भारतीय मानक

IS 4991 (Part 1) : 2024

Indian Standard

संरचनाओं के विस्फोट प्रतिरोधी डिज़ाइन के लिए मानदंड

भाग 1 अधितल विस्फोट

(पहला पुनरीक्षण)

Criteria for Blast Resistant Design of Structures

Part 1 Above-ground Explosions

(First Revision)

ICS 91.120.25

भारतीय मानक ब्यूरो

BUREAU OF INDIAN STANDARDS

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FOREWORD

This Indian Standard (Part 1) (First Revision) was adopted by the Bureau of Indian Standards, after the draft finalized by the Earthquake Engineering Sectional Committee had been approved by the Civil Engineering Division Council.

This standard was first published in 1968 under the title 'Criteria for blast resistant design of structures for explosions above ground'. In 2023, the Committee decided to present the provisions of blast resistant design separately for above ground and underground explosions under IS 4991. Hence, IS 4991 was split in two parts:

Part 2 Underground explosions

Until IS 4991 (Part 2) is published, IS 6922 'Criteria for safety and design of structures subject to underground blasts' will be applicable.

This revision has been brought out as a major exercise to reflect the improvement in knowledge and understanding over the last five decades. This exercise has been motivated and expedited by the growing threat of malevolent and accidental blast loading of structures in the country. Structures due to explosions are subjected to rapidly moving shock wave which may exert very high pressures, though, the peak intensity lasts for a very small duration only. The blast wave loads the exposed surface of the structure and then the load is transmitted to the other elements. Thus, the response assessment of each individual structural element is of prime importance in blast-resistant design of structures.

This standard provides prescriptive requirements for estimation of blast loads and simplified methods for structural analysis that includes:

a) General requirements for blast-protection and categorization of structures;

b) Threat assessment requirements for determination of design explosive charge mass and scaled distance;

c) Blast load estimation methods due to above-ground explosion of high-explosive under three different detonation scenarios:

   1) Surface burst;

   2) Free-air burst;

   3) Air burst;

d) Evaluation of blast loads effects on structures of regular geometry and shape;

e) Prescriptive simplified single degree of freedom (SDOF) analysis procedure for peak response evaluation of structural elements based on flexural mode; and

f) Guidelines for the application of multi-degree of freedom (MDOF) and detailed finite element analysis.

This standard provides a minimum design blast threat that can be adopted in absence of threat assessment. However, the blast threat specified by the standard can be superseded by the values informed through actual threat assessment. The threat assessment shall be initiated by the owner and conducted by qualified personnel to determine the explosive charge mass and standoff distances. The necessity of blast-resistant design for a structure shall be specified by the owner of the facility based on threat assessment and risk analysis. This standard does not provide the methodology for risk analysis. Reference to a particular type of structure in this standard does not automatically requires it to be designed for explosion threats.

The application of blast load estimation charts and the structural analysis methods provided in the standard depends on the scaled distance of the explosive charge mass. In general, the provisions of this standard are not applicable for scaled distances smaller than 0.4 m/kg$^{1/3}$; and secondary post-blast consequences such as fragmentation, fire, and progressive or disproportionate collapse are not included in this standard.

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Indian Standard

CRITERIA FOR BLAST RESISTANT DESIGN OF STRUCTURES

PART 1 ABOVE-GROUND EXPLOSIONS

(First Revision)

1 SCOPE

1.1 This standard (Part 1) prescribes the minimum criteria for analysis and design of new structures for blast effects of above-ground explosions of high-explosives under three different detonation scenarios: surface burst, free-air burst, and air burst. The primary objective of this standard is to minimize human casualties and safeguard assets due to above ground explosions. The performance expectation of a structure designed as per the provisions of this standard would depend on the type of protection category selected for that structure and the blast loading determined through threat assessment.

1.2 This standard shall be used in conjunction with other applicable standards for analysis and design of structures. This standard does not supersede or replace other design standards. This standard shall be applicable if design against malevolent or accidental blast loading is determined to be a credible threat to a structure.

1.3 The owner of a structure is the competent authority who can determine the necessity of blast-resistant design. In the scenario of accidental and malevolent blast event which may pose public health and safety issue, competent authorities other than the owner of building that govern or regulate the construction, operation, and maintenance of the structural facility, can specify the requirement of blast-resistant design.

1.4 The provisions of this standard do not address the following:

a) Estimation of structural loads due to nuclear detonations, pressure vessel explosions, dust, and vapour cloud explosions;

b) Effects of cased explosives and shaped charges;

c) Threats emanating from fragmentation, induced fire or thermal effects, biological or chemical radiations, electromagnetic pulse (EMP) and deflagration events; and

d) Detonations in contact with structural elements.

2 REFERENCES

The standard given below contain provisions which through reference in this text, constitute provision of this standard. At the time of publication, the edition indicated were valid. All standards are subject to revision, and parties to agreements based on this standard are encouraged to investigate the possibility of applying the most recent edition of these standard:

IS No.	*Title*
IS 456 : 2000	Plain and reinforced concrete — Code of practice (*fourth revision*)
IS 800 : 2007	General construction in steel — Code of practice (*third revision*)
IS 875	Code of practice for design loads (other than earthquake) for buildings and structures:
(Part 1) : 1987	Dead loads — Unit weights of building materials and stored materials (*second revision*)
(Part 2) : 1987	Imposed loads (*second revision*)
(Part 3) : 2015	Wind loads (*third revision*)

3 TERMINOLOGY

For the purpose of this standard, the following definitions shall apply.

3.1 Arrival Time — Time taken by the shock after the explosion to reach the location of interest.

3.2 Charge Mass — Mass of the explosive material responsible for generating blast effects.

3.3 Clearance Time — Time in which the reflected pressure decays down to the sum of the side-on overpressure and the dynamic pressure.

3.4 Close-in Detonation — Detonation at a scaled distance (Z) below which the resulting blast pressure distribution is non-uniform over the reflecting surface of the element being considered. It corresponds to a value of *Z* less than 1.2 $m/kg^{1/3}$.

3.5 Decay Parameter — The coefficient of the negative power of exponent *e* governing the fall of pressure with time in the pressure-time curves.

3.6 Detonation — Fast and sudden release of huge amount of energy through a stable chemical reaction in which reaction front propagates into unreacted substance at supersonic speed resulting in formation of shock waves.

3.7 Drag Force — Force on a structure or structural element due to the dynamic pressure of the blast wave. On any structural element, the drag force equals dynamic pressure multiplied by the drag coefficient and the area of the element.

3.8 Ductility Ratio — Ratio of the maximum deflection to the deflection corresponding to the elastic limit.

3.9 Dynamic Increase Factor (DIF) — A factor applied to material strength to account for high strain rate loading effects.

3.10 Dynamic Pressure — The pressure due to air mass movement behind the shock front.

3.11 Exponential Blast Pressure History — The variation of blast pressure with time at a point represented through an exponential decaying function, which considers positive and negative phase of blast overpressures.

3.12 Far-field Detonation — Detonation at a scaled distance (*Z*) above which the resulting blast pressure distribution is uniform over the reflecting surface of element being considered. It corresponds to a value of *Z* greater than 1.2 $m/kg^{1/3}$.

3.13 High-Explosives — Chemical explosive that undergoes detonation to produce shock waves with high pressure and short duration.

3.14 Impulse — The integral of the pressure-time curve per unit projected area due to explosion (*see* **3.27**).

3.15 Mach Number — The ratio of the speed of the shock front propagation to the speed of sound in standard atmosphere at sea level.

3.16 Negative Phase — The duration of reflected overpressure when it is below the ambient atmospheric pressure.

3.17 Overpressure — The rise in pressure above atmospheric pressure due to the shock wave from an air blast.

3.18 Positive Phase —The duration of overpressure when it is above the ambient atmospheric pressure.

3.19 Protection Category — Category assigned to different structures that correlates its importance to the performance goals through qualitative estimate of damage.

3.20 Reflected Overpressure — The overpressure resulting due to reflection of a shock wave front striking any surface. If the shock front is parallel to the surface, the reflection is normal.

3.21 Regular Shaped Structure — A structure that does not have any spatial geometric irregularity.

3.22 Scaled Distance — The distance between the centre of explosion to the point of interest divided by the cube root of the charge mass expressed in equivalent TNT. The unit for scaled distance is $m/kg^{1/3}$.

3.23 Shape Function — The displaced shape of the component as a function of its length normalized to the maximum deflection.

3.24 Shock Wave Front — The discontinuity between the blast wave and the surrounding atmosphere. It propagates away from the point of explosion in all directions at a speed greater than the speed of sound in the undisturbed atmosphere.

3.25 Side-on (Incident) Overpressure — Overpressure if it is not reflected by any surface.

3.26 Simplified Overpressure History — A triangular representation of the overpressure history obtained by equating the impulse and the peak overpressure of the exponential representation of the overpressure history.

3.27 Specific Impulse — The area under the pressure-time curve due to explosion. This is often referred to as impulse (*see* **3.14**).

3.28 Standoff Distance (R) — The shortest distance between the centre of detonation and the point of interest at which the effects of explosion need to be considered.

3.29 Support Rotation — The angle formed by a flexural member in its deflected shape with respect to the chord adjoining the two support ends (*see* Fig. 22).

3.30 Transit Time — Time required for the shock front to travel across the structure or its element under consideration.

4 SYMBOLS

For the purpose of this standard, the following notations shall apply.

b	Decay parameter/coefficient
C_d	Drag coefficient
C_r	Velocity of sound in air
$C_{r\alpha}$	Reflection coefficient
E	Modulus of elasticity of material
H	Height of the structure
I	Moment of inertia of member
i_e	Equivalent specific impulse of a uniformly distributed load
i_{ro}	Peak reflected specific impulse
j	Number of concentrated load points
K_L	Load factor
K_{LM}	Load mass factor
K_M	Mass factor
k_E	Effective stiffness of equivalent single spring-mass system
L	Length of structure in the direction of motion of blast wave (*see* Fig. 16)
M	Charge mass/weight
M_p	Plastic moment of section
M_{pm}	M_p at mid-span
M_{ps}	M_p at support
M_{pfa}	Total positive ultimate moment capacity along mid-span section parallel to short edge
M_{pfb}	Total positive ultimate moment capacity along mid-span section parallel to long edge
m	Distributed mass intensity per unit length
n	Number of concentrated masses
p_a	Ambient atmospheric pressure
P_r	Concentrated dynamic force at point r
p_{ro}	Peak reflected overpressure
p_s	Side-on overpressure
p_{so}	Peak side-on overpressure
P_t	Total dynamic load (at any instant of time)
$p(x)$	Intensity of distributed dynamic load per unit length
p_{o1}	Peak reflected overpressure on a surface including the effects of clearing
p_{o2}	Intercept of the reflected overpressure history on the y-axis corresponding to the clearing phase
q	Dynamic pressure
q_o	Peak dynamic pressure
R	Standoff distance
R_m	Peak resistance of a structural member against a given load distribution
S	$\frac{1}{2}$ W or H whichever is less
T	Elastic time-period of structural member
t_c	Clearance time
t_d	Duration of the equivalent triangular pulse
t_m	Time to the peak displacement of an SDOF system
t_o	Time for positive phase of side-on overpressure
t_t	Transit time
t_{d1}	Time at which clearing starts on a finite reflecting surface (*see* Fig. 24B)
t_{d2}	Total time duration of blast loading on a finite reflecting surface (*see* Fig. 24B)
U	Shock front velocity
u	Peak particle velocity
V_A	Total dynamic reaction along a short edge
V_B	Total dynamic reaction along a long edge

y_E	Displacement corresponding to the elastic limit of an equivalent bilinear representation of idealised resistance-deflection curve
y_{el}	Displacement corresponding to the first onset of yielding (elastic limit) of the idealized resistance-deflection curve of a structural member
y_m	Maximum deflection/deformation permitted in the design of the structure
y_o	Peak static displacement of an SDOF system without inertial effects
W	Span or width of structure across the direction of shock wave propagation
Z	Scaled distance
μ	Ductility ratio $= \frac{y_m}{y_E}$
φ_r	Deflection at point *rr* of an assumed deflected shape for concentrated loads
$\varphi_{(x)}$	Deflection at point x of an assumed deflected shape for distributed loads
$\varphi_{(x,y)}$	Deflected shape function of blast-loaded component

5 GENERAL PRINCIPLES

5.1 Attributes of Blast Resistant Design

The design process of a building against blast should start with planning of building layout and arrangements. The distance between the explosive and the building shall be maximized through a combination of site-access controls, landforms, anti-ram structures, street and site landscaping, and perimeter walls. The building should preferably have short-face oriented towards blast source. Also, it shall be located uphill and placed upstream of wind direction and away from congestion. The structures and their contents that need protection against an explosion event should be decided and all possible scenarios need to be considered which can cause damage or occupant injuries. The overall design of a structure shall aim to incorporate following attributes in the overall planning and design of a facility against explosion threats:

a) *Security* — Discourage potential terrorist attacks on a structure through visible security personnel deployment and defences to reduce chance of such attacks;

b) *Deception* — Disguise the critical area or content of structure so that the focus of the attack is misdirected. Such attacks, even if successfully completed, may not deliver intended impact and realize its goals;

c) *Shielding* — Shield the target structure through physical barriers, including but not limited to blast walls, bollards, anti-ram elements, vehicle and pedestrian entry controls, and landforms. The objective of shielding is to increase the standoff distance between the charge and the target structure and components; and

d) *Design* — The attack must be blunted through appropriate design and detailing of the structure to absorb the energy of the attack and safeguard valuable assets.

5.2 Blast Source

5.2.1 *Charge Shape*

The provisions of this standard shall be applicable for:

a) explosives of spherical shape; and

b) non-spherical charges if the detonation effects are considered at scaled distances greater than 3 m/kg$^{1/3}$.

In case of non-spherical charges at scaled distances smaller than 3 m/kg$^{1/3}$, the provisions of **9.3.3** shall be applicable with explicit modelling of the detonation process.

5.2.2 *Detonation Scenarios*

The standard addresses three scenarios of unconfined explosions based on the location of the detonation with respect to the point of interest:

a) Spherical (free-air) burst (*see* Fig. 1);

b) Hemispherical (surface) burst (*see* Fig. 2); and

c) Air-burst (*see* Fig. 3).

5.2.3 *Standoff Distance*

The standoff distance (R) shall be estimated as the distance between the centre of detonation and the point of interest. If the location of interest is not directly in the line of the sight from the centre of detonation, then Fig. 4 shall be referred to estimate the effective standoff distance.

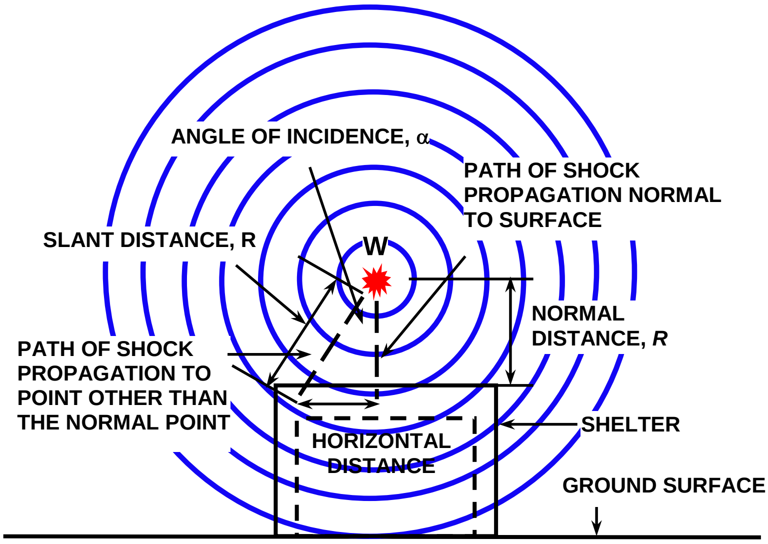


FIG. 1 SPHERICAL (FREE-AIR) BURST

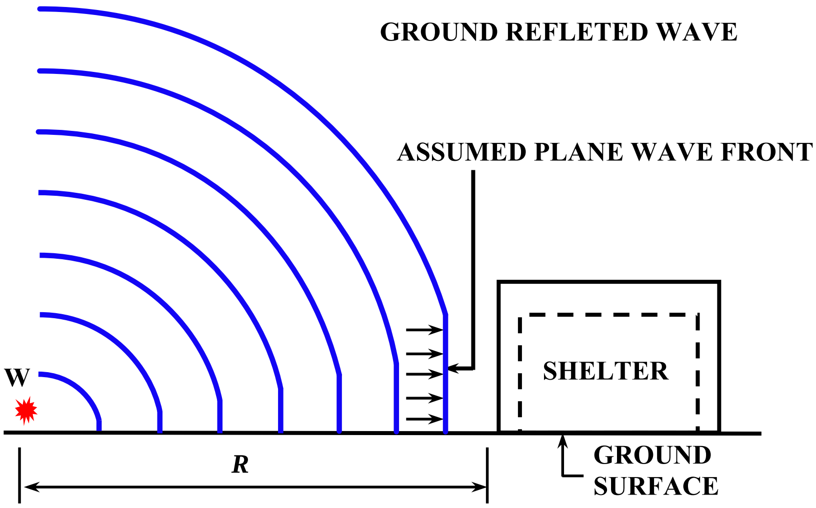


FIG. 2 HEMISPHERICAL (SURFACE) BURST

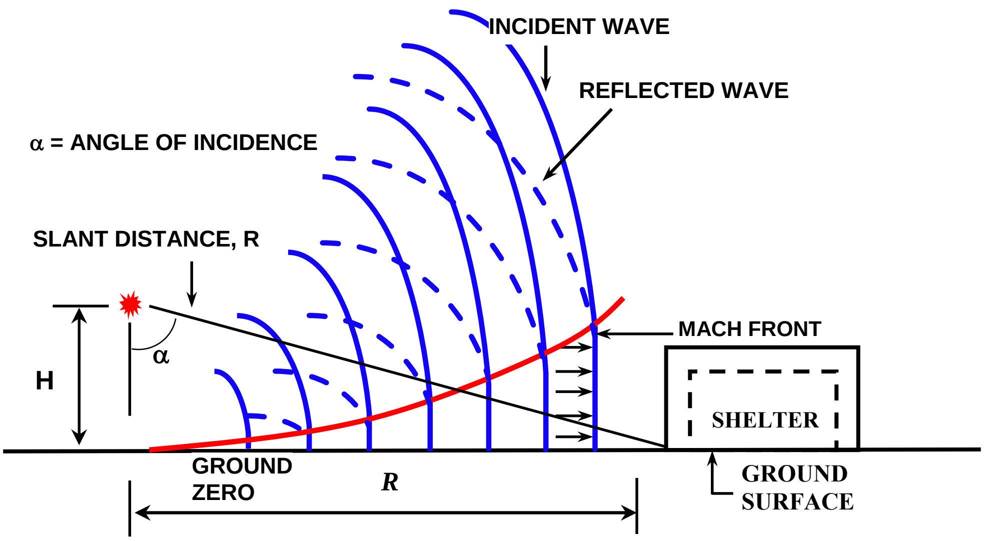
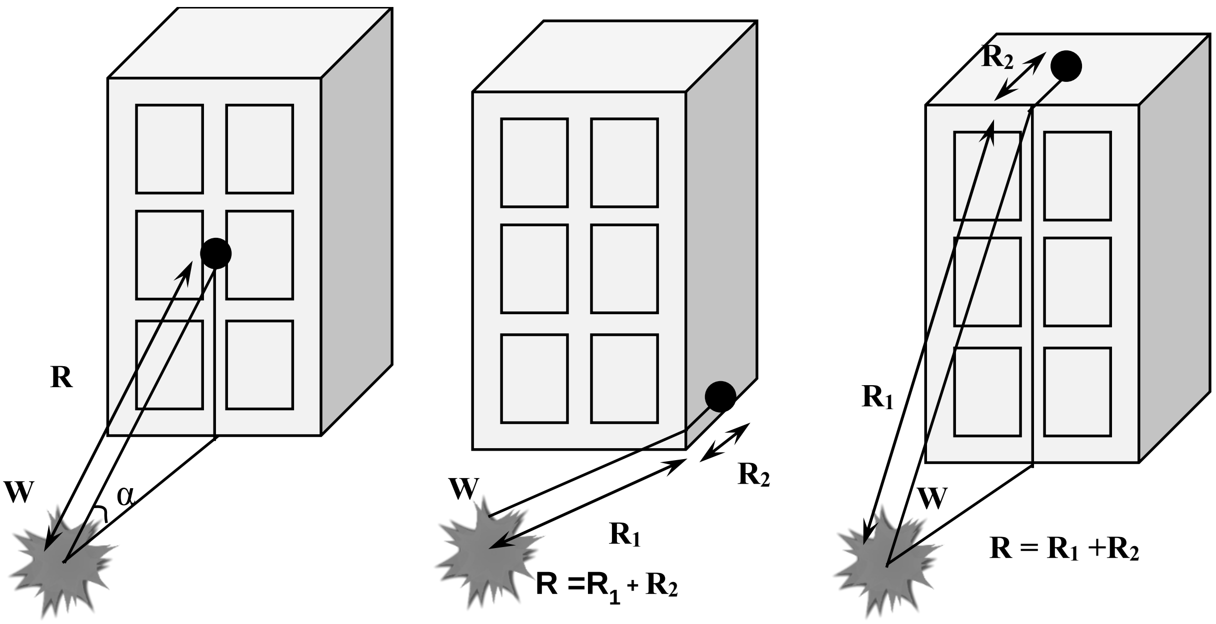


FIG. 3 AIR-BURST



4A FRONT FACE 4B SIDE FACE 4C TOP FACE

FIG. 4 CALCULATION OF STANDOFF DISTANCES AT DIFFERENCE FACES OF A BUILDING STRUCTURE

5.2.4 *Equivalent TNT Charge*

Trinitrotoluene (TNT) shall be used to benchmark all types of explosives. To quantify blast wave parameters (pressure, impulse) from explosives other than TNT, the actual mass of the explosive (M_{EXP}) shall be converted into a TNT-equivalent mass. The TNT equivalent weight (M_{TNT}) of any explosive shall be estimated as:

$$M_{TNT} = \left(\frac{\Delta E_{EXP}}{\Delta E_{TNT}}\right) M_{EXP}$$

where

ΔE_{EXP} and ΔE_{TNT} = the specific energies of the explosive and the TNT, respectively.

The factors presented in Table 1 shall be utilized to obtain equivalent mass of TNT corresponding to the listed explosives.

5.2.5 *Cube Root Scaling*

Cube root scaling shall be used to relate blast parameters from different sources and standoff distances. The cube-root scaling shall be valid if the conditions specified in **5.2.1** are satisfied.

The blast load parameters can be obtained in scaled form as function of scaled distance, *Z*, as:

$$Z = \frac{R}{M^{1/3}}$$

where

R = standoff distance of the point of interest from the centre of charge mass; and

M = mass in equivalent TNT.

The unit for scaled distance shall be m/kg$^{1/3}$, for standoff distance in m and charge mass in kg.

5.3 Performance Expectations

5.3.1 *Categorization of Structures*

Structures shall be categorized based on their importance as per the specification of Table 2.

The categorisation of structures as per Table 2, does not automatically require them to be designed for explosion threats. Only, if the threat assessment carried as per the provisions of **6.1** identifies blast loading due to explosion as a potential threat, the categorisation in Table 2 shall be used to define the importance of a structure.

5.3.2 *Structural Performance*

A protection category shall be decided by the building owner based on the importance of the building and required performance. The performance goals for each protection category are listed in Table 3.

Structures of high, medium, and low importance shall be provided with protection categories 1, 2 and 3 respectively, as at least a minimum as specified in Table 4.

The facility owner may specify a protection category higher than the minimum protection category specified in Table 4 based on the realistic assessment of threat.

The protection categories specified shall comply with the deformation limits of structural elements provided in **9.1.4** when using simplified single degree of freedom (SDOF) and multi degree of freedom (MDOF) methods of analysis as per requirements of **9.3.1** and **9.3.2** respectively. Accordingly, all elements shall be designed and detailed to provide deformation limits specified for different protection categories.

If advanced methods of analysis are used as per **9.3.3**, the response and damage in structure and elements for the performance goals of each protection category mentioned in can be independently established. In such case, deformation limits specified in **9.3.1** and **9.3.2** need not to be used.

Table 1 Equivalent TNT Mass of Different Explosives

(*Clause* 5.2.4)

Sl No.	**Explosive**	**Density** Mg/m³	**Equivalent Mass for Pressure**	**Equivalent Mass for Impulse**
(1)	(2)	(3)	(4)	(5)
i)	Ammonia dynamite (50 percent strength)	NA[*]	0.90	0.90[ǂ]
ii)	Ammonia dynamite (20 percent strength)	NA[*]	0.70	0.70[ǂ]
iii)	ANFO (94/6 ammonium nitrate/fuel oil)	NA[*]	0.87	0.87[ǂ]
iv)	Composition C-4	1.59	1.20	1.19
		1.59	1.37	1.19
v)	Cycloid (75/25 RDX/TNT)	1.71	1.11	1.26
	(70/30)	1.73	1.14	1.09
	(60/40)	1.74	1.04	1.16
vi)	Gelatine dynamite (50 percent strength)	NA[*]	0.80	0.80[ǂ]
vii)	Gelatine dynamite (20 percent strength)	NA[*]	0.70	0.70[ǂ]
viii)	HMX	NA[*]	1.25	1.25[ǂ]
ix)	Nitrocellulose	1.65 to 1.70	0.50	0.50[ǂ]
x)	Nitroglycerine dynamic (50 percent strength)	NA[*]	0.90	0.90[ǂ]
xi)	Nitroglycerine (NQ)	1.72	1.00	1.00[ǂ]
xii)	PETN	1.77	1.27	1.27[ǂ]
xiii)	Picrotol (52/48 Ex D/TNT)	1.63	0.90	0.93
xiv)	RDX	NA[*]	1.10	1.10[ǂ]
xv)	RDX/wax (98/2)	1.92	1.16	1.16[ǂ]
xvi)	RDX/AL/wax (74/21/5)	NA[*]	1.30	1.30[ǂ]
xvii)	Tetryl	1.73	1.07	1.07[ǂ]
xviii)	Tetrytol (75/25 tetryl/TNT)	1.59	1.06	1.06[ǂ]
xix)	TNT	1.63	1.00	1.00

[*] NA — Data not available.
[ǂ] Value is estimated.

Table 2 Categories of Structures

(*Clause* 5.3.1)

Sl No. (1)	Importance (2)	Structure Type (3)
i)	Low	Residential buildings with 50 or less occupants
ii)	Medium	a) Residential buildings with more than 50 occupants b) Buildings with large congregation, such as schools, public offices, and cinemas c) Industrial buildings with continuous human occupancy
iii)	High	a) Lifeline structures (water, power, communication, transportation, hospital) b) Structures housing essential services that are required for disaster management (for example, emergency relief stores, data centres, governance continuity buildings) c) Financial and commercial establishments of strategic importance

Table 3 Protection Category and Associated Performance Goals

(*Clause* 5.3.2)

Sl No. (1)	Protection Category (2)	Performance Goal (3)
i)	1	Minimal damage, resumption after brief halt of operations
ii)	2	Limited damage, operational after repairs and retrofit
iii)	3	Extensive damage, safe replacement, or demolition

Table 4 Protection Category and Associated Category of Structures

(*Clause* 5.3.2)

Sl No. (1)	Building Category (2)	Minimum Protection Category (3)
i)	Low	3
ii)	Medium	2
iii)	High	1

6 THREAT ASSESSMENT

6.1 Threat Assessment Parameters

A threat assessment shall be carried out to determine all potential sources of detonation.

The threat assessment shall identify two parameters for the purpose of calculation of blast load parameters, namely:

a) charge mass; and
b) standoff distance.

If threat assessment identifies multiple sources of detonation, the effect of blast on the structure shall be assessed considering individual, as well as combinations of all identified detonation threats.

6.2 Design Blast (Reference Explosion)

6.2.1 *Design TNT Equivalence*

If the threat scenario results in identification of one of the detonation sources listed in Table 5, then the associated charge mass may be used for determination of equivalent TNT capacity.

6.2.2 *Standoff Distance*

The standoff distance (R) is site specific and shall be estimated as per the actual geometry and access control of the target structure.

7 BLAST LOAD

7.1 Blast Wave Propagation

The blast loads for design of structures and components shall be determined from blast pressure history.

The variation of pressure at a fixed location away from the centre of detonation shall be obtained using following assumptions:

a) The shock wave due to explosion moves away from the centre of detonation at a speed, U;

b) When the shock wave reaches a point at the arrival time t_a, the pressure rises instantaneously from the ambient pressure (p_a) to a peak pressure. This peak pressure, p_{so}, is known as side-on overpressure or peak incident overpressure;

c) The time needed for the pressure to reach its peak value is very small and for design purposes it is assumed to be zero;

d) The overpressure decreases exponentially from its peak value to the ambient pressure over the positive phase duration (t_o). After the positive phase of the pressure-time profile, the pressure becomes smaller than the ambient value (referred to as negative pressure), and finally returns to the ambient pressure, p_a;

e) The movement of air caused by the shock wave induces dynamic pressure, in addition to blast pressure. Air dynamic pressure q_o, and peak particle velocity u shall be obtained using peak incident pressure, p_{so} (*see* Fig. 9 and Fig. 10); and

f) Shock wave upon reflection from a rigid surface would apply reflected overpressure, p_r, to the reflecting surface. The variation of reflected overpressure with time for normal reflection follows the same trend as the incident overpressure but characterized by significantly higher peak overpressure.

7.2 Decay of Blast Overpressure with Time

The blast overpressure is assumed to vary with time, t according to the following exponential relationship (*see* Fig. 5):

$$p_s(t) = p_o\left(1 - \frac{t}{t_o}\right) e^{-b(t/t_o)}$$

where

p_o = peak blast overpressure, which is interpreted as the peak incident and reflected overpressures for the incident and the reflected blast wave, respectively; and

b = decay coefficient; and t_o is the positive phase duration. Special literature is to be referred for the calculation of the blast wave decay parameter, b.

7.3 Blast Overpressure History Parameters

The maximum values of the positive side-on overpressure p_{so}, incident impulse i_s, reflected over pressure p_r, reflected impulse i_r, positive phase duration t_o, shock wave front velocity U, and wave length $L_{w,}$ for varying scaled distances, are to be obtained for a spherical TNT explosion in free air and a hemispherical TNT explosion on the surface using Fig. 7 and Fig. 8, respectively. Further, air dynamic pressure q_o, and peak particle velocity u, are to be obtained using Fig. 9 and Fig. 10, respectively.

7.4 Oblique Reflection of Blast Wave

The clauses of this section are applicable to determine the peak reflected overpressure for scenarios where the angle between the wave front and the interacting surface is α (≤ 90°) as shown in Fig. 6. In such case, the peak reflected overpressure (p_{ro}) is to be determined as multiplication of peak incident overpressure (p_{so}) with a reflection coefficient ($C_{r\alpha}$). The reflection coefficient at any angle is to be determined from Fig. 11 for given incident overpressure.

Table 5 Detonation Source and the Associated Charge Mass

(*Clause* 6.2.1)

Sl No.	Threat Description		Explosive Capacity (TNT Equivalent)
(1)	(2)		(3)
i)		Pipe bomb	2.5 kg
ii)		Suitcase	25 kg
iii)		Hatchback	250 kg
iv)		Sedan/SUV	500 kg
v)		Passenger van/small cargo truck	2 000 kg
vi)		Delivery truck	5 000 kg
vii)		Water tanker/diesel truck/gas truck	15 000 kg

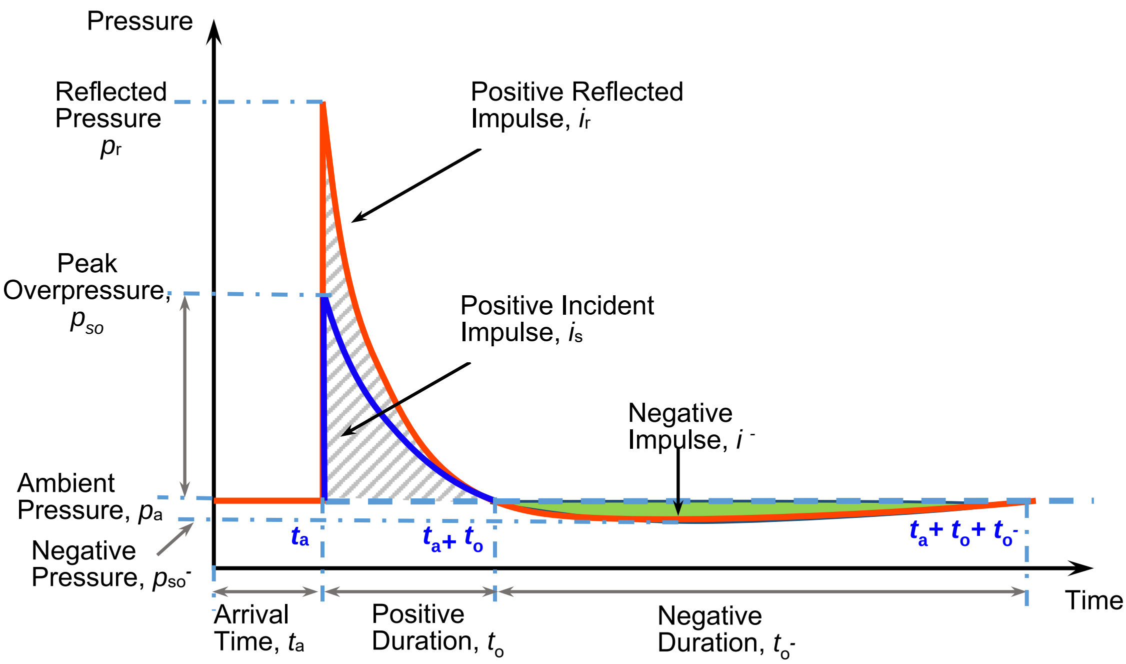


FIG. 5 INCIDENT AND REFLECTED BLAST PRESSURE HISTORY

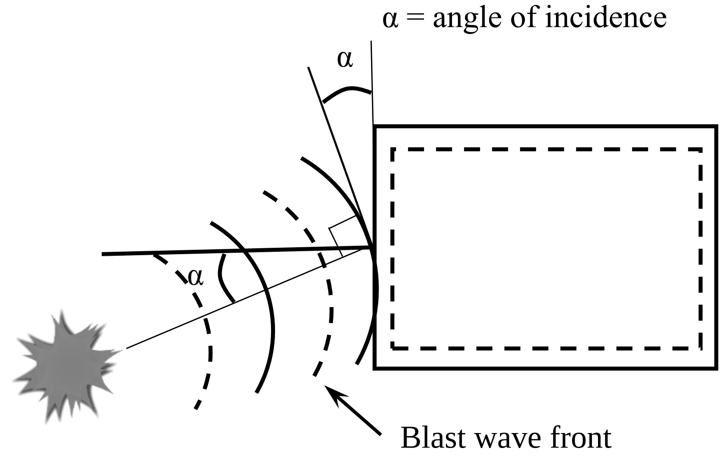


FIG. 6 OBLIQUE REFLECTION OF A BLAST WAVE

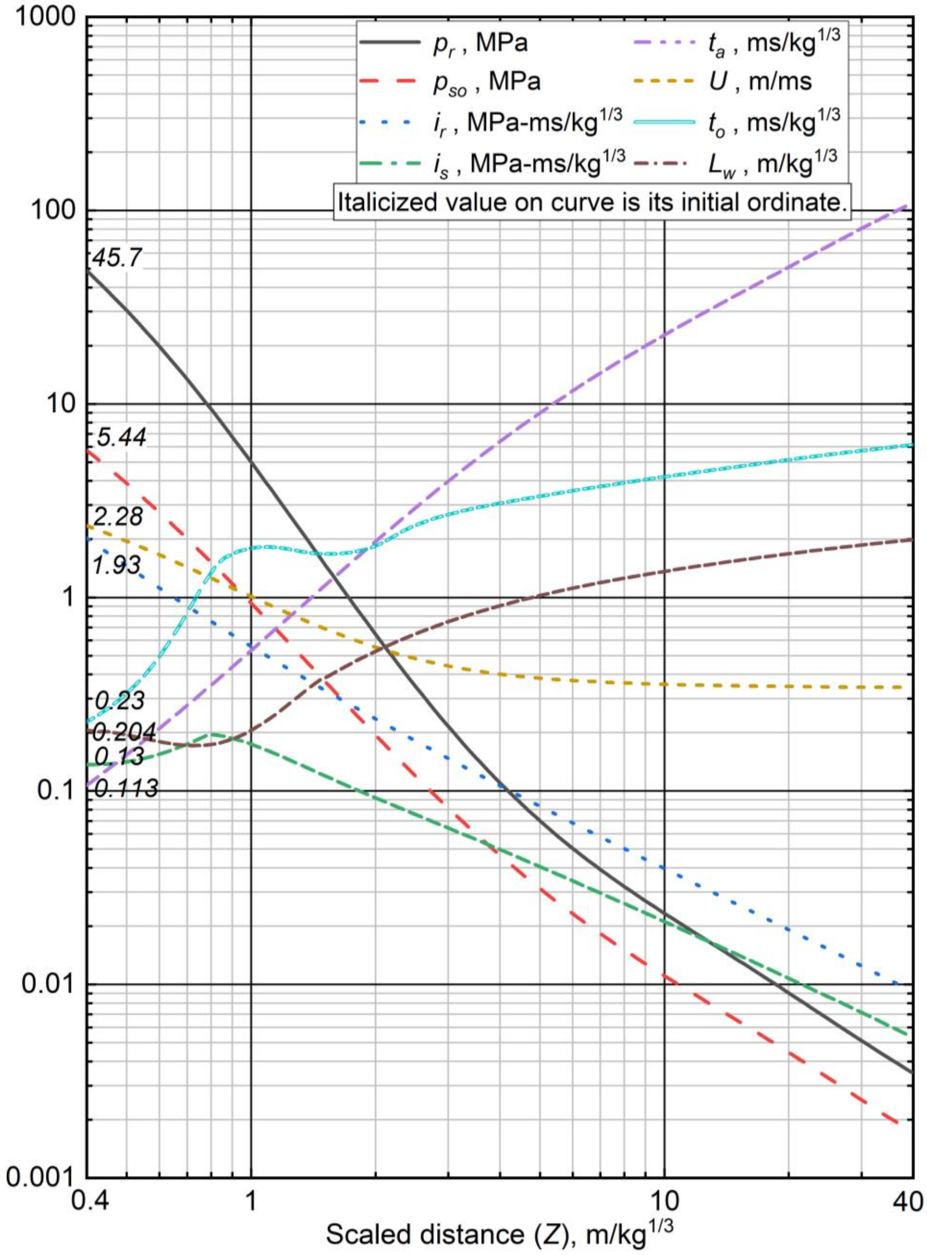


FIG. 7 POSITIVE PHASE SHOCK WAVE PARAMETERS FOR A SPHERICAL TNT EXPLOSION IN FREE AIR

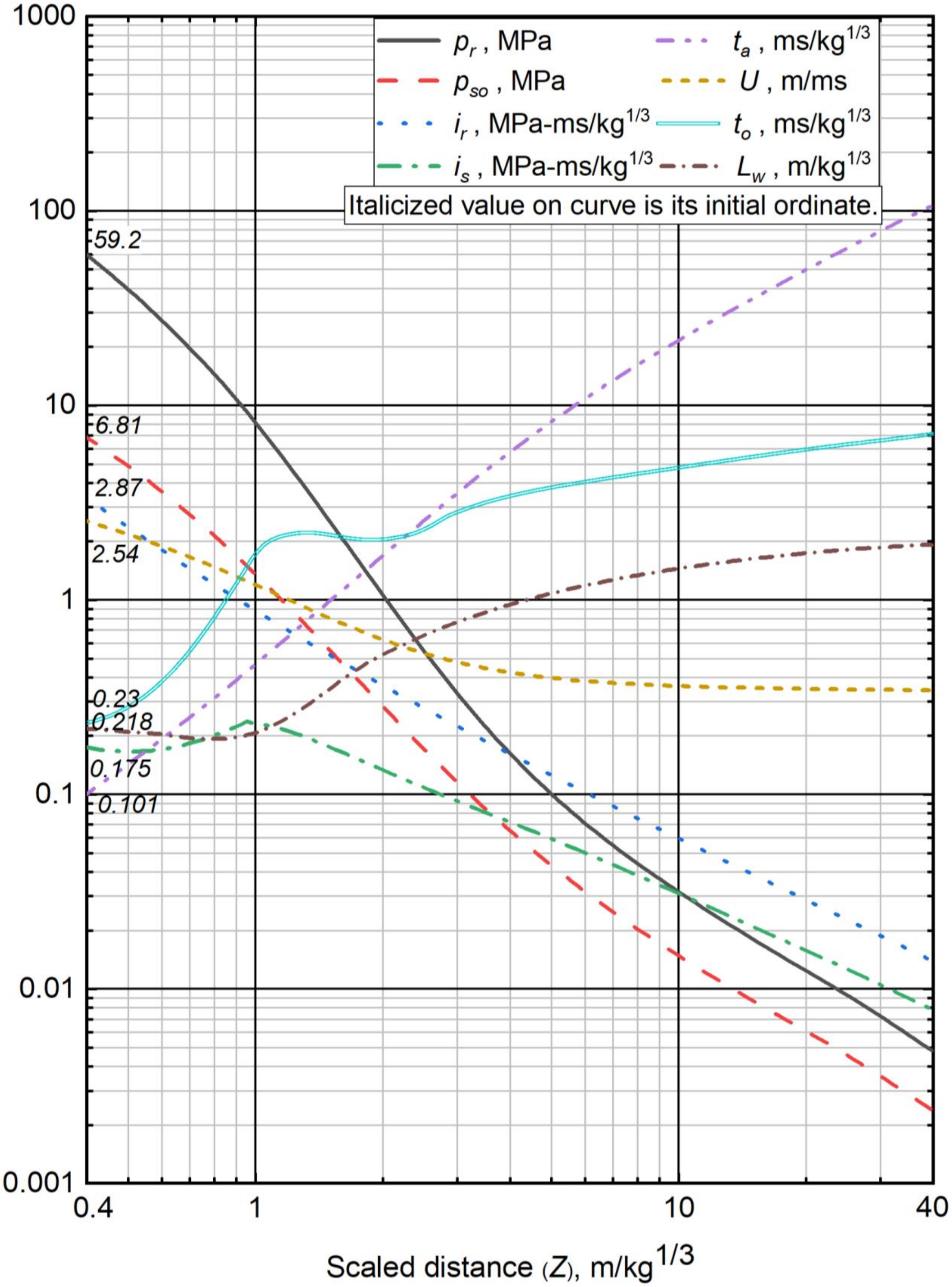


FIG. 8 POSITIVE PHASE SHOCK WAVE PARAMETERS FOR A HEMISPHERICAL TNT EXPLOSION ON SURFACE

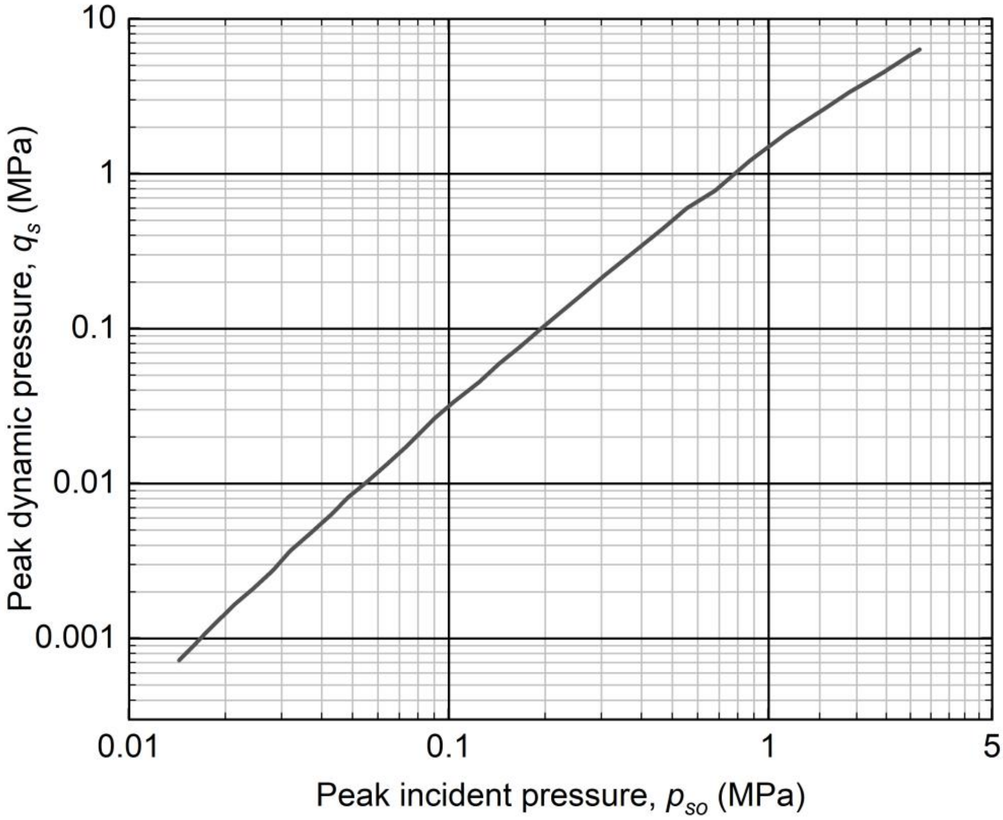


FIG. 9 PEAK DYNAMIC PRESSURE AND PEAK PARTICLE VELOCITY BEHIND A SHOCK FRONT

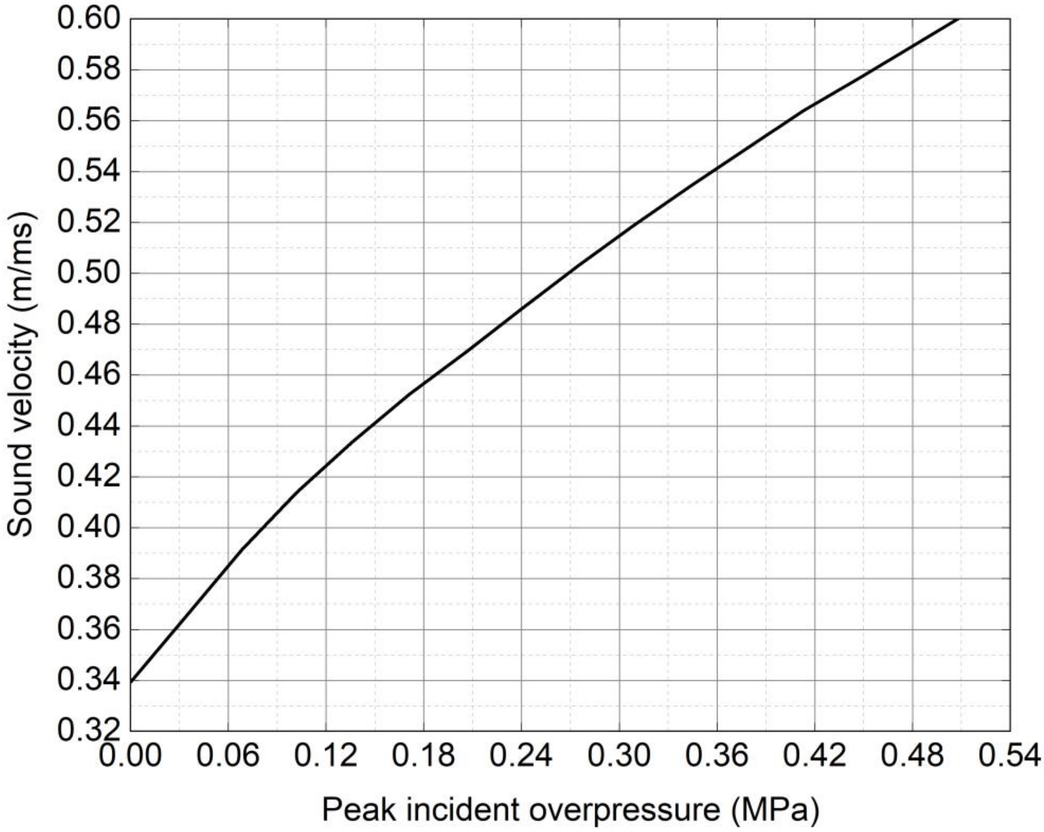


FIG. 10 SOUND VELOCITY IN THE REFLECTED OVERPRESSURE REGION

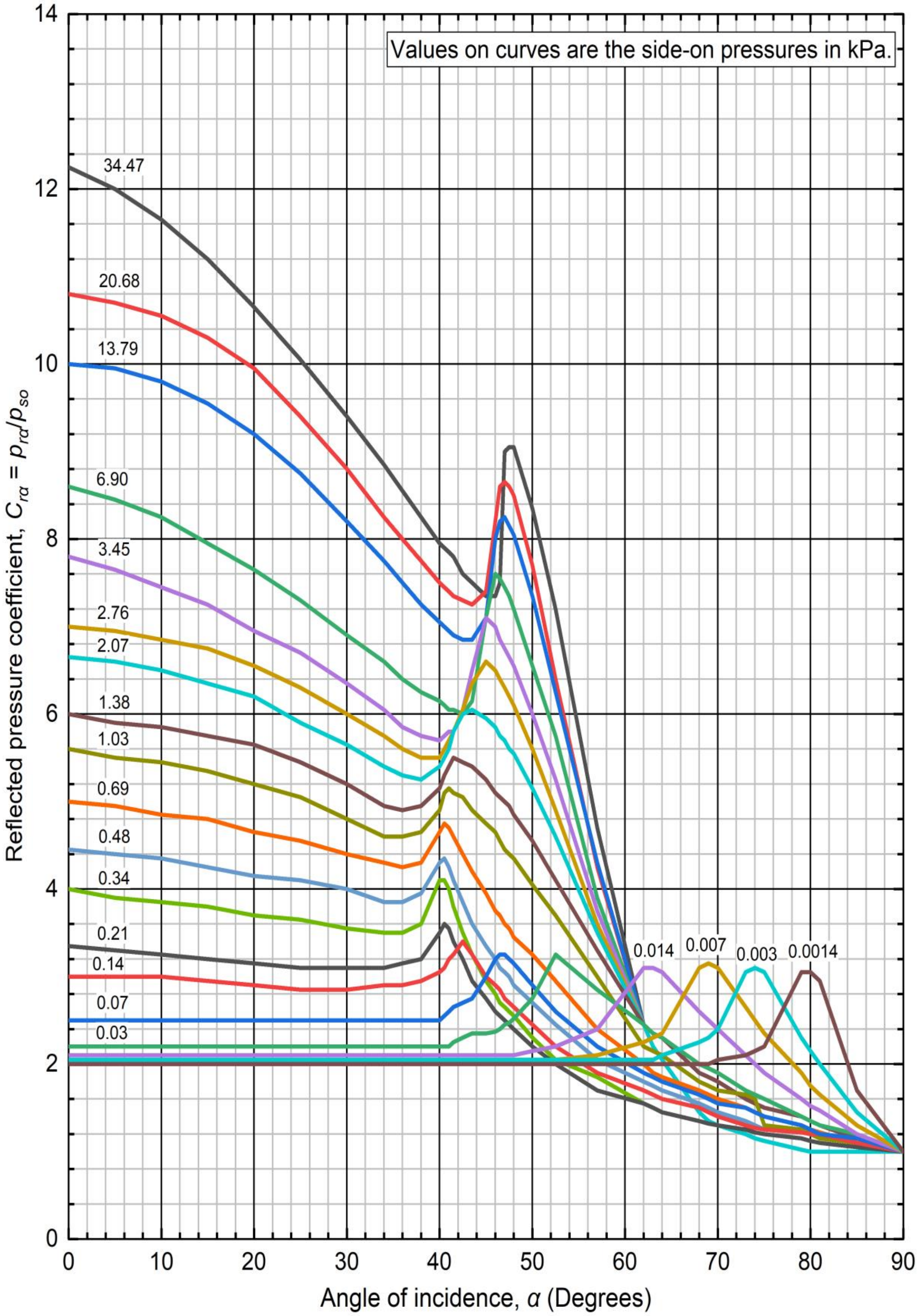


FIG. 11 INFLUENCE OF ANGLE OF INCIDENCE ON THE REFLECTED PRESSURE COEFFICIENT

7.5 Simplified Blast Pressure History

For the purpose of simplified analysis and design, the exponential pressure time relation in the positive phase may be idealised to a triangular pulse as shown in Fig. 12.

The triangular pulse shall be constructed to have the same peak pressure as in the original exponential blast pressure curve and the positive phase duration (t_d) so adjusted that the area under the curve remains unchanged.

The usage of triangular representation of pressure history shall be governed by relevant provisions of this standard.

7.6 Ground Shocks

7.6.1 *Ground Shaking Intensity*

The scaled peak particle displacement, peak particle velocity, and soil pressure generated by surface explosion shall be estimated using following equations:

Scaled peak particle displacement,

$$\frac{x}{M^{1/3}} = 60\,\frac{f}{c}\left(\frac{2.5R}{M^{1/3}}\right)^{1\text{-}n}$$

Soil pressure,

$$p = 160 f \rho c \left(\frac{R}{M^{1/3}}\right)^{-n}$$

where

f = coupling factor;

M = explosive weight;

n = attenuation coefficient;

ρ = soil density;

c = soil seismic velocity; and

R = standoff distance.

Table 6 shall be used for taking the values of c, pc and n.

The coupling factor f shall be taken as 0.14 for explosions above ground.

7.6.2 *Safe Ground Particle Velocity*

For safety of buried and semi-buried structures, the peak particle velocity, u, as calculated using equation provided in **7.6.1** shall not exceed the values given in Table 7.

For safety of unlined underground structures, the peak particle velocity, u, as calculated using equation provided in **7.6.1** shall not exceed the values listed in Table 8.

For safety of above ground structures, the peak particle velocity, u, measured at the foundation level, as calculated using equation provided in **7.6.1** shall not exceed the values listed in Table 9.

7.6.3 *Decay of Ground Shaking with Distance*

Decay of ground shaking with distance is to be estimated based on principle of mechanics and field measurements.

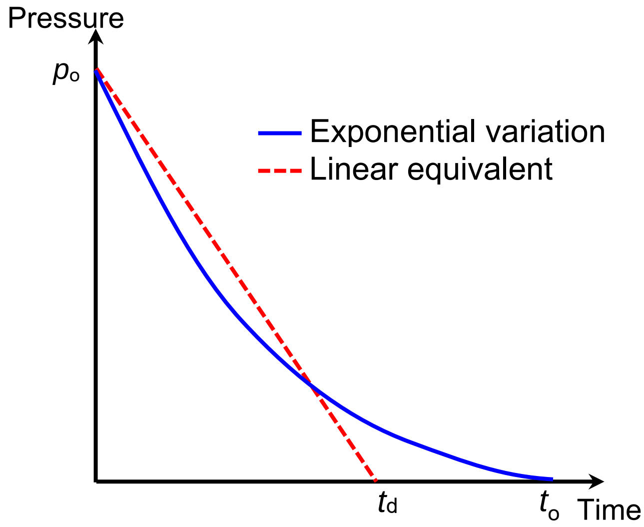


FIG. 12 SIMPLIFIED TRIANGULAR REPRESENTATION OF THE EXPONENTIAL OVERPRESSURE HISTORY

7.7 Confined Explosions

The provisions given here are applicable for scenarios where detonation of explosives takes place inside closed spaces.

For the purpose of evaluating blast load effects, confined explosions are categorized into:

a) fully vented; and

b) partially vented.

The examples of fully and partially vented structures are shown in Fig. 13.

The variation of pressure at any location inside a closed space due to detonation inside the closed space is obtained using the following assumptions:

a) Surfaces of internal structural members are subjected to shock waves from multiple reflections characterized by multiple peaks; and

b) Shock pressure is followed by development of gas pressure of small peak overpressures but significantly longer durations.

The provisions of 7.7 are only applicable when following conditions are satisfied:

a) Structure is of regular geometric shape; and

b) The effect of confined detonation is being evaluated at scaled distances greater than 3 m/kg$^{1/3}$.

Table 6 Soil Properties to Calculate Ground Shock Parameters

(*Clause* 7.6.1)

Sl No.	Material Description	Seismic Velocity, c (m/s)	Acoustic Impedance, ρc (MPa/m/s)	Attenuation Coefficient, n
(1)	(2)	(3)	(4)	(5)
i)	Loose, dry sands and gravels with low relative density	182.9	0.27	3 to 3.25
ii)	Sandy loam, loess, dry sands, and backfill	304.8	0.50	2.75
iii)	Dense sand with high relative density	487.7	1.00	2.5
iv)	Wet sandy clay with air voids (greater than 4 percent)	548.6	1.09	2.5
v)	Saturated sandy clays and sands with small amount of air voids (less than 1 percent)	1 524	2.94	2.25 to 2.5
vi)	Heavy saturated clays and clay shales	< 1 524	3.40 to 4.07	1.5

Table 7 Safe Ground Particle Velocity

(*Clause* 7.6.2)

Sl No.	Type of Rock	Safe Ground Particle Velocity (mm/s)
(1)	(2)	(3)
i)	Soils, weathered or soft rock	50
ii)	Hard rock	70

13A FULLY VENTED

13B PARTIALLY VENTED

FIG. 13 CONFINED EXPLOSION SCENARIO

Table 8 PPV Damage Criterion for Unlined Underground Structures

(*Clause* 7.6.2)

Sl No.	Rock Type	Unit Weight	Compressive Strength	Tensile Strength	Critical Peak Particle Velocity (m/s)			
		kg/m^3	MPa	MPa	No Damage	Slight Damage	Intermediate Damage	Serious Damage
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
i)	Hard rock	2 600 to 2 700	75 to 110	2.1 to 3.4	0.27	0.54	0.82	1.53
		2 700 to 2 900	110 to 180	3.4 to 5.1	0.31	0.62	0.96	1.78
		2 700 to 2 900	180 to 200	5.1 to 5.7	0.36	0.72	1.11	2.09
ii)	Soft rock	2 000 to 2 500	40 to 100	1.1 to 3.1	0.29	0.58	0.90	1.67
		2 000 to 2 500	100 to 160	3.4 to 4.5	0.35	0.70	1.07	1.99

Table 9 Permissible Peak Particle Velocity at the Foundation Level of Structures, in mm/s

(*Clause* 7.6.2)

Sl No.	Type of Structure	Dominant Excitation Frequency (Hz)		
		< 8 Hz	8 Hz to 25 Hz	> 25 Hz
(1)	(2)	(3)	(4)	(5)
i)	Buildings/structures not owned by owners:			
	a) Domestic houses/structure (*kuccha* brick and cement)	5	10	15
	b) Industrial buildings (RCC and framed structures)	10	20	25
	c) Objects of historical importance and sensitive structures	2	5	10
ii)	Buildings belonging to owner with limited span of life			
	a) Domestic houses/structures (*kuccha* brick and cement)	10	15	25
	b) Industrial buildings (RCC and framed structures)	15	25	50

7.7.1 *Fully Vented Structures*

Fully vented structures are to be primarily subjected to shock pressures and the effect of gas pressures may be neglected. The reflected overpressure history at any point inside the structure is represented in Fig. 14. The peak pressure and impulse for the first pulse may be obtained using the procedure described in **7.3** as a case of direct reflection of the blast wave propagating from the centre of detonation to the point of interest. The second and the third triangular pulse shall be constructed from the characteristics of the first pulse.

Further, simplification of pressure variation in Fig. 14 is allowed if the ratio of total load duration ($5t_a + t_d$) to the time-period of the structure is smaller than 0.1. In such cases, the three pulses of pressure history in Fig. 14 may be combined into a single pulse of peak pressure 1.75 p_{ro} and duration equal to t_d.

7.7.2 *Partially Vented Explosion*

Partially vented explosions are characterized by long duration gas pressures with minor effect of the shock pressures.

Wherever, required, the arrival time of the peak gas pressure is taken as the end time of the shock phase, that is, $5t_a + t_d$, as determined in **7.7.1**.

The blast load effects of confined explosion in partially vented structure is evaluated using advanced numerical technique as per the provisions of **9.3.3**.

The interior surface of a partially vented structure subject to confined explosion may conservatively be designed for the peak gas overpressure, p_g, which needs to be applied statically.

This peak gas overpressure is obtained from the chart in Fig. 15. The use of charts requires following condition to be satisfied:

$$0 \leq A / V_f^{2/3} \leq 0.022$$

where

A = structure's total vent area; and

V_f = free volume which shall be calculated as the total volume excluding the volume of all interior equipment and structural elements.

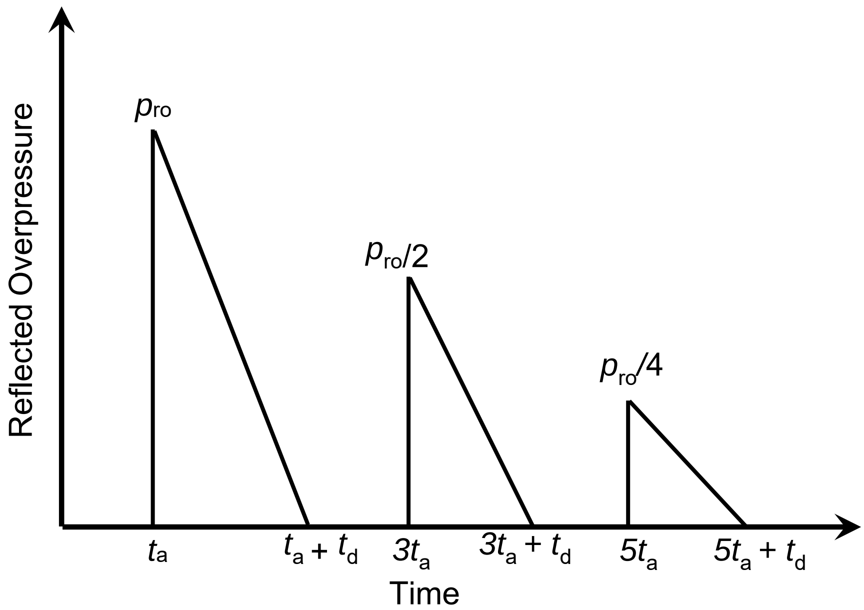


FIG. 14 REFLECTED OVERPRESSURE HISTORY FOR CONFINED DETONATION IN A FULL-VENTED SPACE

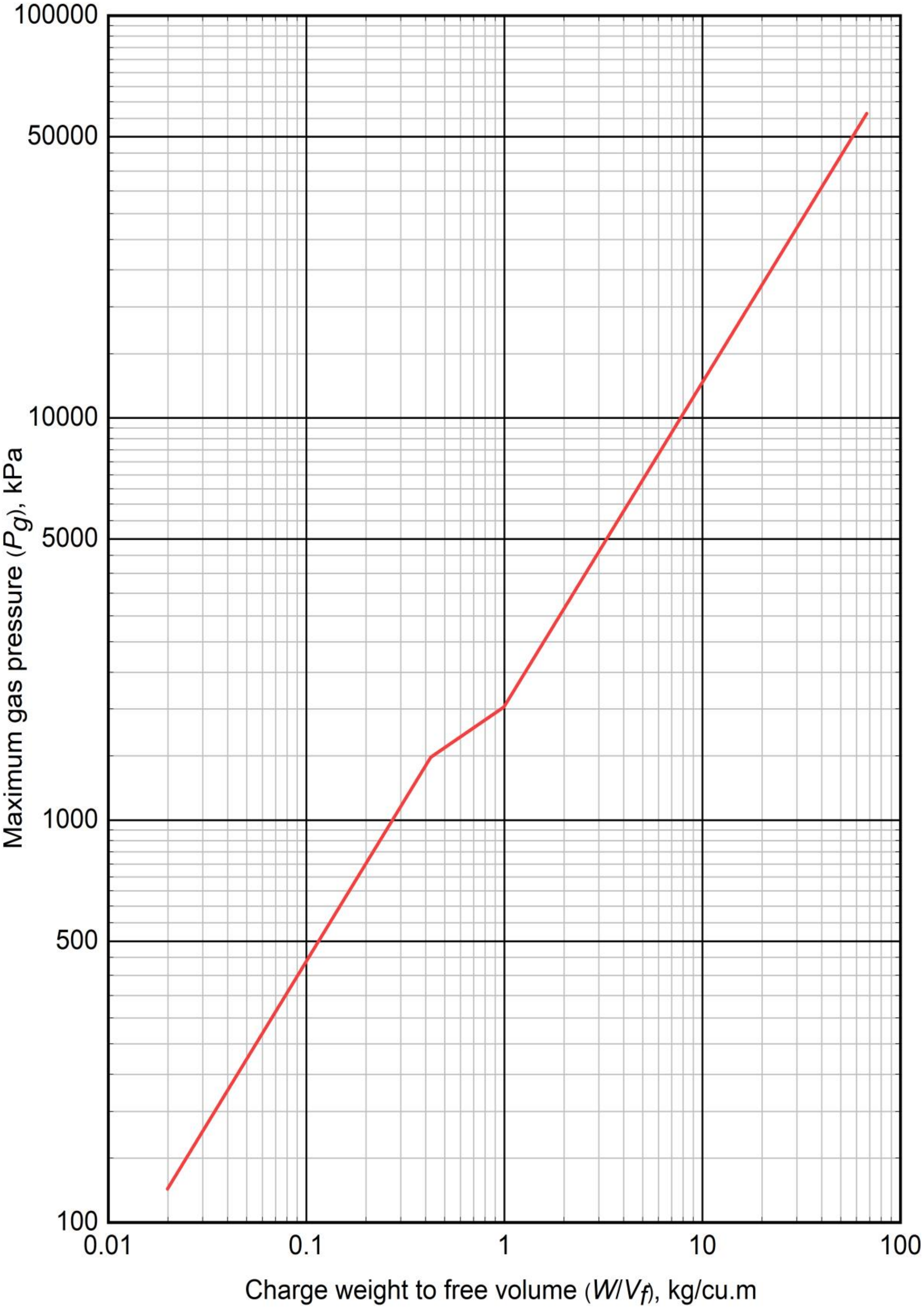


FIG. 15 PEAK GAS PRESSURE PRODUCED BY A TNT DETONATION IN A PARTIALLY VENTED STRUCTURE

8 EFFECTS OF BLAST ON STRUCTURES

8.1 Above Ground Structures

8.1.1 *Types of Structures*

Structures are categorized into following two types for the purpose of determining its interaction with the blast wave and calculating effective blast overpressure history:

a) *Diffraction type structures* — These are the closed structures without openings, with the total area opposing the blast. These are subjected to both the shock wave overpressure p_{so} and the dynamic pressures q caused by blast wind; and

b) *Drag type structures* — These are the open structures composed of elements like beams, columns, trusses, etc, which have small projected area opposing the shock wave. These are mainly subjected to dynamic pressures q.

8.1.2 *Closed Rectangular Structures*

8.1.2.1 *Front face*

The peak reflected overpressure at any point on the front face of a closed rectangular structure shall be obtained using the following equation.

$$p_{r\alpha} = C_{r\alpha} p_{so}$$

where

p_{so} = peak side-on pressure that shall be obtained using Fig. 7 and Fig. 8 for spherical and hemispherical detonations, respectively, using the effective scaled distance; and

$C_{r\alpha}$ = reflection co-efficient, which shall be determined from Fig. 11 using the angle of reflection and the side-on pressure.

It is permitted to neglect spatial variation of pressure on the front face by assuming an equivalent uniform reflected overpressure which is equal to the peak reflected overpressure on the front face that is closest to the point of detonation. This, under most scenario, corresponds to the peak reflected overpressure, p_{ro}, at normal incidence ($\alpha = 0$ degrees).

The reflected overpressure, p_r, on the front face drops from the peak value p_{ro} to zero in time t_{rf}, whichever, is calculated as, $t_{rf} = 2i_{ro}/p_{ro}$, where p_{ro} and i_{ro} shall be obtained from using Fig. 7 and Fig. 8 for spherical and hemispherical detonations, respectively.

The stagnation overpressure, p_s, at the edges of the front face drops from the peak value, $p_{so} + C_d q_o$, to zero in time t_{of}, whichever, is calculated as, $t_{of} = 2i_{so}/p_{so}$, where p_{so} and i_{so} shall be obtained using Fig. 7 and Fig. 8 for spherical and hemispherical detonations, respectively. The drag coefficient, C_d, shall be taken as 1.0. The peak dynamic pressure, q_o, shall be determined from Fig. 9.

The reflected overpressure acting at a point on the front face drops from the peak value p_{ro} to overpressure ($p_{so} + C_d q$) in clearance time t_c or t_{of}, whichever, is less (*see* Fig. 17).

The drag coefficient, C_d, for different faces of closed rectangular structures located above ground shall be taken as per Table 10.

The clearing time, t_{cp}, for a point on the front wall is given by

$$t_{cp} = \frac{3S}{U}$$

where

S = H or $W/2$ whichever, is less (*see* Fig. 16); and

U = shock front velocity to be obtained from using Fig. 7 and Fig. 8 for spherical and hemispherical detonations, respectively.

The average clearing time, t_c, required to clear the front face of reflection effects from the roof down and inwards from the sides shall be calculated using the following expression:

$$t_c = \frac{4HW}{C_r(W + 2H)}$$

where

C_r = velocity of sound in the reflected overpressure region that shall be determined using Fig. 10.

The average net loading on the front face ($W \times H$) as a function of time is shown in Fig. 18 depending on whether t_c is smaller than or equal to t_{of}. The pressures p_{ro}, p_{so} and q, the impulses t_{ro} and I_{so} are for the actual explosion that are determined according to **7.3**.

8.1.2.2 *Rear face*

Using the pressures for the actual explosion, the average loading on the rear face ($W \times H$) in Fig. 16 shall be taken as shown in Fig. 19, where the time has been taken from the instant the shock first strikes the front face. The time intervals of interest are the following:

$\frac{L}{U}$ = the travel time of shock from front to rear face; and

$\frac{4S}{U}$ = pressure rise time on the back face.

8.1.2.3 *Roof and side walls*

The average pressure versus time curve for roof and side walls is given in Fig. 20A when t_d is greater than the transit time $t_t = L/U$. When t_t is greater than t_d, the load on roof and side walls may be considered as a moving triangular pulse having the peak value of overpressure $p_{so} + C_d q_o$ and time t_o as shown in Fig. 20B.

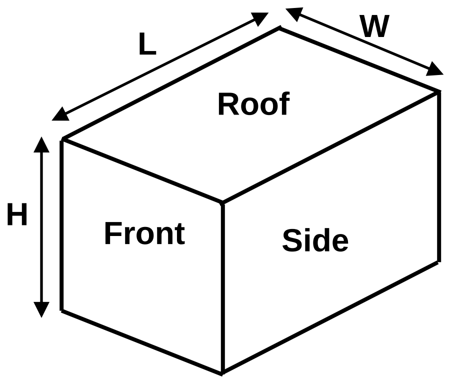


FIG. 16 ABOVE GROUND RECTANGULAR STRUCTURE

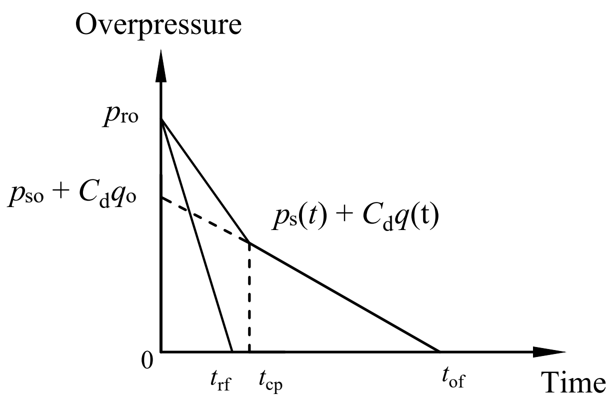


FIG. 17 REFLECTED OVERPRESSURE AT A POINT ON THE FRONT SURFACE OF FINITE REFLECTING SURFACE

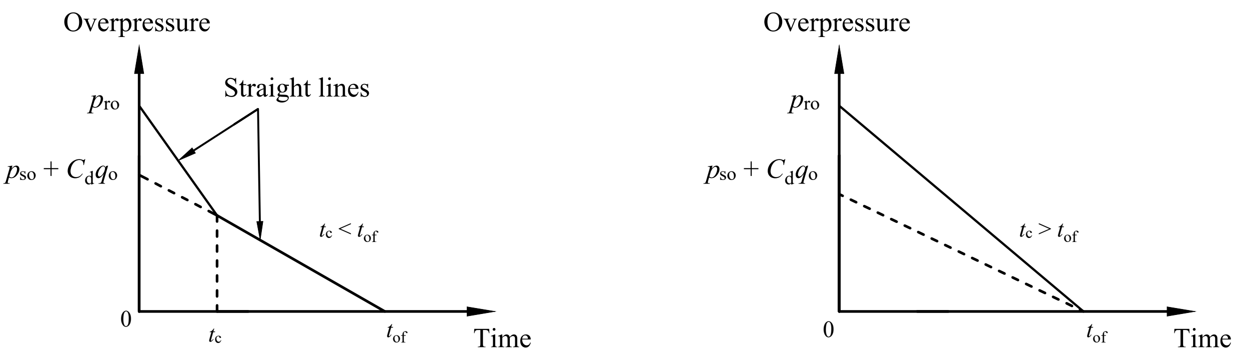


FIG. 18 REFLECTED OVERPRESSURE HISTORY ON THE FRONT FACE OF A CLOSED RECTANGULAR STRUCTURE

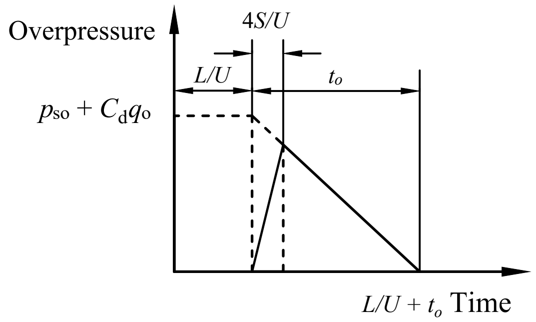


FIG.19 OVERPRESSURE VERSUS TIME FOR REAR FACE

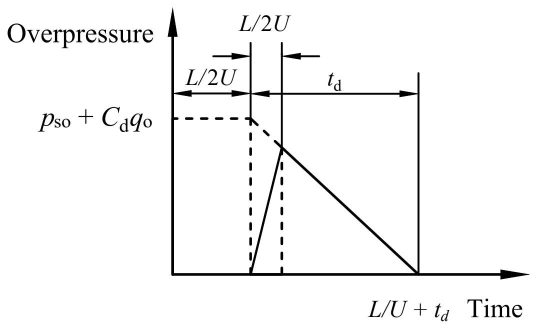


FIG. 20A AVERAGE PRESSURE DIAGRAM FOR $t_t < t_d$

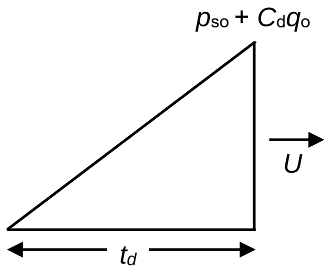


FIG. 20B MOVING PRESSURE PULSE FOR $t_t > t_d$

FIG. 20 OVERPRESSURE VERSUS TIME FOR ROOF AND SIDE WALLS

Table 10 Drag Coefficient C_d

(*Clause* 8.1.2.1)

Sl No.	Shape of Element	Drag Coefficient C_d	Remarks
(1)	(2)	(3)	(4)
i)	Front vertical face	1.0	For closed rectangular structures located above ground
ii)	Roof, near and side faces for:		
	a) q_o = 0 kPa to 170 kPa	- 0.4	
	b) q_o = 170 kPa to 350 kPa	- 0.3	
	c) q_o = 350 kPa to 1 MPa	- 0.2	

8.1.2.4 *Overturning of structure*

The average net load as a function of time which tends to cause sliding and overturning of the building is obtained by subtracting the loading on back face from that on the front face.

8.1.3 *Closed Cylindrical Arch-Shape Structures*

8.1.3.1 *Gable ends*

The loading may be taken same as for front and rear faces of above ground rectangular structure.

8.1.3.2 *Curved surface*

The direction of shock wave propagation is taken transverse to the ridge of the structure and since the usual arch spans are large so that the transit time t_t is greater than the positive phase time t_d, average loading condition cannot be assumed. Therefore, the loading on curved surface may be taken as a moving triangular pulse as shown in Fig. 20B.

8.1.3.3 *Closed dome structures*

The loading on a domical structure may be taken as a moving triangular pressure pulse as shown in Fig. 20B. The variation of pressure transverse to the direction of propagation of the pulse may be considered symmetrical varying according to cosine θ where the angle θ is measured from longitudinal vertical section of the dome, as shown in Fig. 21.

8.2 Blast Load on Below-Ground Structures

8.2.1 *Types of Structures*

The below-ground structures are classified into buried and semi-buried structures depending upon the earth cover and slopes of earth berms. The buried structures are subjected only to the general overpressure p_{so}, the reflected and dynamic pressures being neglected. The semi-buried structures are subjected to partial dynamic pressures besides the general overpressure. Both are acted upon by air-induced ground shock also.

For assessing the effects of above-ground blasts on below-ground structures, dynamic finite element analysis can be performed considering the flexibility of soil and interactions of blast induced ground waves with the structure.

9 RESPONSE OF STRUCTURAL ELEMENTS

9.1 Properties of Materials

The nominal properties of materials are obtained as per relevant codes and guidelines. The change in material properties due to high strain rates associated with blast response are to be incorporated using the provisions given in **9.1.1** and **9.1.2**.

9.1.1 *Elastic Modulus and Poisson's Ratio*

The elastic modulus and Poisson's ratio of the constituent materials used in the structural elements shall be estimated based on relevant standards as applicable to that material.

No increase in elastic modulus and Poisson's ratio is to be considered for the calculation of blast response of structures.

9.1.2 *Dynamic Strength*

9.1.2.1 *Design strength of structural steel*

The average dynamic increase factors of strength for structural, carbon, mild, weldable or rivet steels and high strength alloy steels are listed in Table 11 and Table 12.

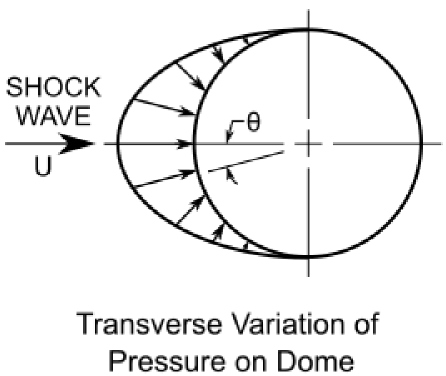


FIG. 21 TRANSVERSE VARIATION OF PRESSURE ON DOME

Table 11 Dynamic Increase Factors for Mild Steels

(*Clause* 9.1.2.1)

Sl No.	Material	Dynamic Increase Factors
(1)	(2)	(3)
i)	Yield strength in bending and shear	1.25
ii)	Yield strength in tension and compression	1.20
iii)	Ultimate strength	1.00

Table 12 Dynamic Increase Factors for High Strength Steels

(*Clause* 9.1.2.1)

Sl No.	Materials	Dynamic Increase Factors
(1)	(2)	(3)
i)	Yield strength in bending and shear	1.10
ii)	Yield strength in tension and compression	1.05
iii)	Ultimate strength	1.00

9.1.2.2 *Design strength of reinforced concrete*

The average dynamic strength increase factors for concrete are listed in Table 13 and for reinforcing steel are listed in Table 14.

9.1.2.3 *Design strength for masonry or plain concrete*

The average dynamic strength increase factors for masonry and plain concrete are listed in Table 15.

9.1.3 *Design Strength*

For obtaining design strength, the resistance of various elements is to be estimated using the strength increased by the dynamic increase factors mentioned in **9.1.2**.

9.1.4 *Deformation Limits*

Structural elements subjected to blast loading can be designed subject to certain allowable dynamic ductility ratios and support rotations.

The blast resistance of a flexural member is to be expressed in terms of maximum permissible deflection. Support rotation and ductility is to be used to characterize the blast resistance of reinforced concrete and structural steel members, respectively.

The support rotation shall be defined as per Fig. 22.

The maximum permissible deflection defines the energy absorption capacity of the element which is equal to the area under the resistance versus deflection curve. For a given blast scenario, greater the permissible deflection, lesser will be the maximum resistance required in the member.

The allowable deformation parameters are included in Table 16.

9.2 Modelling of Structural Members

9.2.1 *Geometry, Material and Boundary Conditions*

The methods of modelling are applicable to structural members of regular prismatic sections.

Table 13 Dynamic Increase Factors for Reinforced Concrete

(*Clause* 9.1.2.2)

Sl No.	Strength	Dynamic Increase Factors
(1)	(2)	(3)
i)	Flexure	1.15
ii)	Compression	1.10
iii)	Diagonal tension, direct shear, and bond	1.00

Table 14 Dynamic Increase Factors for Reinforcing Steel

(*Clause* 9.1.2.2)

Sl No.	Strength	Dynamic Increase Factors
(1)	(2)	(3)
i)	Yield strength in flexure	1.2
ii)	Yield strength in compression, diagonal tension, direct shear, and bond	1.1
iii)	Ultimate strength in flexure	1.1
iv)	Ultimate strength in compression, diagonal tension, direct shear, and bond	1.0

Table 15 Dynamic Increase Factors for Masonry and Plain Concrete

(*Clause* 9.1.2.3)

Sl No.	Strength	Dynamic Increase Factors
(1)	(2)	(3)
i)	Flexure	1.15
ii)	Compression	1.10
iii)	Diagonal tension and direct shear, and bond	1.00

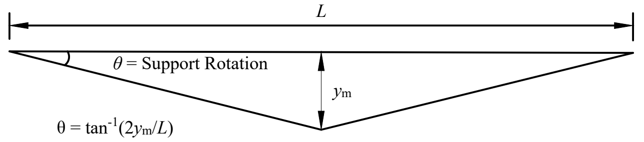


FIG. 22 SUPPORT ROTATION IN A FLEXURAL MEMBER

Table 16 Allowable Deformation Parameters

(*Clause* 9.1.4)

Sl No.	Material	Low Damage	Medium Damage	High Damage
(1)	(2)	(3)	(4)	(5)
i)	Ductility ratio in structural steel members subjected to bending and direct stresses	3	10	20
ii)	Support rotation in reinforced concrete members subjected to bending and direct stresses (in degrees)	1	2	4

9.2.2 *Equivalent Single Degree of Freedom* (*SDOF*) *Model*

The structural element or system may be replaced by an equivalent single spring-mass system having effective stiffness k_E and effective mass equal to $K_{LM}\ M_t$, where M_t is the actual mass of the member under consideration and K_{LM} is a load-mass factor depending upon the stiffness and mass distribution in the member and its boundary conditions. The equivalent system is defined so that the deflection of the equivalent single mass is the same as that of some significant point in the given structure. The effective stiffness k_E is defined with respect to the deflection of this point.

The load-mass factor K_{LM} is equal to the ratio of mass factor K_M to the load factor K_L. The factors are evaluated on the basis of an assumed deflected shape of the structure as given below:

$$K_M = \frac{1}{M_t}\ [\sum_{r=1}^{n} M_r \emptyset_r^2 + \int m\emptyset^2(x)\ dx]$$

$$K_L = \frac{1}{P_t}\ [\sum_{r=1}^{j} p_r \emptyset_r + \int p(x)\emptyset(x) dx]$$

where

M_t = total actual mass;

n = number of concentrated masses;

M_r = concentrated mass at point r;

ϕ_r = deflection at point r of an assumed deflected shape for concentrated loads;

m = distributed mass intensity per unit length;

$\phi_{(x)}$ = deflection at point x of an assumed deflected shape for distributed loads;

P_t = total dynamic load (at any instant of time);

j = number of concentrated load points;

P_r = concentrated dynamic force at point r; and

$p_{(x)}$ = intensity of distributed dynamic load per unit length.

9.2.2.1 *Effective single SDOF parameters*

Values of the factors k_E and K_{LM} for some structural members are given in Table 17 to Table 19. The effective SDOF parameters for other type of structural members shall be calculated by assuming an appropriate deflected shape as per **9.2.2.2**.

9.2.2.2 *Deflected shape*

The deflected shape is suitably chosen to resemble as far as possible the true deflected shape taking into consideration whether the structure or member remains elastic or goes into the plastic range. It may be taken the same as due to static application of the dynamic load on the structure. The deflected shape is normalized such that $\varphi(x) = 1$ at the point with respect to which the effective stiffness k_E is defined.

9.2.2.3 *Effective time-period*

The effective time period T of the structural member may be calculated from the equation:

$$T = 2\pi \sqrt{\frac{K_{LM}\ M_t}{k_E}}$$

9.2.3 *Resistance Function*

The resistance versus deflection diagram of a structural element shall be idealized as elasto-plastic (*see* Fig. 23) by keeping the area under the actual and idealized curves about the same up to the maximum permissible deflection defined as per **9.1.4**.

A trilinear resistance versus deflection diagram may be simplified to its elasto-plastic (bilinear) equivalent of same peak resistance (R_m) and the elastic parameters calculated using following equations:

$$y_E = y_e + y_p \left(1 - \frac{R_m}{R_e}\right)$$

$$k_E = \frac{R_m}{y_E}$$

9.3 Methods of Analysis

These provisions apply to structures, components, and systems subject to near-field and far-field detonations but not for contact or near-contact detonations.

9.3.1 *Analysis of Members*

Blast loaded structural components of a structure may be analysed independently provided it can be demonstrated that such independent member analysis would yield conservative performance of the member and the structure.

9.3.1.1 *Equivalent SDOF analysis*

The equivalent SDOF analysis may be used to determine response of structural members subject to blast loads when the following criteria are satisfied:

a) The equivalent mass, stiffness, and damping parameters in the model must capture the dynamic response and failure mode of the structural member being analysed; and

b) A single flexural deformation mode controls the dynamic response of the structural system.

The equivalent SDOF analysis shall make use of following parameters:

a) Blast-pressure history obtained using **7.3**;

b) The effective time-period calculated using **9.2.2.3**;

c) Resistance function of the structural element established using provisions of **9.2.3**; and

d) Maximum permissible deflection provided in **9.1.4**.

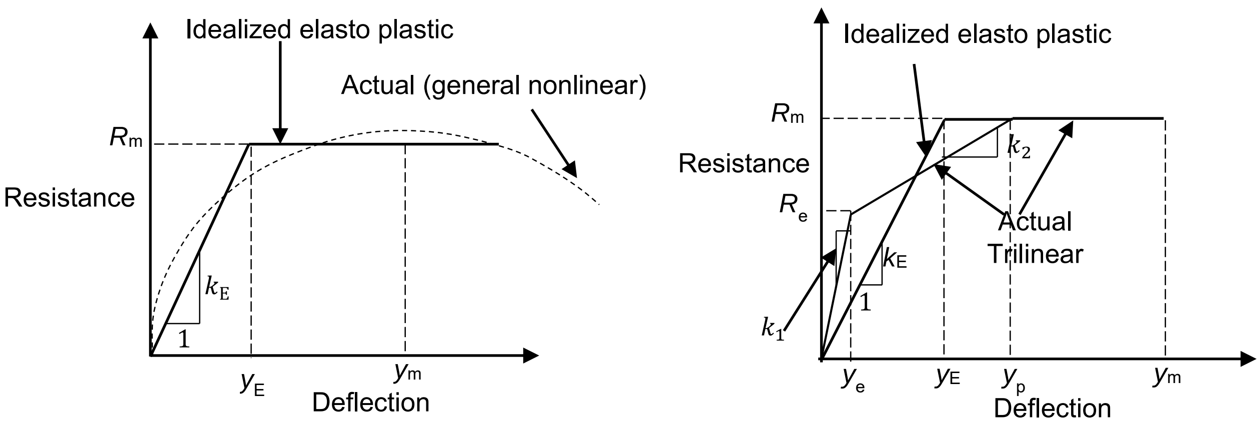
9.3.1.2 *Blast overpressure history*

The exponential blast pressure history of Fig. 5 shall be idealized to a triangular pulse (Fig. 24A) as per provisions of **7.5** for the purpose of obtaining equivalent SDOF blast loads. Such idealization may neglect the effect of negative phase, unless otherwise it is expected to influence the flexure response of the structural member.

The positive phase duration of the triangular loading shall be calculated as:

$$t_d = \frac{2\, i_{ro}}{p_{ro}}$$

If the influence of clearing on the blast pressure history applied to the front face of the structural element is found to be significant, then the equivalent SDOF blast loads shall be idealized to Fig. 24B using **8.1.2**.



23A GENERAL NONLINEAR

23B TRILINEAR

FIG. 23 IDEALIZATION OF RESISTANCE DEFLECTION DIAGRAM

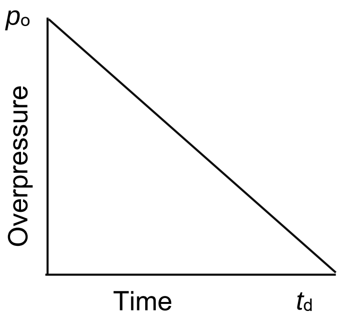


FIG. 24A LINEAR

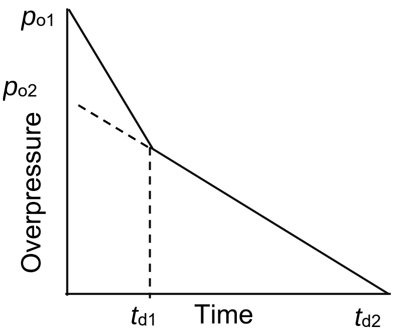


FIG. 24B BILINEAR

FIG. 24 IDEALIZED BLAST PRESSURE HISTORIES

When the ratio of time duration t_{d1}, or t_{d2}, to the natural period of the element is less than 0.1, the problem may be considered as an impulse problem taking the area under the pressure versus time curve as impulse per unit area. In such a case, the shape of pressure-time curve is not important.

9.3.1.3 *Uniform blast pressure distribution*

A uniform distribution of blast pressure over the length of the structural member may be considered provided one of the following conditions are satisfied:

a) The minimum scaled distance to the structural component is greater than 1.2 m/kg$^{1/3}$; and

b) The variation of peak reflected pressures over the middle two-third of the component span length is less than 25 percent.

For uniform distribution of blast pressure, the pressure value calculated at the closest point on the structure from the centre of detonation may be considered over the whole reflecting surface.

9.3.1.4 *Non-uniform blast pressure distribution*

For members subjected to spatially non-uniform blast loads, the effect of non-uniformity on the response shall be considered.

These provisions shall be applicable for non-uniform blast pressure distribution, provided the conditions outlined in 9.3.1.3 are not satisfied and the minimum scaled distance to the structural component is between 0.4 m/kg$^{1/3}$ and 1.2 m/kg$^{1/3}$.

An equivalent impulse weighted using deflected shape function shall be considered for the structural component as per following expression:

$$i_e = \frac{\int_0^L \int_0^H i\,(x,y)\ \phi(x,y)dxdy}{\int_0^L \int_0^H \phi(x,y)dxdy}$$

where

i_e	=	equivalent impulse of uniformly distributed load;
L	=	length of loaded area;
H	=	width of loaded area;
i(x,y)	=	non-uniform impulse applied to loaded area.
$\emptyset(x,y)$	=	deflected shape function of blast-loaded component; and

The simplified triangular representation of the pressure time history shall be constructed as per provisions of 7.5 by calculating the positive loading phase duration using the equivalent uniform impulse (i_e) and the peak overpressure at the midspan or geometric centre of the reflecting surface.

A safety factor of 1.25 shall be used if a non-uniform blast load is represented as an equivalent uniform impulse in with blast analysis.

9.3.1.5 *Elastic SDOF response*

For elastic analysis (ductility ratio $\mu \leq 1.0$) of structures, the effective stiffness k_E and load mass factor K_{LM} shall be used as given in Table 17 to Table 19.

For elastic response of an SDOF system, the peak response (y_m) and the time to peak response (t_m) may be obtained using following expressions:

$$\frac{y_m}{y_0} = 2 - \frac{t_m}{t_d}; \qquad \text{when } t < t_d$$

$$t_m = \frac{2}{\omega} \tan^{-1}(\omega t_d) \qquad \text{when } t < t_d$$

$$\frac{y_m}{y_0} = \frac{1}{\omega t_d} \sqrt{2(1 - \cos \omega t_d - \omega t_d \sin \omega t_d) + (\omega t_d)^2}; \qquad \text{when } t > t_d$$

$$t_m = \frac{1}{\omega} \tan^{-1}\left(\frac{1 - \cos \omega t_d}{\sin \omega t_d - \omega t_d}\right) \qquad \text{when } t > t_d$$

where

y_m = peak elastic displacement;

y_o = peak static displacement; and

t_m = time to the peak displacement.

9.3.1.6 *Inelastic SDOF response*

For elasto-plastic analysis, k_E shall be used as given in Table 17 to Table 19 but value of K_{LM} may be chosen in between the elastic and plastic cases depending upon the ductility factor.

Based on the elasto-plastic resistance-deflection curve shown in Fig. 23 and triangular pressure-time curve shown in Fig. 24A, the ratio of resistance R_m, required to the peak dynamic load (F_1) is given in Fig. 25 for various values of t_d/T and μ.

The time to peak deflection (t_m) shall be obtained from Fig. 26 for various values of t_d/T and μ.

When the time ratio, t_d/T, is less than 0.1, the ratio k may be computed from the following equation:

$$k = \frac{\pi}{\sqrt{2\mu - 1}} \cdot \frac{t_d}{T}$$

When the pressure-time diagram is given by Fig. 24B, the following equation shall be satisfied:

$$\frac{P_{01}}{R_m} k_1 + \frac{P_{02}}{R_m} k_2 = 1$$

where

R_m = the required resistance; and

k_1, k_2 = the values of ratios k for ductility ratio μ and time ratios t_{d1}/T and t_{d2}/T respectively.

For elasto-plastic design of fixed slabs, the modified value of k_E is to be worked out in accordance with Fig. 23B using the stiffness values of slab in elastic and elasto-plastic cases as given in Table 19 and K_{LM} is to be suitably chosen depending upon the ductility factor.

The value of modular ratio shall be taken the same as in static design for calculating *EI*.

For calculating moment of inertia *I* of reinforced concrete sections, the concept of effective transformed area shall be used.

For analysis of reinforced concrete sections when the simplified elasto-plastic resistance function is used, the elastic stiffness shall be based on effective moment of inertia that shall be calculated as the average of the gross moment of inertia of the transformed section and the cracked moment of inertia.

The gross and cracked moment of inertias shall be obtained from Fig. 27.

9.3.2 *MDOF Analysis*

The multi degree of freedom (MDOF) analysis is permitted for structural systems which cannot be analysed using the simplified SDOF analysis, provided the expected failure mechanism under blast load can adequately be represented using the MDOF model representation.

9.3.2.1 The numerical model of MDOF system may comprise of distributed elements or combined lumped mass and stiffness elements.

9.3.2.2 The spring in the lumped-mass stiffness should adequately represent the member resistance-deflection behaviour by incorporating appropriate constitutive relationship.

9.3.2.3 The lumped-mass stiffness system should be able to properly account for the dynamic behaviour and failure modes at the structure and the element level.

9.3.2.4 The use of structural damping is permitted for MDOF system when the expected response is within elastic range. These structural damping values for elastic response shall conform to the typical values used for different type of structures.

9.3.2.5 The energy dissipation during inelastic response should be accounted by using appropriate nonlinear member resistance-deflection curves. Supplemental damping in form of structural damping is not permitted for nonlinear range of response.

9.3.2.6 The time-step for MDOF analysis shall be determined based on accuracy and convergence requirement. The time-step should be sufficiently small such that it captures the rise and decay of the applied blast load time history.

9.3.3 *Finite Element Analysis*

Advanced numerical analysis techniques, such as explicit dynamic finite element analysis (FEA), may be used to analyse structures subject to blast loading, if the methods described in 9.3.1 and 9.3.2 cannot be reliably used to obtain the response. Such methods shall be required if one or more of the following conditions are encountered:

a) Detonation involves scaled distances smaller than 0.4 m/kg$^{1/3}$;

b) Detonation of non-spherical charge masses at scaled distances smaller than 3 m/kg$^{1/3}$;

c) The blast wave propagation involves interaction with surrounding environment before it reaches the location of interest;

d) The variation in geometric and material properties of structural elements or the structure cannot appropriately be represented using the simplified methods; and

e) The interaction between the blast wave and structure is of significance and the assumption of rigid reflecting surface is not valid.

The FEA shall consider the modelling of non-ductile failure modes and failure surfaces that cannot be considered in the simplified SDOF and the MDOF methods of analysis.

The time-step of analysis need to be decided based on size of the smallest element of interest with appropriate consideration to the accuracy and convergence of the response.

The interaction of blast load with the surrounding shall be considered using computational fluid dynamics (CFD) methods.

Where the shape and size of the charge mass is expected to play a major role in the detonation process and the resulting blast load, the detonation process itself may be modelled.

The FEA methods shall not be subjected to the response limits typically used for structural members in the simplified SDOF and the MDOF analysis.

10 LOAD COMBINATIONS FOR DESIGN

10.1 Partial Safety Factor

A partial safety factor of unity shall be assumed for blast loads.

10.2 Imposed Loads

Imposed load on floors shall be considered as per specifications of IS 875 (Part 2). No Imposed load shall be considered on roof at the time of blast.

10.3 Load Combination

Wind or earthquake forces shall not be assumed to occur simultaneously with blast effects. Effects of temperature and shrinkage shall be neglected.

Following load combinations for limit state design shall be considered:

$$1.0\, DL + 0.5\, IL + 1.0\, B$$
$$0.9\, DL + 1.0\, B$$

where

DL = design dead load;

IL = design imposed load; and

B = design blast load.

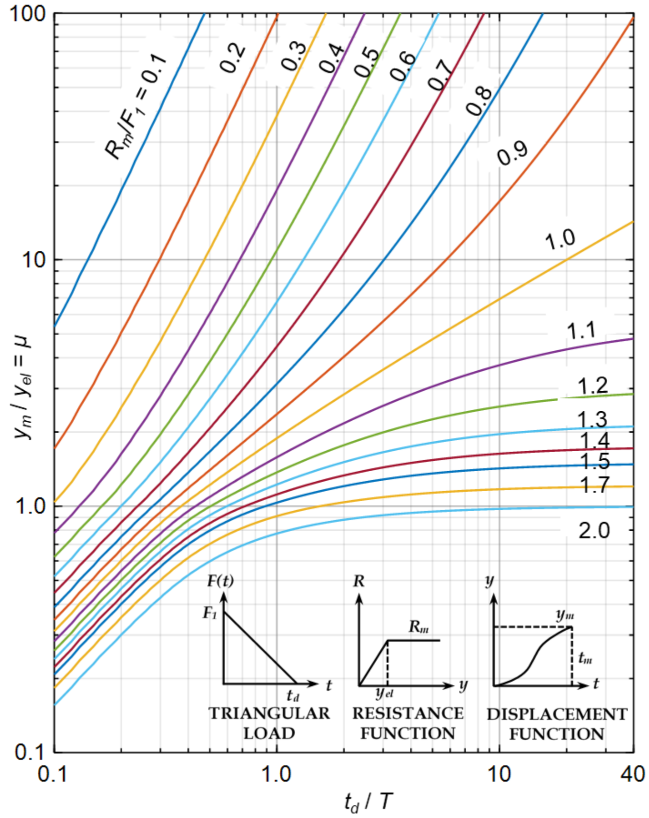


FIG. 25 PEAK DUCTILITY DEMAND OF SDOF SYSTEM SUBJECT TO TRIANGULAR PULSE LOADING

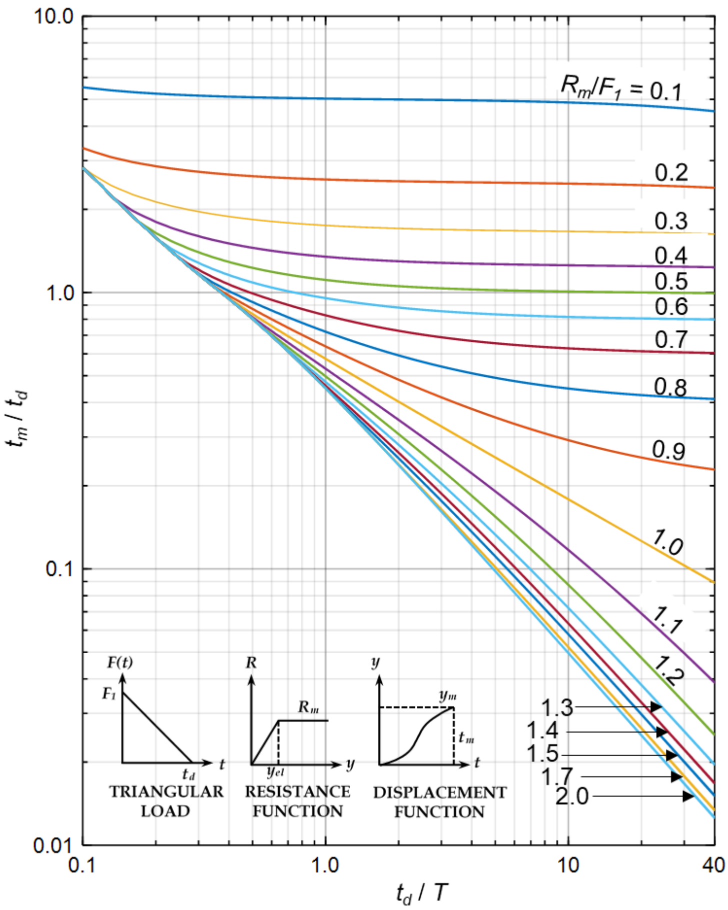
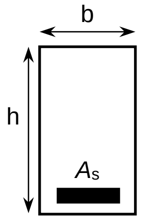
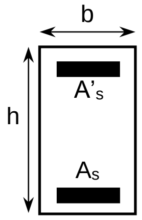
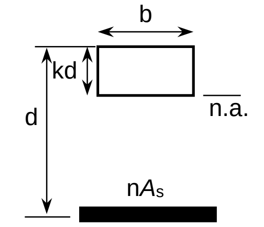
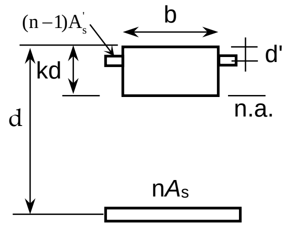


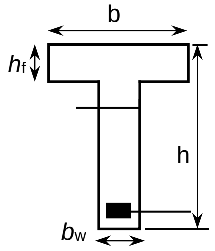
FIG. 26 TIME TO PEAK DUCTILITY DEMAND OF SDOF SYSTEM SUBJECT TO TRIANGULAR PULSE

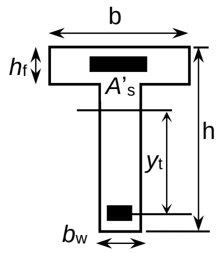
<table>
<tr><th>Gross Section</th><th>Cracked Transformed Section</th><th>Gross and Cracked Moment of Inertia</th></tr>
<tr>
<td>




</td>
<td>


Without compression steel


With compression steel
</td>
<td>
$n = \frac{E_s}{E_c}$
$B = \frac{b}{(nA_s)}$
$I_g = \frac{bh^3}{12}$
Without compression steel:
$kd = (\sqrt{2dB + 1} - 1)/B$
$I_{cr} = bk^3d^3/3 + nA_s(d - kd)^2$
With compression steel:
$r = (n - 1)A'_s/(nA_s)$
$kd = \left[\sqrt{2dB(1+rd'/d) + (1 + r)^2} - (1 + r)\right]/B$
$I_{cr} = bk^3d^3/3 + nA_s(d - kd)^2 + (n - 1)A'_s(kd - d')^2$
</td>
</tr>
<tr>
<td>




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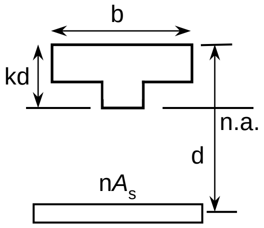
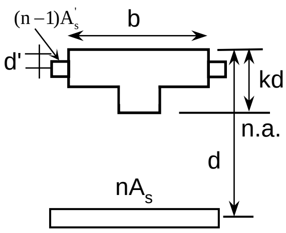
<td>


Without compression steel


With compression steel
</td>
<td>
$n = \frac{E_s}{E_c}$
$C = b_w/(nA_s), f = h_f(b - b_w)/(nA_s)$
$y_t = h - 1/2\left[(b - b_w)h_f^2 + b_w h^2\right]/\left[(b - b_w)h_f + b_w h\right]$,
$I_g = (b - b_w)h_f^3/12 + b_w h^3/12 + (b - b_w)h_f(h - h_f/2 - y_t)^2 + b_w h(y_t - h/2)^2$
Without compression steel:
$kd = \left[\sqrt{C(2d + h_f f) + (1 + f)^2} - (1 + f)\right]/C$
$I_{cr} = (b - b_w)h_f^3/12 + b_w k^3d^3/3 + (b - b_w)h_f(kd - h_f/2)^2 + nA_s(d - kd)^2 + (n - 1)A_s(kd - d')^2$
With compression steel:
$kd = \left[\sqrt{C(2d + h_f f + 2rd') + (f + r + 1)^2} - (f + r + 1)\right]/C$
$I_{cr} = (b - b_w)h_f^3/12 + b_w k^3d^3/3 + (b - b_w)h_f(kd - h_f/2)^2 + nA_s(d - kd)^2 + (n - 1)A'_s(kd - d')^2$
</td>
</tr>
</table>

FIG. 27 CRACKED MOMENT OF INERTIA OF PRISMATIC SECTIONS

Table 17 Transformation Factors for Beams and One-way Slabs

(*Clauses* 9.2.2.1, 9.3.1.5 *and* 9.3.1.6)

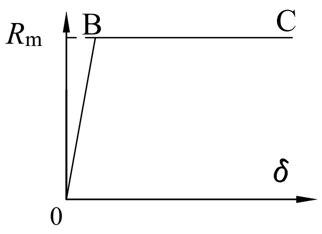
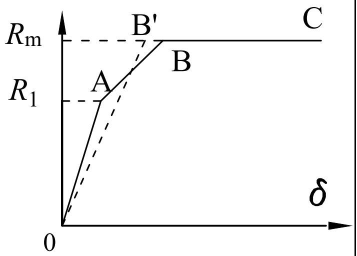
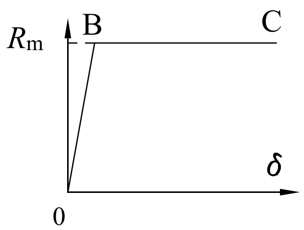
Sl No.	Resistance Curve, End-Conditions of Beams or One-way Slabs	Dynamic Loading	Strain Range	Load-Mass Factor K_{LM}		Maximum Resistance R_m	Effective Spring Constant k_E	Dynamic Reaction
				Concentrated Mass	Uniform Mass			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
i)		Uniform $P_t = pL$	Elastic	–	0.78	$8M_p/L$	$384EI/5L^3$	$0.39\ R_m + 0.11\ P_t$
			Plastic	–	0.66	$8M_p/L$	–	$0.38\ R_m + 0.12\ P_t$
		Concentrated at mid-span P_t=P	Elastic	1.0	0.49	$4M_p/L$	$48EI/L^3$	$0.78\ R_m - 0.28\ P_t$
			Plastic	1.0	0.33	$4M_p/L$	–	$0.75\ R_m - 0.25\ P_t$
		Concentrated at third points P/2 each $P_t = P$	Elastic	0.87	0.60	$6M_p/L$	$56.4EI/L^3$	$0.62\ R_m - 0.12\ P_t$
			Plastic	1.0	0.56	$6M_p/L$	–	$0.52\ R_m - 0.02\ P_t$
ii)	 Fixed at both ends	Uniform $P_t = pL$	Elastic (OA)	–	0.77	$12M_{ps}/L$	$384EI/L^3$	$0.36\ R_1 + 0.14\ P_t$
			Elastic (OB')	–	0.78	$(8/L)(M_{ps} + M_{pm})$	$307EI/L^3$	$0.39\ R_m + 0.11\ P_t$
			Plastic	–	0.66	$(8/L)(M_{ps} + M_{pm})$	–	$0.38\ R_m + 0.12\ P_t$
		Concentrated at mid-span $P_t = P$	Elastic (OB)	1.0	0.37	$(4/L)(M_{ps} + M_{pm})$	$192EI/L^3$	$0.71\ R_m - 0.21\ P_t$
			Plastic	1.0	0.33	$(4/L)(M_{ps}+ M_{pm})$	–	$0.75\ R_m - 0.25\ P_t$

Table 17 (*Concluded*)

Sl No.	**Resistance Curve, End-Conditions of Beams or One-way Slabs**	**Dynamic Loading**	**Strain Range**	**Load-Mass Factor K_{LM}**		**Maximum Resistance R_m**	**Effective Spring Constant k_E**	**Dynamic Reaction**
				Concentrated Mass	Uniform Mass			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
iii)	Fixed at one end and simply supported at the other	Uniform $P_t = pL$	Elastic (OA)	–	0.78	$(8/L)(M_{ps})$	$185EI/L^3$	$V_1 = 0.26R_1 + 0.12P_t$ $V_2 = 0.43R_1 + 0.19P_t$
			Elastic (OB')	–	0.78	$(4/L)(M_{ps} + 2M_{pm})$	$160EI/L^3$	$0.39\,R_m + 0.11\,P_t \pm M_{ps}/L$
			Plastic	–	0.66	$(4/L)(M_{ps} + 2M_{pm})$	–	$0.38\,R_m + 0.12\,P_t \pm M_{ps}/L$
		Concentrated at mid-span $P_t = P$	Elastic (OA)	1.0	0.43	$(16/3L)M_{ps}$	$107EI/L^3$	$V_1 = 0.54R_1 + 0.14P_t$ $V_2 = 0.25R_1 + 0.07P_t$
			Elastic (OB')	1.0	0.49	$(2/L)(M_{ps} + 2M_{pm})$	$106EI/L^3$	$0.78\,R_m - 0.28\,P_t \pm M_{ps}/L$
			Plastic	1.0	0.33	$(2/L)(M_{ps} + 2M_{pm})$	–	$0.75\,R_m - 0.25\,P_t \pm M_{ps}/L$

where

L = span;

p = dynamic pressure;

P_t = dynamic load;

M_p = plastic moment of section;

M_{pm} = M_p at mid-span

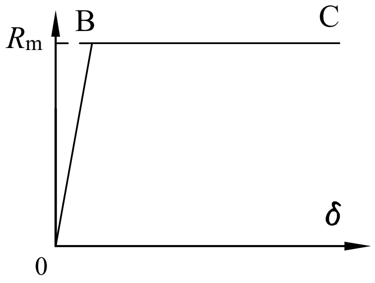
M_{ps} = M_p at support;

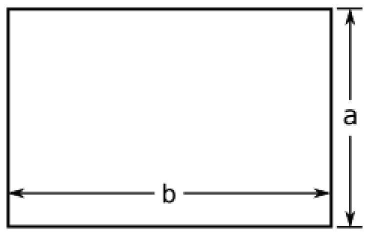
R_m = maximum resistance; and

k_E = effective spring constant.

Table 18 Transformation Factors for Two-way Slabs — Four Sides, Simply Supported, Uniformly Loaded

(*Clauses* 9.2.2.1, 9.3.1.5 *and* 9.3.1.6)





Sl No.	Strain Range	a/b	Load Mass Factor K_{LM}	Max Resistance R_m	Spring Constant k_E	Dynamic Reactions	
						V_A	V_B
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	Elastic (OB)	1.0	0.68	$12(M_{pfa} + M_{pfb})/a$	$252EI/a^2$	$0.07P_r + 0.18R_m$	$0.07P_r + 0.18R_m$
		0.9	0.70	$(12M_{pfa} + 11M_{pfb})/a$	$230EI/a^2$	$0.06P_r + 0.16R_m$	$0.08P_r + 0.20R_m$
		0.8	0.71	$(12M_{pfa} + 10.3M_{pfb})/a$	$212EI/a^2$	$0.06P_r + 0.14R_m$	$0.08P_r + 0.22R_m$
		0.7	0.73	$(12M_{pfa} + 9.8M_{pfb})/a$	$201EI/a^2$	$0.05P_r + 0.13R_m$	$0.08P_r + 0.24R_m$
		0.6	0.74	$(12M_{pfa} + 9.3M_{pfb})/a$	$197EI/a^2$	$0.04P_r + 0.11R_m$	$0.09P_r + 0.26R_m$
		0.5	0.75	$(12M_{pfa} + 9.0M_{pfb})/a$	$201EI/a^2$	$0.04P_r + 0.09R_m$	$0.09P_r + 0.28R_m$

Table 18 (*Concluded*)

Sl No.	Strain Range	a/b	Load Mass Factor K_{LM}	Max Resistance R_m	Spring Constant k_E	Dynamic Reactions	
						V_A	V_B
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
ii)	Plastic (BC)	1.0	0.51	$12(M_{pfa} + M_{pfb})/a$	–	$0.09P_r + 0.16R_m$	$0.09P_r + 0.16R_m$
		0.9	0.51	$(12M_{pfa} + 11M_{pfb})/a$	–	$0.08P_r + 0.15R_m$	$0.09P_r + 0.18R_m$
		0.8	0.54	$(12M_{pfa} + 10.3M_{pfb})/a$	–	$0.07P_r + 0.13R_m$	$0.10P_r + 0.20R_m$
		0.7	0.58	$(12M_{pfa} + 9.8M_{pfb})/a$	–	$0.06P_r + 0.12R_m$	$0.10P_r + 0.22R_m$
		0.6	0.58	$(12M_{pfa} + 9.3M_{pfb})/a$	–	$0.05P_r + 0.10R_m$	$0.10P_r + 0.25R_m$
		0.5	0.59	$(12M_{pfa} + 9.0M_{pfb})/a$	–	$0.04P_r + 0.08R_m$	$0.11P_r + 0.27R_m$

M_{pfa} = total positive ultimate moment capacity along mid-span section parallel to short edge;

M_{pfb} = total positive ultimate moment capacity along mid-span section parallel to long edge;

I = moment of inertia per unit width of slab;

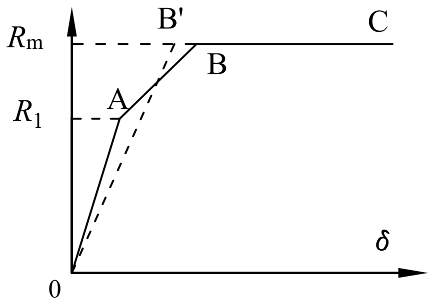
P_t = dynamic load;

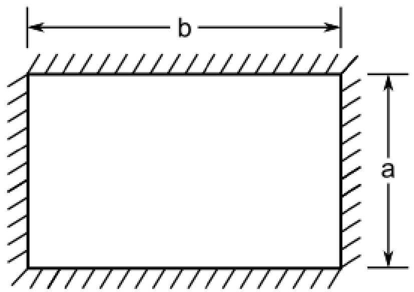
V_A = total dynamic reaction along a short edge; and

V_B = total dynamic reaction along a long edge.

Table 19 Transformation Factors for Two-way Slabs — Fixed Four Sides, Uniform loaded

(*Clauses* 9.2.2.1, 9.3.1.5 *and* 9.3.1.6)





Sl No.	Strain Range	a/b	Load Mass Factor K_{LM}	Max Resistance R_m	Spring Constant k_E	Dynamic Reactions	
						V_A	V_B
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	Elastic (OA)	1.0	0.63	29.2 M^o_{psb}	810 EI/a^2	$0.10P_t + 0.15R_1$	$0.10P_t + 0.15R_1$
		0.9	0.68	27.4 M^o_{psb}	742 EI/a^2	$0.09P_t + 0.14R_1$	$0.10P_t + 0.17R_1$
		0.8	0.69	26.4 M^o_{psb}	705 EI/a^2	$0.08P_t + 0.12R_1$	$0.11P_t + 0.19R_1$
		0.7	0.71	26.2 M^o_{psb}	692EI/a^2	$0.07P_t + 0.11R_1$	$0.11P_t + 0.21R_1$
		0.6	0.71	27.3 M^o_{psb}	724 EI/a^2	$0.06P_t + 0.09R_1$	$0.12P_t + 0.23R_1$
		0.5	0.72	30.2 M^o_{psb}	806 EI/a^2	$0.05P_t + 0.08R_1$	$0.12P_t + 0.25R_1$
ii)	Elasto-plastic (OB')	1.0	0.67	$(1/a)[12(M_{pfa} + M_{psa}) + 12(M_{pfb} + M_{psb})]$	252 EI/a^2	$0.07P_t + 0.18R_m$	$0.07P_t + 0.18R_m$
		0.9	0.70	$(1/a)[12(M_{pfa} + M_{psa}) + 11(M_{pfb} + M_{psb})]$	230 EI/a^2	$0.06P_t + 0.16R_m$	$0.08P_t + 0.20R_m$
		0.8	0.71	$(1/a)[12(M_{pfa} + M_{psa}) + 10.3(M_{pfb} + M_{psb})]$	212 EI/a^2	$0.06P_t + 0.14R_m$	$0.08P_t + 0.22R_m$

Table 19 (*Concluded*)

Sl No.	Strain Range	a/b	Load Mass Factor K_{LM}	Max Resistance R_m	Spring Constant k_E	Dynamic Reactions	
						V_A	V_B
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
		0.7	0.73	$(1/a)[12(M_{pfa} + M_{psa}) + 9.8(M_{pfb} + M_{psb})]$	$201\ EI/a^2$	$0.05P_t + 0.13R_m$	$0.08P_t + 0.24R_m$
		0.6	0.74	$(1/a)[12(M_{pfa} + M_{psa}) + 9.3(M_{pfb} + M_{psb})]$	$197\ EI/a^2$	$0.04P_t + 0.11R_m$	$0.09P_t + 0.26R_m$
		0.5	0.75	$(1/a)[12(M_{pfa} + M_{psa}) + 9.0(M_{pfb} + M_{psb})]$	$201\ EI/a^2$	$0.04P_t + 0.09R_m$	$0.09P_t + 0.28R_m$
iii)	Fully plastic (BC)	1.0	0.51	$(1/a)[12(M_{pfa} + M_{psa}) + 12(M_{pfb} + M_{psb})]$	–	$0.09P_t + 0.16R_m$	$0.09P_t + 0.16R_m$
		0.9	0.51	$(1/a)[12(M_{pfa} + M_{psa}) + 11(M_{pfb} + M_{psb})]$	–	$0.08P_t + 0.15R_m$	$0.09P_t + 0.18R_m$
		0.8	0.54	$(1/a)[12(M_{pfa} + M_{psa}) + 10.3(M_{pfb} + M_{psb})]$	–	$0.07P_t + 0.13R_m$	$0.10P_t + 0.20R_m$
		0.7	0.58	$(1/a)[12(M_{pfa} + M_{psa}) + 9.8(M_{pfb} + M_{psb})]$	–	$0.06P_t + 0.12R_m$	$0.10P_t + 0.22R_m$
		0.6	0.58	$(1/a)[12(M_{pfa} + M_{psa}) + 9.3(M_{pfb} + M_{psb})]$	–	$0.05P_t + 0.10R_m$	$0.10P_t + 0.25R_m$
		0.5	0.59	$(1/a)[12(M_{pfa} + M_{psa}) + 9.0(M_{pfb} + M_{psb})]$	–	$0.04P_t + 0.08R_m$	$0.11P_t + 0.27R_m$

Range (OA) = moment at centre of long edge just becomes plastic;

Range (OB') = moment at supports and mid-span sections just becomes plastic;

M^o_{psb} = negative ultimate moment capacity per unit width at centre of long edge;

M_{psa} = total negative ultimate moment capacity along a short edge support;

M_{psb} = total negative ultimate moment capacity along a long edge support;

M_{pfa}, M_{pfb}, I, P_t = (*see* Table 5);

V_A = total dynamic reaction along a short edge; and

V_B = total dynamic reaction along a long edge.

ANNEX A

(*Foreword*)

COMMITTEE COMPOSITION

Earthquake Engineering Sectional Committee, CED 39

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Visvesvaraya National Institute of Technology, Nagpur	DR M. D. GOEL

(*Continued from second cover*)

The methods described in the standard to estimate blast overpressures are independent of the type of structure (for example, buildings, bridges, etc) and shall be applicable provided loading criteria in relevant sections are satisfied. The applicability of an analysis method is governed by the response characteristics (like flexure) of the blast loaded system. The simplified SDOF methods of analysis can be adopted for flexure response evaluation of individual blast loaded member.

The Committee involved with formulation of this standard has incorporated the views of various stakeholders from academia, industry, security agencies, and defence organisations. As such, information presented in this standard has been compiled from the knowledge and literature available in the public domain.

In the formulation of this standard, assistance has been derived from the following publications:

American Society of Civil Engineers (ASCE) (2011) "Blast protection of buildings" ASCE/SEI 59-11, Reston, VA.

DoD (2008) "Structures to resist the effects of accidental explosions" Report No. UFC-3-340-02, US Department of Defence, Washington, DC.

American Society of Civil Engineers (ASCE) (2010) "Design of blast-resistant buildings in petrochemical facilities" Task Committee on Blast-Resistant Design of the Petrochemical Committee of the Energy Division of ASCE, Reston, VA.

Canadian Standards Association (CSA) (2012) "Design and assessment of buildings subjected to blast loads" S850-12, Canada.

This standard contributes to the United Nations Sustainable Development Goal 9: 'Industry, Innovation and Infrastructure', particularly its target to develop quality, reliable, sustainable and resilient infrastructure, and also promote inclusive and sustainable industrialization.

The composition of the Committee responsible for the formulation of this standard is given in Annex A.

For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or estimated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2 : 2022 'Rules for rounding off numerical values (*second revision*)'. The number of significant places retained in the rounded-off value should be the same as that of the specified value in this standard.

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