the width of the sidewalks. The standoff distance will be adopted in this study is 2, 4 and 6 meters.

- The surface blast pressure will be applied only to the exterior column under investigation (C1) on the ground and first stories and the exterior beams on the ground story.

- The blast pressure will be generated automatically by the analytical program (ELS) which is following the blast parameters of UFC 3-340-02.

- It is well known that material dynamic strengths for both concrete and reinforcing steel subjected to rapid loading (high strain rate) are higher than its static strength when subjected to normal strain rates (blast loading versus gravity and seismic loads) [19]. The material strengths under dynamic loads will be increased by a Dynamic Increase Factor (DIF). The DIF adopted in this study is following the values presented in UFC 3-340-02 guidelines for close-in design range [1].

- The column behavior and structural vulnerability to progressive collapse will be investigated, under charge weights located at 1.5 m high from the ground.

8. ANALYSIS RESULTS AND DISCUSSION
Firstly, the detonation charge weight of 500 lb. of TNT (bombed car capacity) was located at standoff distance of 2 m from the exterior column on the ground floor (300×100 mm) and a nonlinear dynamic analysis has been performed to investigate the structural behavior.
The analytical results showed that the model has collapsed. Fig. 8 shows successive 3D views for the structural behavior of the collapsed model.

The exterior column could not resist the applied pressure from the blast load and started to gain large displacements. There was concrete cover spalling, and stirrups rupture at most of the outer stirrups of the column, where the column experienced a large deformation until it failed. The longitudinal reinforcement bars were highly deformed but without rupture. Fig. 9 shows the deformations of the column’s reinforcement. The failure started at the exterior column on the ground floor caused the failure of the connected beams resulting in a progressive collapse of the structure as illustrated in Fig. 10.

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure8}
\caption{Successive 3D views of the collapsed model (case of 500 lb. at 2 m)}
\end{figure}

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure9}
\caption{Deformed shape of the column’s reinforcement at}
\end{figure}
Then, the standoff distance of 4 m was investigated, keeping the same charge weight of 500 lb. (TNT). In this case, as shown in Fig. 11, the structure was stable and no progressive collapse took place, where the column was able to resist the blast pressure with minor damage.

Secondly, the structural behavior was investigated against a blast charge of 1000 lb. TNT (bombed van capacity). The detonation charge placed at standoff distance of 2 m from the exterior column on the ground floor. The numerical results showed that the column could not resist the blast pressure and collapsed. The standoff distance was increased to 4 m, where the structure experienced a progressive collapse, too. Fig. 12 shows the deformed shape of the collapsed model for case 1000 lb. at 4 m standoff distance.
Finally, the standoff distance was increased to 6 m keeping the same charge weight of 1000 lb. The analytical results showed that the model was stable as illustrated in Fig. 13 with high damage in the ground floor front beams. The failure pattern of exterior column was investigated for the collapsed cases (case of 500 lb. at 2 m and 1000 lb. at 4 m). The results showed that the failure started when the column has gained large deformation in the direction of the wave propagation (Y direction); causing the formation of two inclined diagonal shear cracks at top and bottom of the column. Fig. 14 shows highly localized major principal strains at the top and the bottom of the exterior column on the ground floor, which represent the shear cracks.

![Fig. 13. Deformed shape at time=0.255 Sec (case of 1000 lb. at 6 m)](image)

![Fig. 12. Deformed shape at time=0.24 Sec (case of 1000 lb. at 4 m)](image)

Fig. 14. Principal strains in direction 1-1 at t=0.012 Sec for the collapsed columns on the ground floor
After this high damage in columns, the floor above the column highly deflected (in Z direction) caused large bending in the column with more excessive deformations. Fig. 15 illustrates the deflection history for the collapsed cases. The exterior columns on the upper floors have lost their vertical support due to the failure of the exterior column on the ground hence they became not capable of transferring the applied vertical loads. The internal normal forces in the exterior column on the first floor has dramatically decreased with time until reaching zero as shown in Fig. 16.

The behavior of the exterior column was investigated for the safe cases (case of 500 lb. at 4 m and 1000 lb. at 4 m). The column was stable and able to resist the blast pressure. As shown Fig. 17, the column experienced instantaneous peak displacement in the direction of the wave propagation (Y direction) at time $5 \times 10^{-3}$ and $6 \times 10^{-3}$ Sec with a magnitude of 6.26 mm and 5.8 mm for case of 500 lb. and 1000 lb., respectively. This peak displacement value was followed by a rapid decay then the displacement values vibrating with average permanent deformation of 1.05 mm and 0.5 mm for case of 500 lb. and 1000 lb., respectively. Fig. 18 shows the maximum displacement contours in the direction of wave propagation (Y direction) at the peak response time. This peak response comes relatively late after the detonation time ($1 \times 10^{-5}$ Sec) due to nature of the blast load as an impulsive load. The column showed vibrated deflection in Z direction with averages of 1.1 mm and 1.15 mm for an element at the top of the column for case of 500 lb. and 1000 lb., respectively as illustrated in Fig. 19. As the structure was stable, the upper columns were able to transfer the vertical loads safely to the columns on the ground floor. Fig. 20 shows the internal normal forces in the exterior column on the ground floor. The blast loading caused instantaneous increase in the internal force then the values vibrated about an average value equal to initial value of the internal force.

**Fig. 15. Deflection history in (Z) for an element at the top of the collapsed column on the ground floor**

**Fig. 16. Internal normal force at the exterior column on the first floor for collapsed cases (500 lb. at 2 m and 1000 lb. at 4 m)**

It is can be obviously observed that for the safe cases, the exterior column behavior and response is convergent. Referring to the concept of scaled distance (Z) and using (4), the scaled distances are 0.66 and 0.78 m/kg$^{1/3}$ for case of 500 lb. at 4 m and case of 1000 lb. at 6 m, respectively. It can be concluded that in the current case study, for blast loading with scaled distance value in that range (0.66 ~ 0.78 m/kg$^{1/3}$), the column will not experience progressive collapse and will have peak response values convergent to the values obtained for the case of 500 lb. at 4 m and case of 1000 lb. at 6 m.

**Fig. 17. Displacement history in (Y) for an element at the mid height of the exterior column on the ground floor**

**Fig. 18. Maximum displacement contours in Y (m)**
9. CONCLUSIONS
A three dimensional nonlinear dynamic analysis has been
published using Extreme Loading for Structures program.
Based on this case study of 12 stories reinforced concrete
structure with solid slab floor system, it can be concluded
that: The standoff distance (width of the sidewalks) is one of
the most controlling parameters on the structural behavior
under blast loading. The failure pattern for columns subjected
to blast loading is controlled mainly by shear diagonal
cracking. The collapsed cases showed formation of two
inclined diagonal shear cracks accompanied by failure in the
stirrups at the top and the bottom of the collapsed column.
Column confinement reinforcement is an important parameter
for controlling the column behavior under blast loading. The
collapsed cases showed vast rupture of the stirrups while the
longitudinal bars did not. Therefore, the column confinement
reinforcement should be intensified at the top and the bottom
depth of the exterior columns to mitigate the progressive
collapse occurrence. Whereas the blast analysis is costly and
time consuming, the number of blast analysis cases under
investigation can be reduced based on the concept of scaled
distance (Z). For a column with known cross-sectional
dimensions and reinforcement, the column will almost have
the same behavior and peak response for the scaled
distances (Z) with equal values. For blast attack of 500 lb.
charge weight of TNT explosive material (bombed car), the
progressive collapse behavior can be avoided if the standoff
distance (sidewalk width) has been increased to 4 m or more.
While for blast attack of 1000 lb. charge weight of TNT
explosive material (bombed van), the progressive collapse
behavior can be avoided if the standoff distance has been
increased to 6 m or more.

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