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Design of Structures Subjected to Blast Loads: Analysis and Design Review

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Abstract: When designing structures to withstand explosions, the main goals are to minimize the number and extent of occupant injuries and to reduce the chance of catastrophic damage to structures. Although there is uncertainty in the source, extent, and location of explosions, the assessment of blast loading and structural performance is important when designing blast-resistant structures. This study is a review of the literature on the prediction of blast loads, structural modeling and analysis, and design criteria for structures to resist explosions. The paper provides in one concise document the general guidelines, references, and tools that structural engineers and researchers need to analyze and design structures subjected to blast loading. References on the topics discussed in this work are provided for more detail.

Keywords: Blast loads; Blast-resistant structures; Structural analysis; Blast design criteria

1. Introduction

Interest in the behavior of structures subjected to blast loading has increased over the last few decades as terrorist attacks have increased around the globe. Attacks on the World Trade Center in New York City in 1993 and the Murrah Federal Building in Oklahoma City in 1995 showed the great damage that could happen due to a blast. In both attacks, structural failure caused more casualties and injuries than the blast wave itself [1]. Normally, conventional structures (many are moment-resisting frames) are not designed to tolerate blast loads, which are very high compared to service loads and happen in less than a second. For instance, 4.53 kg (10 lbs) of trinitrotoluene (TNT) at a distance of 15.24 m (50 ft) causes a peak pressure of roughly 17.24 kPa (360 psf) over a very short duration compared to the natural period of the structure. In comparison, the design snow load in the Midwest ranges from 0.24 kPa (5 psf) to 2.39 kPa (50 psf) [2]. Thus, a small-charge explosion could cause catastrophic local or global failure of the structure. Analysis and design of blast-resistant structures requires good knowledge of the blast phenomena, dynamic response of structures, and design requirements. However, threats cannot be predicted accurately, and it is not possible to design a fully protected structure. Thus, acceptable damage to the structure is expected according to a predefined level of protection [3].

The purpose of this paper is to review the literature and provide the reader with a concise reference for the analysis and design of structures for blast resistance. It provides basic considerations for blast load calculation, structural modeling and analysis, and design criteria. This study is limited to surface bursts where the explosive charge is detonated close to ground level and the structure is regularly shaped.

The paper is organized into eight sections. Following this introduction, Section 2 provides an overview of the literature. Section 3 discusses the blast phenomena and ways to assess blast load and its duration, and Section 4 provides a review of material strength under a high strain rate condition. Section 5 discusses stress increase and reduction factors, and Section 6 discusses modeling and analysis of structural components and systems subjected to blast loads. In Section 7, design criteria for structural components and systems are discussed, and Section 8 provides a definition of progressive collapse that designers should be aware of. References on all topics are provided for more detail.

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2. Literature Overview

The subject areas of blast load prediction and blast-resistant design are quite broad. In this review, many references have been used to collect information on these subject areas and provide the reader with a concise document. This section provides a brief overview of the key references used in this study along with some information discussed in each reference. The U.S. Department of Defense (DoD) publication [4] provides a manual for evaluating blast loads and design criteria for members and structural systems. It is considered one of the most important references for blast-resistant design. The American Society of Civil Engineers (ASCE) [5] prepared a report to provide guidance for blast resistance of petrochemical facilities. Pape et al. [6] published a three-part paper on the blast phenomena and its effect on structures. The work provides a practical overview of types of explosions, prediction of explosion effects, and methods for analysis under blast conditions. The ASCE also wrote a standard [7] that provides planning, design, construction, and assessment requirements for existing and new structures subjected to blast loading. Draganić, H., Sigmund [8] discuss the challenges in defining blast loads, and investigates vulnerability assessment and risk mitigation using standard structural analysis software. The study focuses on utilizing conventional software like SAP2000 for simulating blast effects on structures, employing pressure-time history records derived from literature calculations. In this research, a numerical example is studied. Gilsanz et al. [9] wrote a guide published by the American Institute of Steel Construction (AISC) that focuses on blast resistance and progressive collapse mitigation of steel structures. It provides a few detailed design examples. In light of the rising number of terrorist acts, Jamakhandi and Vanakudre [10] tackle the important topic of blast loads on buildings. It highlights the need for blast loads to be considered as dynamic forces in structure design, similar to winds and earthquakes. The study emphasizes the significance of comprehending these components for efficient blast-resistant design by elucidating explosives and explosion processes. It outlines methods from an architectural and structural standpoint for improving building security against explosives. The structural reaction is greatly impacted by increased charge weight and decreased standoff distance, according to the results, which suggests regular frame models for the best blast resistance and economical design. Cheng et al. [11] provides an extensive overview of the dynamic response, damage assessment, and mitigation strategies for tunnels under explosion loads. It highlights the critical role of road tunnels in transportation networks and the potential risks they face from terrorist attacks, accidental explosions, and construction activities. The review covers various explosion scenarios, blast wave characteristics, tunnel response analysis methods, damage assessment criteria, and mitigation measures. Key findings include the need for improved prediction methods, studies on tunnel response in different media, exploration of damage modes, assessment methods, and development of cost-effective mitigation measures. Goel and Matsagar [3] discussed different strategies for blast mitigation and the mechanics of sacrificial blast walls using different materials. Books by Smith and Hetherington [12], Bangash and Bangash [13], Cormie et al. [1], and Dusenberry [14] provide detailed information on the analysis and design of buildings subjected to blast conditions. This paper summarizes the most important analysis and design information provided in these references and others with a MATLAB code to predict blast loads based on the method described by the DoD [4].
(https://www.mathworks.com/matlabcentral/fileexchange/70105matlab_code_blast_load_dod_2008).

3. Prediction of Blast Loading

This section provides the necessary background and references to calculate external blast loading. Although there is uncertainty in predicting the size, type, and location of the explosive, calculation of blast loads is essential in the design of blast-resistant structures.

3.1 Blast Phenomena

The explosion generates hot gas that can be at a pressure of 10000-30000 MPa (1450-4351 ksi) and a temperature of 3000-4000°C [12]. If a blast happens in the air, the high-temperature gas that is produced by an explosive charge expands spherically to take up the available space. In other words, the violent expansion forces the surrounding air out of its occupied space. Simultaneously, the air around the explosion expands and its molecules pile up. What is known as a blast wave occurs next, and it carries a large amount of energy. As the wavefront moves away from the source of the explosion, its pressure decreases at an exponential rate until it falls to the normal atmospheric pressure; this is called the positive phase. After that, it decreases to less than the atmosphere pressure (negative phase) and finally back to the ambient value (see Fig. 1). Thus, the blast pressure is a time history loading. In Fig. 1, P_{so} is the peak overpressure or the incident pressure, P_o is the ambient pressure, P_{so}^- is the minimum negative pressure, P_r is the reflected pressure, P_r^- is the minimum negative reflected pressure, t_a is the arrival time, t_o is the positive phase duration, t_o^- is the negative phase duration, i_s is the positive reflected impulse, and i_s^- is the negative incident impulse. When the blast wave travels parallel to a surface and is unimpeded by any object, free-field (side-on or incident) pressure is applied to the surface (see Fig. 1 (a)). When a surface is struck by a blast wave perpendicularly or at an angle, reflected pressure is applied to the surface.

Friedlander's exponential equation is usually used to describe the pressure-time history of a blast wave [1]:

$$P_s(\mathrm{t}) = P_{so}\left(1 - \frac{t}{t_o}\right) e^{-bt/t_o} \tag{1}$$

where b is the decay coefficient of the waveform (calculated through a nonlinear fitting of an

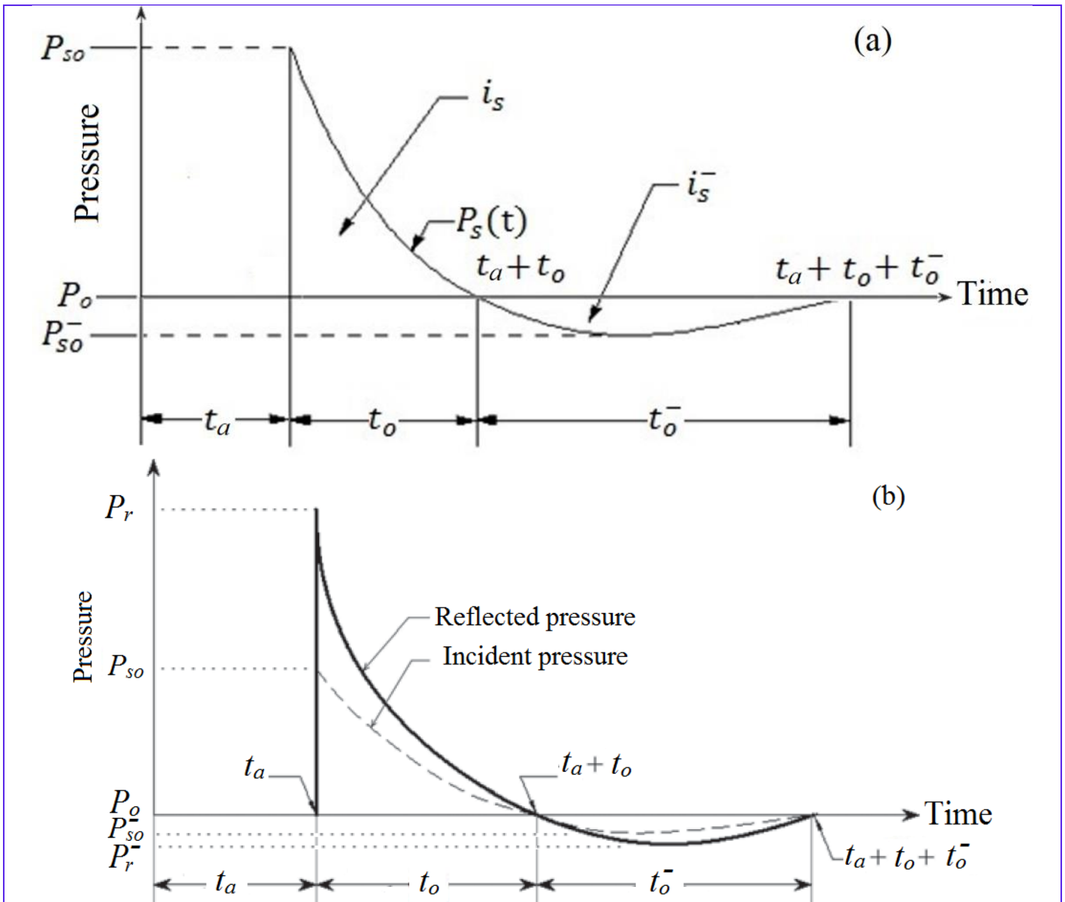

Fig. 1. Idealized pressure-time profile for the blast wave: (a) free-field pressure, (b) reflected pressure (modified from [4])

experimental pressure time curve over its positive phase).

There are three techniques to calculate blast loads [1]:

First principle methods: These are the most accurate methods that involve solving partial differential equations based on computational fluid dynamics (CFD). The CFD models determine a numerical solution to fluid (air) flow equations. These equations are based on the principles of conservation of mass, momentum, and energy. The reader is referred to the work of Cormie et al. [1] and Zienkiewicz et al. [15] for more details on this topic. There are many computer codes available for modeling the detonation of explosives, such as LS-DYNA [14], ABAQUS [17], and Air3d [18]. The blast loads calculated with CFD are used to compute the structural response. However, when the structure is expected to move significantly due to the blast event, the blast wave and the structural response could be coupled to obtain more accurate results [8].
Semi-empirical or phenomenological methods: These are simplified methods that represent the essential physical phenomena of the explosion.
Empirical methods: These are based on an analysis of the experimental data [3]. Scaling Law is the most common empirical method used in the analysis and design of blast-resistant structures. Blast parameters such as incident and reflected pressures are functions of the scaled distance (Z). Report UFC 3-340-02 developed by the DoD [4] provides guidelines to predict blast loads using the empirical method. ConWep [19] and ATBlast are examples of computer programs that are widely used to determine blast wave parameters. They are an implementation of the method described by the DoD [4]. This method is widely used in analysis and design of strctures subjected to blast loading. The selection of an analysis method depends on the project requirements and the type of components to be designed [7]. Blast load decreases rapidly with distance. Therefore, based on the distance from the source of the blast and the angle of the incident, blast loads and their durations can change considerably over the surface of the structure. The common approach is to divide the surface into a grid and then calculate blast loads and their durations at the center of each section of the grid.

3.2 Scaling Law

The distance of the structure from the detonation point is an important parameter in calculating the blast loads. The Hopkinson-Cranz scaling approach (cube-root scaling) is the most widely used approach for blast wave analysis for spherical explosions. The scaling distance is defined as follows:

$$Z = \frac{R}{\sqrt[3]{W}} \tag{2}$$

where Z is the scaled distance, R is the distance from the detonation source to the point of interest expressed in meter (m), and W is the charge mass expressed in kilograms (kg) of TNT.
There are many types of explosives. TNT was chosen to be the blast parameter, so an equivalent TNT weight needs to be computed in order to use Eq. (2). Equation (3) below is used to find the equivalent weight of TNT, and Table 1 shows the conversion factors for some explosives [4].

$$W_e = W_{exp} \frac{H^d_{exp}}{H^d_{TNT}} \tag{3}$$

where W_e is the equivalent TNT weight, W_{exp} is the weight of the explosive, H^d_{exp} is the heat of detonation of the explosive, and H^d_{TNT} is the heat of detonation of the TNT.
Identifying explosive size is an important part of the threat assessment process. Table 2 shows the estimated ranges of explosives. Bangash and Bangash [13] categorize explosives as small, medium, large, and very large (Table 3).

Table 1. Heat of detonation for some explosives [4].

Explosive name	Heat of detonation, m-kg/kg (ft-lb/lb)
TNT	0.600 E+06 (1.97 E+06)
Composition B	0.655 E+06 (2.15 E+06)
Composition C4	0.677 E+06 (2.22 E+06)
RDX	0.692 E+06 (2.27 E+06)

HMX	0.692 E+06 (2.27 E+06)

Table 2. Estimated Quantities of Explosive [20]

Type	Charge weight
Luggage	4.54-45.36 kg (10-100 lb) TNT
Automobile	45.36-204.12 kg (100-450 lb) TNT
Van	204.12-1814.37 kg (450-4,000 lb) TNT
Truck	1814.37-45359.24 kg (4,000-100,000 lb) TNT

Table 3. Size of Explosive [13].

Type	Charge weight
Small	Up to 5 kg (11 lb) TNT
Medium	Up to 20 kg (44 lb) TNT
Large	Up to 100 kg (220 lb) TNT
Very large	Up to 2500 kg (5512 lb) TNT

3.3 Explosion and Blast-Loading Types

There are three types of explosions, as shown in Fig. 2:
i- Free-air bursts: In this case, the charge is detonated in the air away from any reflecting surface. The blast waves can be characterized by a spherical wave that moves outward from the source and impinges directly onto the structure.
ii- Air bursts: The explosive charge is detonated in the air. The blast waves propagate spherically outward from and impinge on the structure after having interacted first with the ground. What is called Mach reflection might occur because of the interaction of the blast wave and the reflected wave.
iii- Surface bursts: The explosive charge is detonated near the ground surface. The blast waves immediately interact locally with the ground and then propagate hemispherically outwards, impinging on the structure.

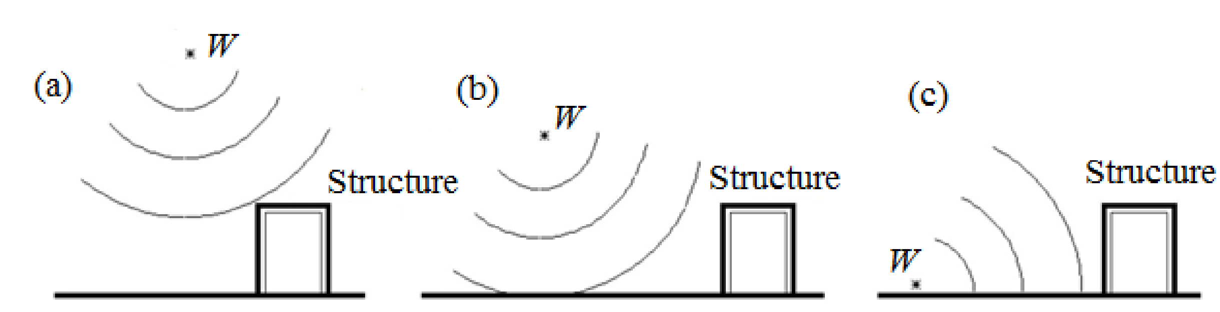

Fig. 2. Types of explosions: (a) free-air burst, (b) air burst, (c) surface burst (modified from [21].

3.4 Blast Wave Reflection

The blast waves will reflect when they impact an object made of a medium denser than that carrying the wave. In this case, the pressure acting on the structure is not the same as the incident peak pressure (P_{so}). In fact, the reflected pressure could be several times greater than the incident pressure, as shown in Fig. 1 [1].

In the discussion above, the angle of the incident (α) is taken as zero. When $\alpha = 90^o$, the blast wave travels parallel to the surface. That is, there is no reflection, and the structure is loaded with side-on pressure that is equal to the incident overpressure. If α is between 0^o and 90^o, either regular or Mach reflection happens. The effect of the angle of the incident on the reflection coefficient ($C_{r\alpha} = \frac{P_r}{P_{so}}$) is shown in Fig. 3 [21]. The influence of the angle of incident can be ignored for the large pressure, and the structure can be studied under a normal reflected pressure, which is a conservative approach. In general, one can use Fig. 3 to determine the reflection coefficient. The mach reflection is a complex process. When the reflected wave catches up with the incident wave, the so-called Mach stem occurs. This is the reason for the jump in the angle of the incident-reflected pressure curves shown in Fig. 3. Conventionally, facades are assumed to be perfectly rigid so that they perfectly reflect the blast wave front. In reality, however, facades displace when the blast wave impinges on them. This displacement reduces the effectiveness of the reflected pressure.

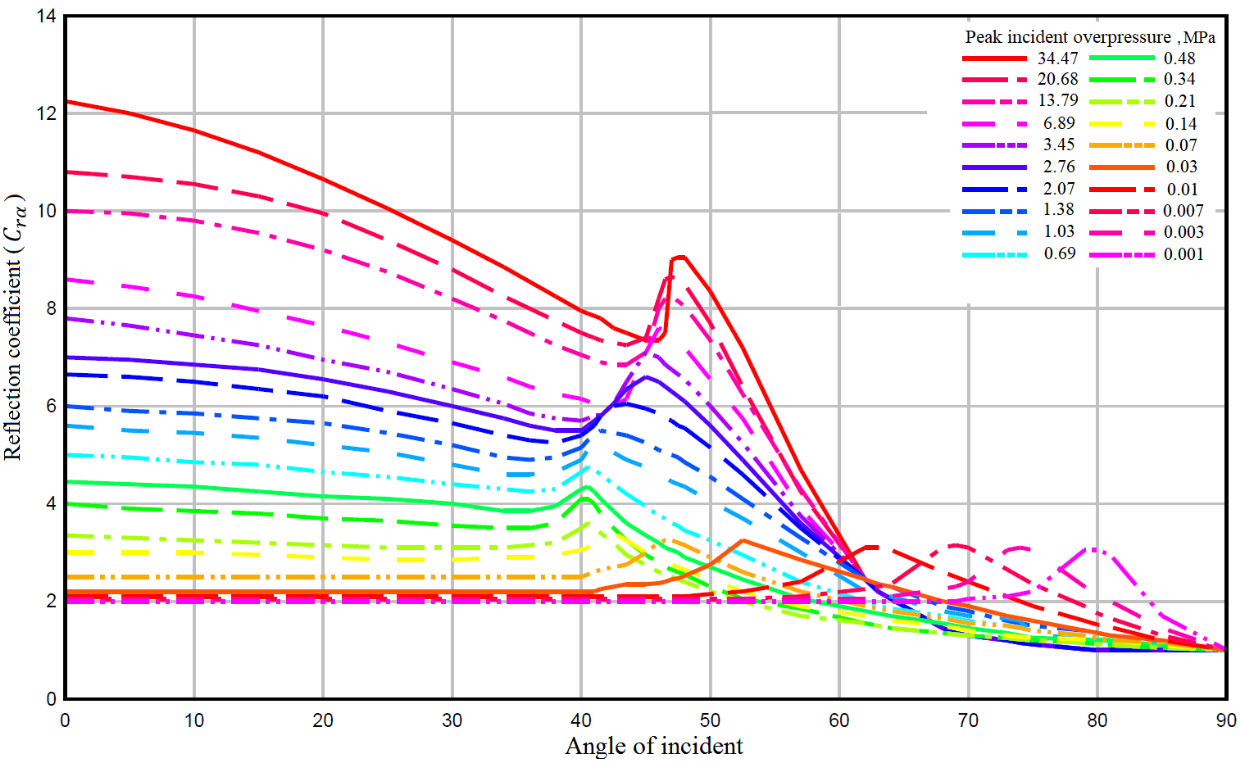

Fig. 3. Influence of angle of incident on the reflection pressure (modified from [4]).

3.5 Surface Burst and Loading

When the explosive charge is placed close to the ground, a modification must be made to the charge weight. The incident wave is reflected immediately from the ground and interacts with the blast wave. This is called a hemispherical burst. Practically, due to the creation of a crater, some energy absorption takes place from the ground. Figs. 4 and 5 show the blast wave parameters of a hemispherical wave of TNT charge for the positive and negative phases, respectively. The wave parameters are presented on the y-axis while the x-axis represents the scaled distance (Z).

In Fig. 4, W is the weight of the charge, P_{so} is the incident peak overpressure, P_r is the reflected pressure, i_r is the positive reflected impulse, i_s is the positive incident impulse, t_a is the arrival time, t_o is the positive duration, U is the wave speed, and L_w is the wavelength. They are presented on the y-axis, while the x-axis represents the scaled distance Z. In Fig. 5, the superscript "−" refers to the negative phase.

After calculating the scaled distance for a specified distance and charge weight, Figs. 4 and 5 can be used to determine the positive and negative parameters to plot the equivalent pressure time history for the front, roof, and side and rear walls (Fig 6). Numerical examples showing all the steps to find the equivalent load time history are available in the work of the DoD [4], Gilsanz et al. [7], and Karlos et al. [21]. A MATLAB code that follows the methods presented by DoD [4] is provided. The code can be used to plot the triangular shape of the pressure time history (like those shown in Fig. 6). Note that the scaled distance must be within the range of Figs. 4 and 5. For close-in explosions, this simplified approach is not allowed. CFD or test data should be used to find the blast loading, and explicit nonlinear dynamic analysis should be performed to consider breach, diagonal tension, direct shear, and spall failure mode.

Fig. 6 shows the simplification of the pressure-time history profile of the blast wave (Fig. 1). In Fig. 6, w is the width of the front wall and the back wall, H is the height of all walls, L is the length of the side wall, P_r is the reflected pressure, P_{so} is the incident peak, C_D is the drag coefficient (C_D is 1 for the front wall), q_o is the incident dynamic pressure, i_r is the total reflected pressure impulse, t_{rf} is the duration of the reflected pressure, t_c is the clearing time, t_{orf} is the actual positive phase duration, and t_o is the positive phase duration. In the roof and side wall loading figure, L_w is the wavelength, C_{Ef} is the equivalent load factor, P_{sof} is the incident pressure, C_{Df} is the drag coefficient at point f, q_{of} is the dynamic pressure, t_f is the time when the blast wave reaches the point f, t_{df} is the time when the peak equivalent uniform pressure is reached, t_{of} is the actual positive phase duration, and t_{opf} is the positive phase duration. In the rear wall loading figure, the notations are similar to the roof and side wall loading figure, except that point b is used instead of point f. The superscript "−" refers to the negative phase.

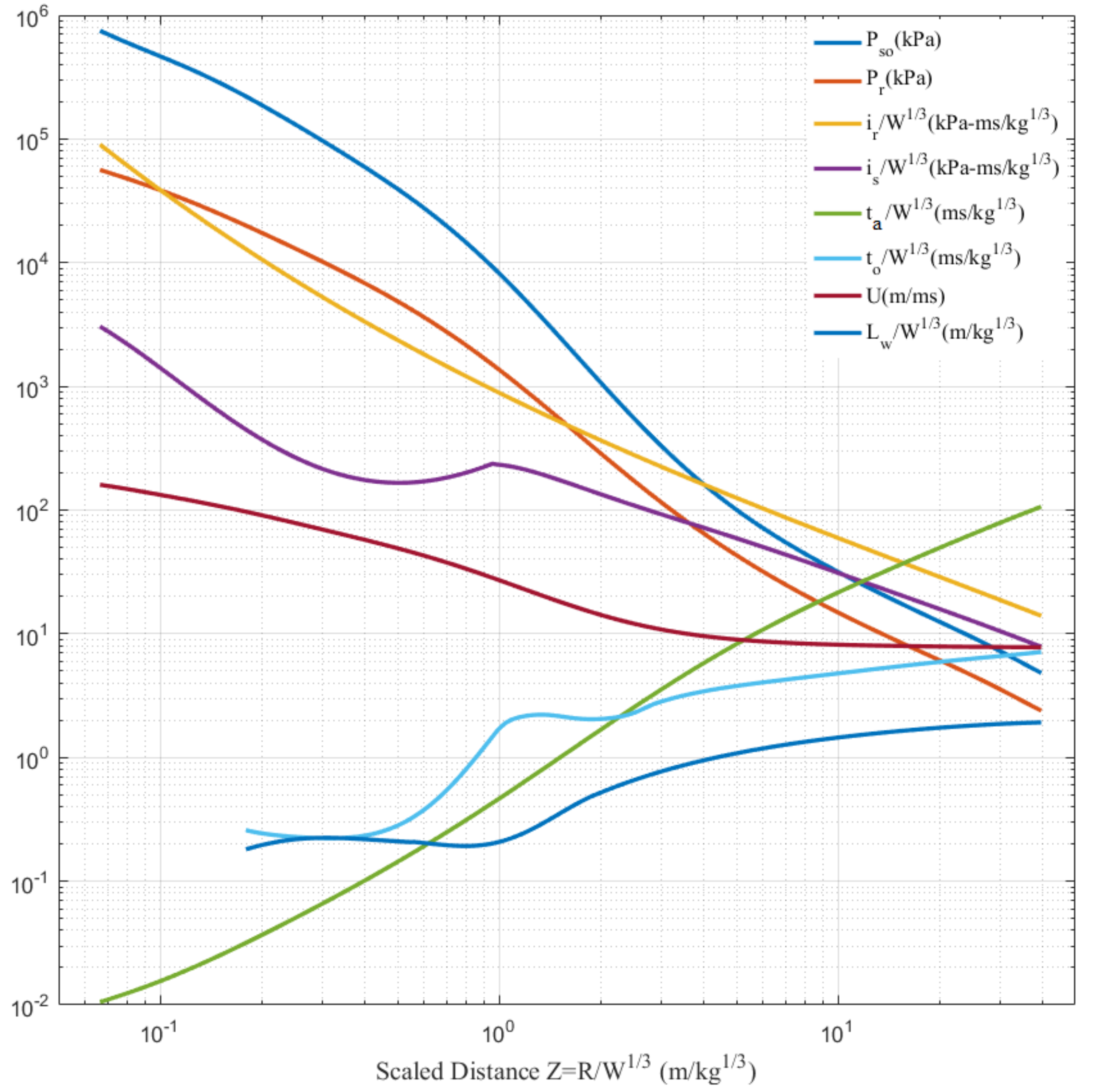

Fig. 4. The positive phase parameters of the hemispherical wave of TNT charges (modified from [4])

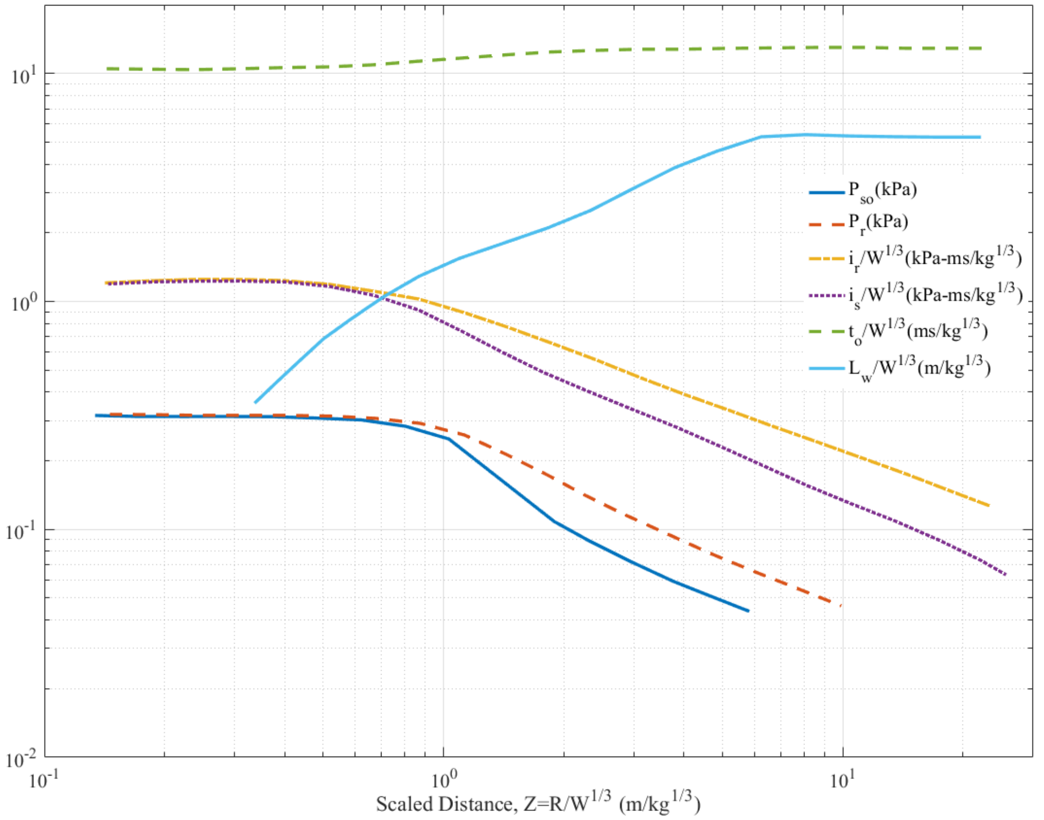

Fig. 5. The negative phase parameters of the hemispherical wave of TNT charges [21].

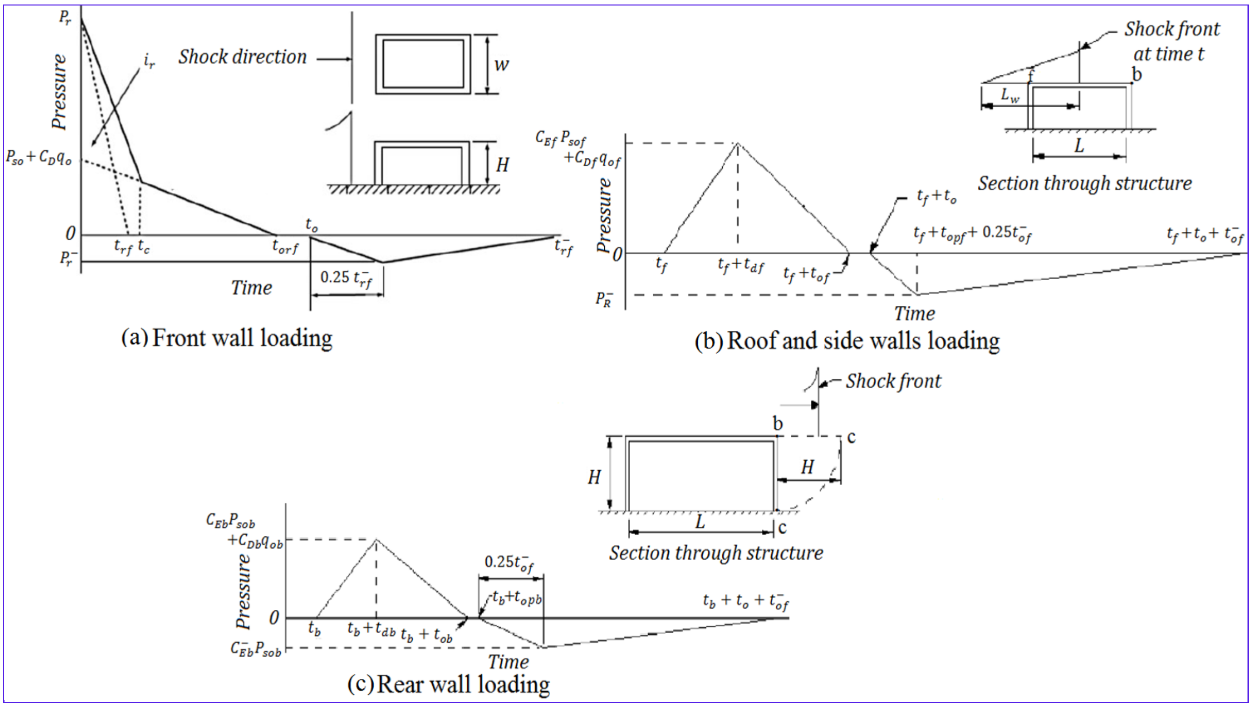

Fig. 6. Triangular assumption of pressure time history on the (a) front wall loading, (b) roof and side wall loading, (c) rear wall loading (modified from [4]).

3.6 Negative Phases

Compared with the positive phase, the negative wave has a longer duration and a lower pressure magnitude, as shown in Fig. 1. It reduces the effect of the peak response, and it is usually ignored in design because the main structural damage results from the positive phase loads [21]. However, its effect should be examined for members that have a shorter fundamental period in comparison with negative load duration [7].

3.7 Internal Pressure

In the previous sections, blast pressure has been discussed with the assumption that there are no openings in the walls. Structures, however, have windows and doors that may leak pressure into the building, causing a reduction in the effective new load on the external walls. Internal pressure is important in evaluating the effects on personnel and the internal damage. The internal pressure effect is usually ignored when the openings are small [7]. The DoD [4] provides a procedure to evaluate internal pressure.

4 Material Design Strength

Steel and reinforced concrete are the most commonly used materials in the construction of blast-resistant structures, but masonry and timber are permitted. For a close and high-impulse blast event, concrete structures are generally used to provide protection against fragments and to limit deformation [4][6].

The ductility of members (or general structures) is an essential factor in blast design: the greater the ductility, the greater the members' resistance to failure. Low-carbon steel and properly reinforced concrete are suitable for blast-resistant design because they can deform beyond the elastic limit without rupturing [5].

The mechanical properties of material under high strain rate loadings such as blast loads are different from low rate and static loads. Generally, materials become stiffer under high-rate loadings, which leads to an improvement in their mechanical properties. Also, in blast design, it is allowable to use the expected actual strength of the material instead of the minimum specified values.

4.1 Material Properties of Steel

The effects of high strain rate on some of the mechanical properties of steel are summarized as follows:

i- The modulus of elasticity (E_s) remains the same. The yield strength (f_y) and ultimate tensile strength (f_u) increase the dynamic yield strength (f_{dy}) and the dynamic ultimate strength (f_{du}), respectively. Fig. 7 shows the effect of increasing strain rate on steel.

ii- Dynamic increase factors (DIF) are used to modify the static strength due to high-rate dynamic loads. Table 4 presents the values of DIF for different types of steel and different strain rates.

The average yield stress of steel of grades 50 or less is about 10% higher than the stress value specified by ASTM. Thus, for blast-resistant design, the yield stress is 1.1 times the minimum yield stress. This factor is called the strength increase factor (SIF) or the average strength factor (ASF). The SIF should not be used with high-strength steels [7].

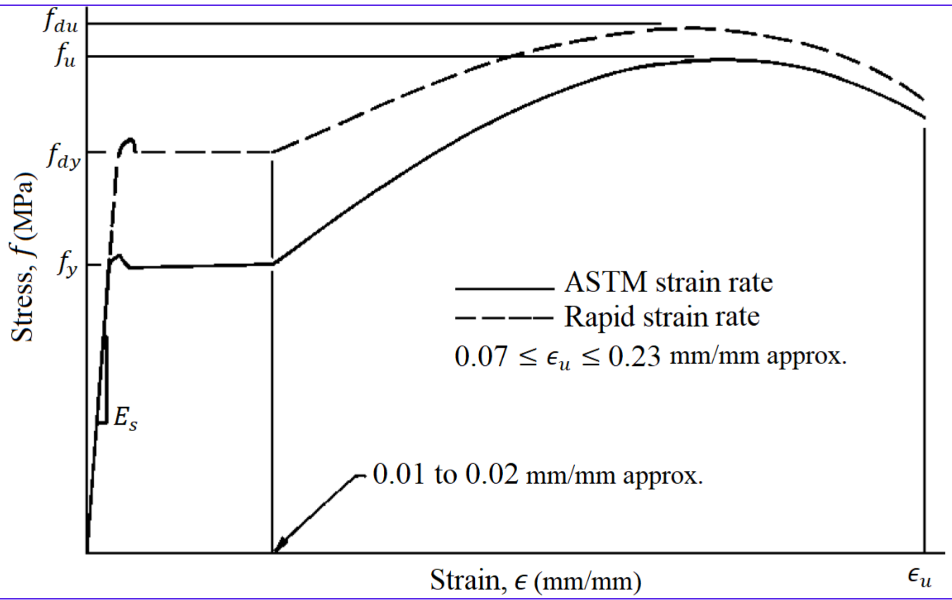

Fig. 7. The effect of high strain rate on the mechanical properties of steel (modified from[4]).

Table 4. Dynamic Increasing Factor (DIF) for Yield Stress and Ultimate Stress for Structural Steel [4].

Steel type	Yield DIF				Ultimate stress DIF
	Bending		Tension or compression		
	Low Pressure ($\dot{\varepsilon}$=0.1 mm/mm/sec)	High Pressure ($\dot{\varepsilon}$=0.3)	Low Pressure ($\dot{\varepsilon}$=0.02)	High Pressure ($\dot{\varepsilon}$=0.05)	
A36	1.29	1.36	1.19	1.24	1.10
A588	1.19	1.24	1.12	1.26	1.05
A514	1.09	1.12	1.05	1.07	1.00

4.2 Material Properties for Reinforced Concrete

Similar to steel, reinforced concrete shows improvements in its mechanical properties when it is subjected to blast loadings. The effect of high strain rates on reinforced steel and concrete are shown in Figs. 7 and 8, respectively. Table 5 provides the DIF values of reinforced steel and concrete. The SIF of reinforced steel is discussed in Section 4.1, and the SIF for the compressive strength of concrete is 1.1 [6].

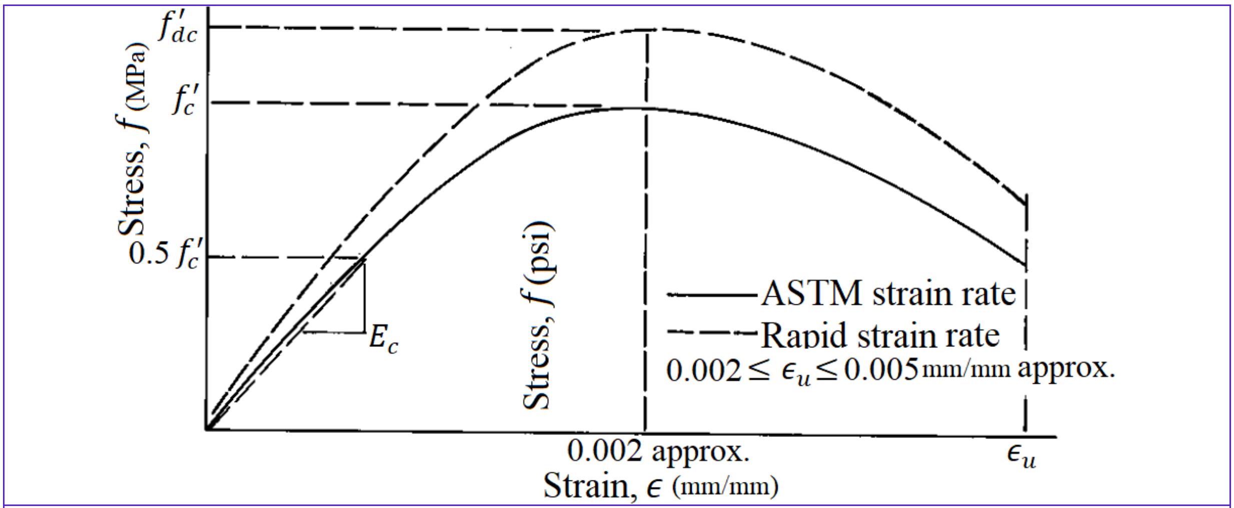

Fig. 8. The effect of high strain rate on the mechanical properties of concrete (modified from [4])

Table 5. Dynamic Increase Factor for Reinforced Concrete Design [4].

Type of stress	Reinforced bars		Concrete
	Yield stress	Ultimate stress	
Bending	1.17	1.05	1.19
Diagonal tension	1.00	-	1.00
Compression	1.10	-	1.12

4.3 Plastic Hinge

In designing for blast loading, some members are allowed to have plastic behavior to achieve an economical design. Therefore, it is important to understand the local performance of members and the global performance of the structure when one or more plastic hinges start to form. Also, the locations and modeling of the plastic hinges are important. To allow a plastic hinge to form in a component, lateral supports must be provided to prevent premature buckling. It is good practice to design columns to remain elastic to prevent extended structural failure [7]. This is the "strong column, weak beam" approach. That is, beams are forced to fail before columns.

A plastic hinge is formed at the point of maximum stress. It starts when the outer fiber reaches the material yield limit. Then, the interior of the section starts to yield gradually as the load increases and the stress-strain relationship becomes nonlinear. At other locations, the resistance continues to increase as the load increases. That is, some points respond plastically while others respond elastically, and elastic-plastic conditions occur [5].

Modeling the nonlinear behavior of sections depends on the material to be used and the internal force in the section. For example, an ideal elastic-plastic behavior is accepted in the design of a single-degree-of-freedom (SDOF) system. Fig. 9 shows the idealized resistance-deflection curve, where R_m is the ultimate dynamic resistance, X_E is the deflection at the limit of the elastic range, K_e is the elastic stiffness, and X_m is the maximum allowed deflection corresponding to the ductility ratio (u) or rotation (θ) given in Section 5. In more complex scenarios such as a steel member subjected to tension and compression, a plastic hinge can be modeled using FEMA 356 [22], as shown in Fig. 10 and Table 6, where a, b, and c are hinge parameters that are functions of the elongation, P_n is the tensile strength, F_{cr} is the critical buckling load, Δ_T refers to axial deformation at tensile yield load, and Δ_C refers to axial deformation at bucking yield load.

When there are axial force and bending moments in one or two directions, the plastic hinge may be represented using a P-M-M yield surface [23]. Here, P is the axial force, and M-M refers to the minor and major bending moments.

The yield surface defines the strength of the material under biaxial stress. Any elastic-plastic material has a yield surface. When the stress point is on the yield surface, the material has yielded, and its behavior is elastic-plastic. But when the stress point is inside the yield surface, the material is elastic. Stress points outside the yield surface are not allowed. Software such SAP2000 [24] implements what is called Parametric P-M2-M3 based on the P-M-M yield surface method [25].

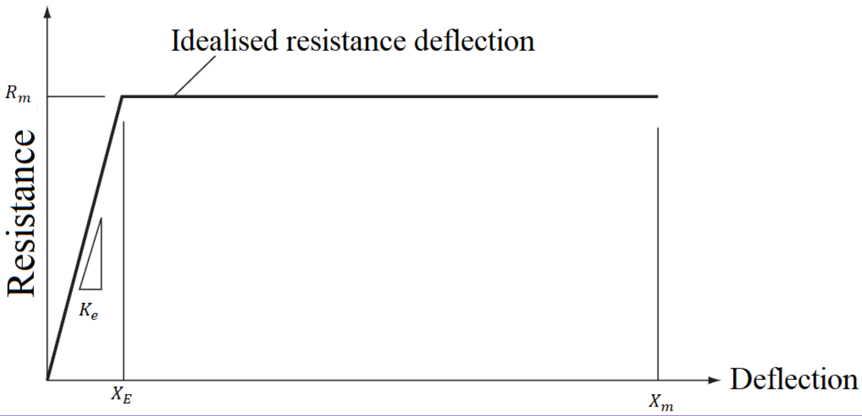

Fig. 9. Idealized resistance-deflection curve (modified from [1]).

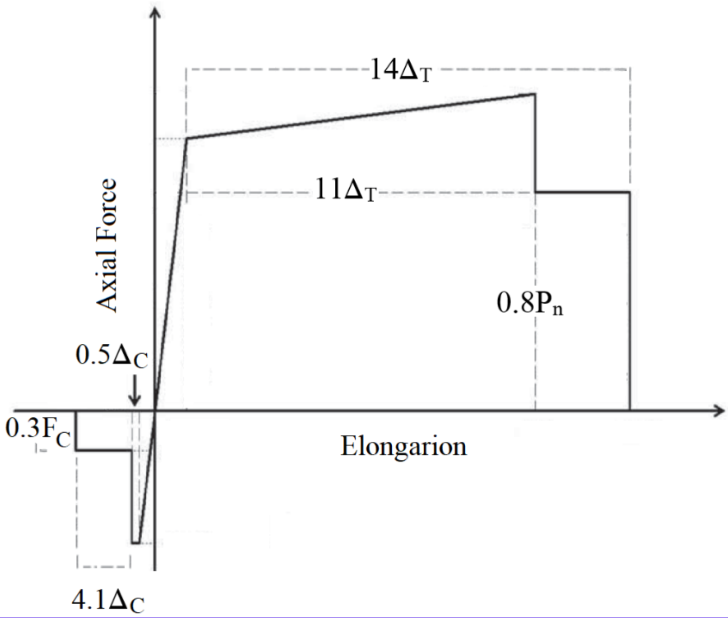

Fig. 10. Tension-compression hinge properties (modified from [22] and [7]).

Table 6. Tension-Compression Hinge Parameters [22][7].

Loading	a	b	c
Tension	$11\Delta_T$	$14\Delta_T$	$0.8P_n$
Compression	$0.5\Delta_C$	$4.1\Delta_C$	$0.3F_{cr}$

5 Strength Reduction Factors and Load Combination

Because of the nature of the blast load and to achieve economical design, plastic deformations are allowed in the design of structures subjected to blast loads. Also, it is permissible to use the nominal strength without a strength reduction factor (i.e., $\phi = 1$) for all modes of failure [6]. Blast loads are not combined with loads that are not expected to be present when the blast happens. That is, wind, earthquake, part or all the live loads are not combined with blast loads; the basic load combination for all construction materials is as follows [5]:

$$1.0\,DL + 1.0\,LL + 1.0\,BL \tag{4}$$

where DL is the dead load, LL is the live load, and BL is the blast load. In the absence of other governing criteria, [7] allow the following load combination:

$$1.0\,DL + 0.25\,LL + 1.0\,BL \tag{5}$$

6 Blast Load and Structure Interaction (Structural Response)

For an isolated building, as the blast wave propagates, its front engulfs the structure. Therefore, all faces of the structure are subjected to positive and negative pressure at different times and for different durations. The structure resists the kinetic energy of moving components by converting it to strain energy in the resisting elements [14]. Due to high strain rates, nonlinear inelastic material behavior, time-dependent deformation, and uncertainties of blast load and location, the structural dynamic response is complex [26]. Depending on the predicted structural failure mechanism, designers can select the best analytical method to compute the structural response. Pressure-impulse (P-I) charts, single element response analysis, and detailed finite element analyses are the most common approaches to computing structural response [6]. Designers must select an appropriate analytical approach based on expected failure mechanisms.

6.1- Pressure-Impulse Charts

Pressure-impulse or iso-damage curves are based on analytical or experimental data where the peak pressure and impulse represent the explosive loading on the P-I curve to check the performance condition of a target member. This simple method can be used to design secondary elements but not primary elements, and it is limited to flexural modes in response to blast loads [6]. Fig. 11 shows a typical P-I diagram for an elastic SDOF component, where F is the impulse force, K is the member stiffness, M is the total mass of the member, I is the impulse (I=peak blast load×duration of idealized triangular blast load/2), u is the displacement, and u_{max} is the maximum dynamic response.
Once the maximum response is specified (damage criterion), Fig. 9 can be used to find the impulse and the load that causes failure or to check whether the section to be designed is damaged. That is, when the combinations of impulse and pressure fall to the right and above, the curve will result in failure; when the combinations fall to the left and below, the curve will not induce failure. Note that axes $\frac{2F}{u_{max}K}$ and $\frac{I}{u_{max}\sqrt{MK}}$ represent pressure and impulse, respectively, and they have no physical units. Smith and Hetherington [12] discussed this approach with numerical examples.

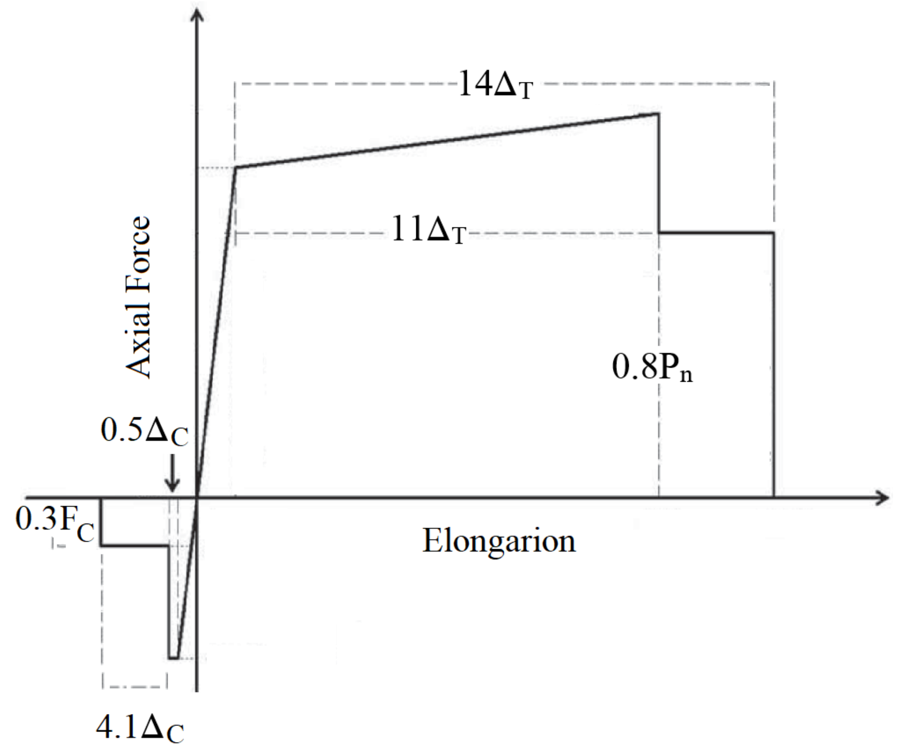

Fig. 11. Pressure-impulse diagram for the elastic SDOF component (modified from [12]).

6.2 The Single-Element Analysis Method

This method involves analyzing and designing individual members subjected to blast loading. This is either an SDOF or multi-degree-of-freedom (MDOF) system with elastic or inelastic dynamic analysis. The SDOF approach is the most common, and its accuracy depends on selecting a model that ade quately represents the failure mechanism. In this approach, the member's mass is concentrated at one point and is allowed to move along a single axis by assuming one response mode. The linear equation of motion for SDOF is:

$$M\ddot{u}(t) + C\dot{u} + Ku(t) = f(t) \tag{6}$$

where M is the total mass of the member, C is viscous damping, K is the member stiffness, u is displacement, $\dot{u}$ is velocity, and $\ddot{u}$ is the acceleration at time t. Equation (6) can be solved by numerical integration using structural analysis software programs such as ABAQUS [15], ANSYS [25], LS-DYNA [16], and SAP2000 [25-26]. This model can be simplified further by considering an elastic undamped SDOF system subjected to a triangular pulse load (just the equivalent positive phase). Thus, Eq. (6) becomes [1]:

$$M\ddot{u}(t) + Ku(t) = F(1 - \frac{t}{t_d}) \tag{7}$$

where F is peak force and t_d is positive phase duration. To solve Eqs. (6) and (7), the time increment should not be greater than 1/20 of the natural period of the member or 1/20 of the pulse duration t_d to provide numerical stability [7]. The reader is referred to UFC 3-340-02 [4] and the works of ASCE [5] and Gilsanz et al. [7] for more details.

In Eq. (6), the damping effects are commonly ignored because the blast load duration is short and energy dissipates through inelastic deformation [5]. However, it is allowable to include the damping effect when the response is nearly elastic [6].

The MDOF approach, described in the next section, is more accurate than the SDOF approach because all potential modes of failure can be represented, especially when nonlinear finite element analysis is carried out with geometric nonlinearity.

6.3 Multi-Degree-of-Freedom Finite Element System

The single-element modeling discussed above does not represent the actual boundary conditions, nor does it consider the interaction between elements and the phasing of their response or the dissipation of the energy of the whole structure [6]. On the other hand, MDOF modeling of structural systems does not ignore these important parameters. Moreover, the distribution of the mass and stiffness can be modeled throughout the structure instead of for only one member. In this approach, the linear or nonlinear time-history analysis methods can be used to determine the entire structural response. The complexity of the model depends on the type of element used in finite element analysis, where the spring element is the simplest and the solid element is the most complex.

Discrete System: In this type of structural modeling, a beam element can be used. Depending on the symmetry of the structure and the loading, and the model can be two- or three-dimensional. The relative flexibility and strength of the connected elements are considered. Moreover, this structural system analysis considers the phasing of the responses between structural elements [6]. Structural analysis outputs that include nodal and element displacements and plastic hinge(s) rotations (when material nonlinearity is considered) can be used directly to check the design criteria.

Implicit or Explicit Linear or Nonlinear Finite Element Analysis: This approach is necessary for complex structures and to obtain more accurate results. Linear or nonlinear plate/shell elements and solid elements can be used. Implicit, explicit, or mixed-hybrid modeling can be carried out [13]. The implicit method involves a numerical solver to invert the stiffness matrix to directly find the displacement vector. Thus, the implicit scheme is not a function of time. This method is unconditionally stable, but it is computationally expensive when the structure is large. Implicit methods are used in software such as ABAQUS and ANSYS. An explicit scheme is a function of time since it involves solving for velocity and acceleration as well as the inverse of the mass matrix (diagonal matrix), but the inverse of the stiffness matrix is not needed. This approach is conditionally stable. That is, small time steps should be used to obtain accurate results. The explicit method is a good choice for large models and blast load problems because the propagation of the blast load through the structure requires small time steps [16]. The explicit method is used in software such as LS-DYNA and ABAQUS.

For both approaches, the interaction between the primary structural system and the nonstructural components can be considered to avoid any possible local failure. ASCE [6] recommends not directly connecting vertical load-carrying elements to exterior envelope components unless they are designed to have greater strength than the exterior envelope components they are to be connected to. Also, one-way walls without backing elements can be designed to transfer loads directly into floor diaphragms.

6.4 Equivalent Static Method

In this method, the blast load is transferred to its equivalent static load, and then the structural static analysis is carried out. This method does not represent the actual response because dynamic parameters such as stain rate, mass, plastic deformation, and time-varying load are ignored. However, when the blast source is far from the structure, the blast loading can be represented as an “equivalent wind” [5].

Another way to transfer the blast load to its equivalent static load is by using the Equivalent Static Loads Method (ESLM) [28-31]. This method is based on the displacement field obtained using dynamic analysis of the structure. In other words, several comparable static load sets are created from the dynamic load. The linear static response optimization procedure then takes into account the equivalent static loads (ESLs) as numerous loading circumstances.

7 Criteria for Responses (Response Limits)

In static design philosophy (the working stress, ultimate load, and limit state methods), the level of stress in components and deflection are typically the criteria to define failure. In blast design (similar to seismic design), it is expected that some of the components will experience a substantial nonlinear response because designing them to remain elastic is usually uneconomical. However, when a structure is required to be reused following a blast, it must be designed to remain elastic [5]. That is, in designing blast-resistant structures, the maximum dynamic deflection and rotation are the criteria to prevent component failure. The performance of the entire structure is defined by life safety, functionality, and reusability [14]. Moreover, designers must check that the failure of key members will not cause any progressive collapse by providing sufficient redundancy (alternate load paths). The level of protection (LOP) (see Table 7) for the structure or component, the type of component, and the material to be used define the design criteria [5]. For example, the response limit of individual elements is less than the allowable response of individual frame elements because frames have higher redundancy. Also, for structural components (such as beams and columns), the response limits are less than nonstructural components (such as purlins).

There are several sources for response limits, including UFC 3-340-02 [4], Design of Blast-Resistant Buildings in Petrochemical Facilities [5], FEMA 356 [22], and the New York City Building Code [32]. Although all these sources define the criteria based on deformation, the limiting values are different, so the designer may need to review these limits. This review, however, is limited to a portion of what is provided in Design of Blast-Resistant Buildings in Petrochemical Facilities [5]. Before defining the response limit values, three important terminologies are defined:

1. *Ductility ratio (μ)*: This is the ratio between the total displacement, X_m, and the elastic displacement, X_E, as follows:

$$\mu = X_m / X_E \tag{8}$$

where displacement is the elongation of components subjected to axial load or the deflection of components subjected to bending, as shown in Fig. 12 [6]. Ductility is a measure of how much a component can carry beyond the elastic range before it drops the load.

2. *Rotation (θ)*: This is the tangent angle at the support caused by the maximum deflection. Figs. 10 and 11 show the rotation of a single element and a frame, respectively. Note that plastic hinges can happen not just at the mid-span of a member but also at other locations. This criterion indicates the degree of stability in a component.

3. *Side-sway deflection or lateral drift (δ)*: This is the movement of a vertical member relative to its bottom (Fig. 13). Side-sway limits allow framed structures to minimize the P-delta effects on columns and the chance of progressive collapse [5]. Side-sway deflection limit can be defined as follows:

$$\delta \leq response\ limit \tag{9}$$

where the response limit is story height H divided by some factor.

Similar to the modeling and analysis methods discussed in Section 2, there are two types of criteria: one for elements that are modeled and analyzed as SODF, and one for MDOF systems such as framed structures [4].

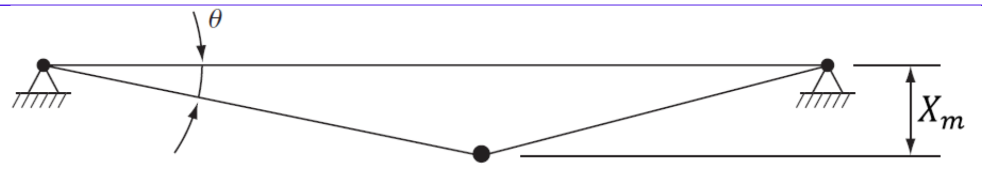

Fig. 12. Member (beam, slab, or panel) support rotations (modified from [4]).

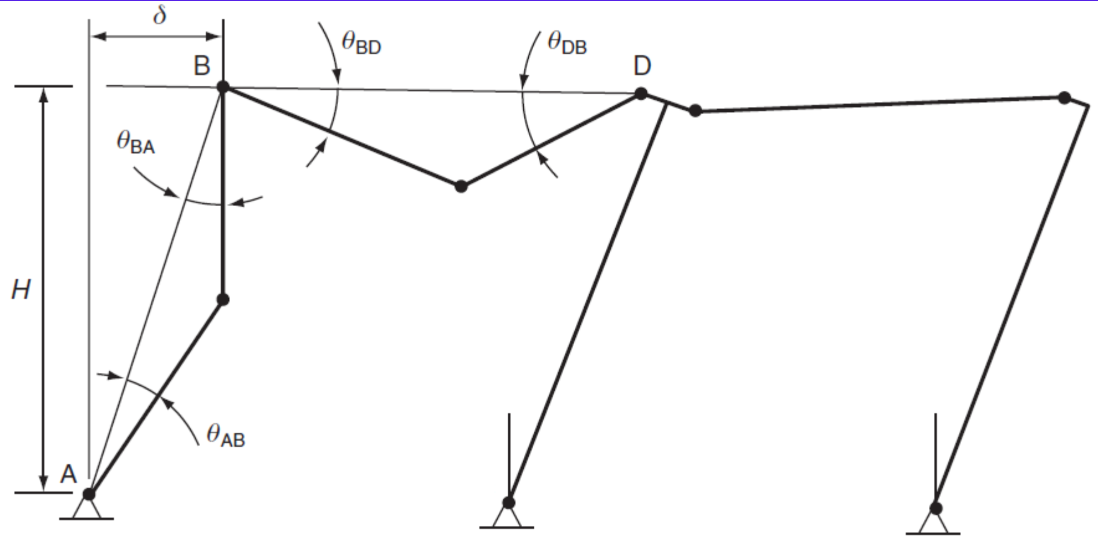

Fig. 13. Frame support rotations and side-sway deflection (modified from [4]).

Table 7. Damage and Response Level [5].

Damage level	Description	Response level	Description
Low	Localized component damage. The structure can be utilized, but it needs repairing. Total cost of repairs is moderate.	Low	Component has none to slight visible permanent damage.
Medium	Widespread component damage. Building should not be occupied until repaired. Total cost of repairs is significant.	Medium	Component has some permanent deflection. It is generally repairable, if necessary, although replacement may be more economical and aesthetic.
High	Component has some permanent deflection. It is generally repairable, if necessary, although replacement may be more economical and aesthetic.	High	Component has not failed, but it has significant permanent deflections causing it to be unrepairable.

7.1 Design Criteria for Individual Elements

Most of the design criteria are provided for individual components. Table 8 shows the response criteria for some steel components for different levels of response.

In Table 8, component response refers to the level of damage. Low response means there is no or only slight visible damage. Medium response refers to some permanent damage to the component that can be repaired. A component with high response has not failed, but it has experienced permanent damage that cannot be repaired.

Table 8. Response Limits for Different Components* [5].

Component	Component response					
	Low		Medium		High	
	μ	θ	μ	θ	μ	θ
Steel Primary Frame Members (with significant compression)**	1.5	1	2	1.5	3	2
Steel Primary Frame Members (without significant compression)	1.5	1	3	2	6	4
R/C Beams, Slabs, & Wall Panels (no shear reinforcement)	-	1	-	2	-	5

* Response limits are for components responding primarily in flexure
** Significant compression is when the axial compressive load is more than 20% of the dynamic axial capacity of the member.

7.2 Design Criteria for a Structural System

The ductility ratio criteria concept for individual members is intractable in the design of frame structures because of the wide range and time-varying nature of the end conditions of components [4]. That is, in addition to the support rotation criteria, the side-sway limits should be checked for framed structures. Table 9 presents side-sway deflection limits for different levels of response for steel-frame structures.

Table 9. Side-Sway Limits for Steel Frame Structures (ASCE, 2010)

Response	Low Response	Medium Response	High Response
δ	$H/50$	$H/35$	$H/25$

8 Progressive Collapse

ASCE [5] defines progressive collapse as the “chain-reaction failure of a building’s structural system or elements as a result of, and to an extent disproportionate to, initial localized damage, such as that caused by an explosion.”

As a result of a blast loading, structural components may fail, and their loads may be distributed to neighboring members. If the surrounding members cannot tolerate this extra load, failure can propagate vertically or horizontally. The entire structural system should be evaluated when a blast is expected to cause local failure or plastic hinges of structural components. In blast-resistant design, local damage is expected, but the whole structural system should be stable.

To prevent progressive collapse, the primary members or key elements must be strengthened, and/or the global structural redundancy should be increased so that only local failures are permitted.

The DoD [33] requires that buildings of three or more stories must comply with progressive collapse standards. The reader is referred to work by the DoD [33] and Marchand and Alfawakhiri [34] for further details.

9 Concluding Remarks

In this review paper, an overview of topics related to the design of blast-resistant structures is provided. Three methods to predict blast loading are discussed, and the modeling of structural response and material behavior under blast loading is reviewed. Design philosophies and criteria are explained, and basic concepts related to the blast-resistant design field are summarized. References on each topic are provided for further details.

In the area of blast design of structures, there are few practical design examples available in the literature. Therefore, as future research, formulations need to be developed for an efficient design of structures subjected to blast loading.

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