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Indian Standard

CRITERIA FOR BLAST RESISTANT DESIGN OF STRUCTURES FOR EXPLOSIONS ABOVE GROUND

(Third Reprint AUGUST 1993)

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BUREAU OF INDIAN STANDARDS
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
NEW DELHI 110002

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Indian Standard

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Earthquake Engineering Sectional Committee, BDC 39

(*Continued on page* 2)

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Indian Standard

CRITERIA FOR BLAST RESISTANT DESIGN OF STRUCTURES FOR EXPLOSIONS ABOVE GROUND

0. FOREWORD

0.1 This Indian Standard was adopted by the Indian Standards Institution on 17 November 1968, after the draft finalized by the Earthquake Engineering Sectional Committee had been approved by the Civil Engineering Division Council.

0.2 Structures designed to resist blast loads are subjected to completely different type of load than that considered in conventional design. Here they are hit with a rapidly moving shock wave which may exert pressures many times greater than those experienced under the greatest of hurricanes. However, in blast phenomenon, the peak intensity lasts for a very small duration only.

0.3 The blast wave loads the exposed surface of the structure and then the load is transmitted to the other elements. Thus, the response of each individual element is important unlike the ground motion wherein the whole structural system is simultaneously causing inertia effects on all parts.

0.4 To design a structure capable of resisting these intense but short duration loads, members and joints are permitted to deflect and strain much greater than is allowed for usual static loads. This permitted deflection is, ordinarily, well into the plastic range of the material. Large amounts of energy are absorbed during this action, thus reducing the required design strength considerably below that required by conventional design within elastic range. Moreover, under higher rates of loading the strength developed by the material, increases with the rate of loading, and may often be adequately described as a function of time within a certain range.

0.5 Whereas, if the location of the ground zero (*see* **2.8**), and the size of bomb are known, the corresponding blast loading for an existing structure may be found by the methods explained in this standard. However, it will never be possible to have exact data for specifying the expected ground zero and bomb size. Therefore, three different standard blast loadings (*see* **12.2**), are recommended in this standard. Nevertheless, this standard also contains necessary information for evaluating various parameters of a blast wave generated by any other size of explosion at a given distance and the design of a structure for the same. It may be mentioned that if a building receives a shock stronger than the one for which it is designed, it is likely to suffer a higher category of damage.

0.6 Fires may generally follow an air raid, it is, therefore, important to design structures against fire. Attention to the relevant Indian Standards on fire safety is, therefore, invited.

0.7 Flying splinters from a bomb may also cause considerable damage to the portion of the structure exposed to them. Wall thicknesses considered safe against flying splinters from a bomb are given in Appendix C.

0.8 In the formulation of this standard, in addition to other publications the following have been consulted:

BIGGS (J M). Introduction to structural dynamics, 1964. McGraw Hill Book Company, Inc, New York.

KINNEY (G F). Explosive shocks in air, 1962. The Machmillan Company, New York.

NORRIS (C H), HANSEN (R J), HOLLEY JR (M J), BIGGS (J M), NAYMET (S), and MINAMI (J K). Structural design for dynamic loading, 1959. McGraw Hill Book Co, Inc, New York.

Design of structures to resist nuclear weapon effects. Manual of Engineering Practic No. 42, 1964. American Society of Civil Engineers, New York.

UNITED STATES OF AMERICA. Department of Army, Army Corps of Engineers. Design of structures to resist the effect of atomic weapons, Manual No. TM-5-856-3, 1957, Government Printing Office, Washington.

0.9 For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS : 2-1960*. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

1. SCOPE

1.1 This standard covers the criteria for design of structures for blast effects of explosions above ground. This standard does not cover the design for blast effects of nuclear explosions.

*Rules for rounding off numerical values (*revised*).

2. DEFINITIONS

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

2.1 Blast Wind — It is the moving air mass along with the over-pressures resulting from pressure difference behind the shock wave front. The blast wind movement during the positive phase of the overpressures is in the direction of shock front propagation.

2.2 Clearance Time — This is the time in which the reflected pressure decays down to the sum of the side on overpressure and the drag pressure.

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

2.4 Drag Force — It is the force on a structure or structural element due to the blast wind. On any structural element, the drag force equals dynamic pressure multiplied by the drag coefficient of the element.

2.5 Ductility Ratio — It is the ratio of the maximum deflection to the deflection corresponding to the elastic limit.

2.6 Dynamic Pressure — It is the pressure effect of air mass movement called the blast wind.

2.7 Equivalent Bare Charge — It is the weight of a bare high explosive charge geometrically similar to any given cased charge, which produces the same blast field as the given cased charge.

2.8 Ground Zero — It is the point on the earth surface vertically below the explosion.

2.9 Impulse — Impulse per unit of projected area is the pressure-time product given by the area under the pressure-time curve considered for the positive phase only unless otherwise specified.

2.10 Mach Number — It is the ratio of the speed of the shock front propagation to the speed of sound in standard atmosphere at sea level.

2.11 Overpressure — It is the rise in pressure above atmospheric pressure due to the shock wave from an air blast.

2.12 Reflected Overpressure — It is 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.

2.13 Shock Wave Front — It is 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.

2.14 Side-on Overpressure — It is the overpressure if it is not reflected by any surface.

2.15 Transit Time — It is the time required for the shock front to travel across the structure or its element under consideration.

2.16 Yield — It is a measure of the size of the explosion expressed in equivalent weight of reference explosive.

3. NOTATIONS

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

a	Velocity of sound in air
B	Span or width of structure across the direction of shock wave propagation
C_d	Drag coefficient
E	Modulus of elasticity of material
H	Height of the structure
I	Moment of inertia of member
j	Number of concentrated load points
K_a	Coefficient of earth pressure
K_{LM}	Load mass factor
K_L	Load factor
K_M	Mass factor
k	Ratio of resistance required to peak overpressure
k_1 & k_2	Values of k for μ and time ratios $\frac{t_{d1}}{T}$ and $\frac{t_{d2}}{T}$
k_E	Effective stiffness of equivalent single spring-mass system
L	Length of structure in the direction of motion of blast wave (*see* Fig. 2, 7 and 9)
M	Mach number for incident shock front
M_t	Total actual mass
M_r	Concentrated mass at point r
m	Distributed mass intensity per unit length
n	Number of concentrated masses
P_t	Total dynamic load (at any instant of time)

P_r	Concentrated dynamic force at point r
$p_{(x)}$	Intensity of distributed dynamic load per unit length
p_a	Ambient atmospheric pressure
p_{ro}	Peak reflected overpressure
p_s	Side-on overpressure
p_{so}	Peak side-on overpressure
q	Dynamic pressure
q_o	Peak dynamic pressure
R_m	Resistance required by a structural member
S	$\frac{1}{2}$ B or H whichever is less
T	Effective time period of structural member
t_c	Clearance time
t_d	Duration of the equivalent triangular pulse
t_o	Time for positive phase of side-on overpressure
t_q	Duration of the equivalent triangular pulse for drag loading only
t_t	Transit time
U	Shock front velocity
W	Yield of explosion
Y_e	Deflection/deformation at yield point of idealised resistance deflection curve
Y_m	Maximum deflection/deformation permitted in the design of the structure
Z	Tension steel ratio
Z'	Compression steel ratio
α	Decay parameter
ϕ_r	Deflection at point r of an assumed deflected shape for concentrated loads
$\phi_{(x)}$	Deflection at point x of an assumed deflected shape for distributed loads
μ	Ductility ratio $= \dfrac{Y_m}{Y_e}$

4. GENERAL CHARACTERISTICS OF BLAST AND EFFECTS ON STRUCTURES

4.1 The Source — The conventional chemical charge is considered spherical. The shock front at the ground surface from a contact burst is approximately vertical. The effective yield of a contact burst is almost double of an equal explosion high in the air. This condition is assumed to give most serious effects.

4.2 Shock Wave — As a result of explosion, a shock wave is generated in the air which moves outward in all directions from the point of burst with high speed causing time-dependent pressure and suction effects at all points in its way. The shock wave consists of an initial positive pressure phase followed by a negative (suction) phase at any point as shown in Fig. 1. The shock wave is accompanied by blast wind causing dynamic pressures due to drag effects on any obstruction coming in its way. Due to diffraction of the wave at an obstructing surface reflected pressure is caused instantaneously which clears in a time depending on the extent of obstructing surface.

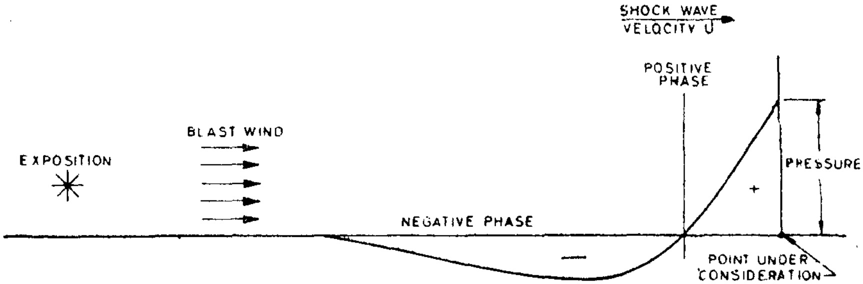


FIG. 1 SHOCK WAVE PRODUCED BY BLAST

4.3 Pressures and Duration — At any surface encountered by the shock wave, the pressure rises almost instantaneously to the peak values of side-on overpressure and the dynamic pressure or their reflected pressure. The peak values depend upon the size of explosion, the distance of the surface from the source, and other factors like ambient pressure and temperature in air.

4.3.1 The incident blast wave characteristics are described by the peak initial overpressure p_{so}, the overpressure p_s *versus* time t curve; the maximum dynamic pressure q_o, the dynamic pressure q *versus* time t curve and the duration of positive phase t_o.

4.3.2 The peak positive intensity quickly drops down to zero; the total duration of the positive phase being a few milliseconds. The maximum negative overpressure is much smaller than the peak positive overpressure, its limiting value being one atmosphere. But the negative phase duration is 2 to 5 times as long as that of the positive phase.

4.4 General Principles — As in the case of normal loads, members subjected to blast pressures resist the applied force by means of internal stresses developed in them. However, the effective load due to blast, for which resistance should be developed in the member, depends upon the dynamic properties of the member itself. Longer the natural time period of the member smaller is the effective load for design.

4.4.1 The duration of positive phase of blast is generally small as compared with the natural period of the structural elements, hence may be treated as an impulse problem.

4.4.2 Considering the probability of occurrence of blast loading to be small, structures may be permitted to deform in the plastic range for economical design. Permitting plastic deformations increases the energy absorption and has the further advantage that the effective time period of the structural element is elongated, thereby reducing the effective load for its design.

4.4.3 Most severe blast loading on any face of a structure is produced when the structure is oriented with the face normal to the direction of propagation of the shock front. However, for lack of known orientation of future explosion, every face of the structure shall be considered as a front face. When the blast field surrounds the structure, the difference of pressures on front and rear faces tends to tilt and overturn the structure as a whole.

5. BLAST FORCE

5.1 Maximum Values for Reference Explosion — The maximum values of the positive side-on overpressure p_{so}, reflected overpressure p_{ro} and dynamic pressure q_o, as caused by the explosion of one tonne explosive at various distances from the point of explosion, are given in Table 1. The duration of the positive phase of the blast t_o and the equivalent time duration of positive phase t_d are also given in Table 1.

5.2 Decay of Pressure with Time — The pressure varies with time according to the following relations:

$$p_s = p_{so}\left(1 - \frac{t}{t_o}\right) e - \alpha \frac{t}{t_o}$$

$$q = q_o\left(1 - \frac{t}{t_o}\right)^2 e - 2\alpha \frac{t}{t_o}$$

NOTE 1 — As will be noted from the above equations the dynamic pressure q decays much faster with time than the side-on overpressure p_s.

NOTE 2 — Since the use of these relations in design problem would involve tedious calculations, the pressure time relations in the positive phase are idealised by using a straight line starting with the maximum pressure value but terminating at a time t_d or t_q such that the impulse value remains the same. This criteria has been adopted in this standard.

TABLE 1 BLAST PARAMETERS FROM GROUND BURST OF 1 TONNE EXPLOSIVE

(*Clause* 5.1)

DISTANCE, m x	PEAK SIDE ON OVER-PRESSURE RATIO p_{so}/p_a	MACH NO. M	POSITIVE PHASE DURATION t_o, milli-secs	DURATION OF EQUIVALENT TRIANGULAR PULSE t_d, milli-secs	DYNAMIC PRESSURE RATIO q_o/p_a	PEAK REFLECTED OVERPRESSURE RATIO p_{ro}/p_a
(1)	(2)	(3)	(4)	(5)	(6)	(7)
15	8·00	2·80	9·50	5·39	10·667	41·60
18	5·00	2·30	11·00	7·18	5·208	22·50
21	3·30	1·96	16·38	9·33	2·643	12·94
24	2·40	1·75	18·65	11·22	1·532	8·48
27	1·80	1·60	20·92	13·30	0·920	5·81
30	1·40	1·48	22·93	15·39	0·583	4·20
33	1·20	1·42	24·95	16·31	0·439	3·45
36	1·00	1·36	26·71	17·94	0·312	2·75
39	0·86	1·32	28·22	19·20	0·235	2·28
42	0·76	1·28	29·74	20·20	0·186	1·97
45	0·66	1·25	31·25	21·60	0·142	1·66
48	0·59	1·23	32·26	22·70	0·115	1·46
51	0·53	1·20	33·52	23·70	0·093	1·28
54	0·48	1·19	34·52	24·70	0·077	1·14
57	0·43	1·17	35·53	26·40	0·062	1·01
60	0·40	1·16	36·29	26·60	0·054	0·93
63	0·37	1·15	37·30	27·80	0·046	0·85
66	0·34	1·14	38·05	28·76	0·039	0·77
69	0·32	1·13	38·81	29·25	0·035	0·72
72	0·30	1·12	39·56	29·87	0·031	0·67
75	0·28	1·11	40·32	30·71	0·027	0·62
78	0·26	1·104	40·82	31·85	0·023	0·58
81	0·25	1·100	41·58	31·92	0·022	0·55
84	0·24	1·098	42·34	32·00	0·020	0·53
87	0·23	1·095	42·84	32·26	0·018	0·50
90	0·22	1·086	43·60	33·39	0·016	0·47
93	0·20	1·082	44·35	34·70	0·014	0·43
96	0·19	1·077	45·46	35·37	0·013	0·41
99	0·18	1·072	45·61	36·22	0·012	0·40

NOTE 1 — The value of p_a the ambient air pressure may be taken as 1 kg/cm² at mean sea level.

NOTE 2 — One tonne of explosive referred to in this table is equivalent to $1·5 \times 10^9$ calories.

NOTE 3 — Velocity of sound in m/s may be taken ($331·5 + 0·607\ T$) where T is the ambient temperature in centigrade.

5.3 Scaling Laws — For any explosion other than the reference explosion, the peak pressures and time durations may be found from the peak values given in Table 1 by the cube root scaling laws as given below:

$$\text{Scaled distance } x = \frac{\text{Actual distance}}{W^{1/3}} \quad \ldots \quad \ldots \quad (1)$$

$$\text{Scaled time } t_o = \frac{\text{Actual time}}{W^{1/3}} \quad \ldots \quad \ldots \quad \ldots \quad (2)$$

where

W = yield of explosion in equivalent weight of the reference explosive measured in tonnes,

x = scaled distance for entering the Table 1 for reading peak values, and

t_o = scaled time read from Table 1 against scaled distance.

NOTE — Actual distance is measured from the ground zero to the point under consideration. Actual time is the time for actual explosion.

6. BLAST LOAD ON ABOVE GROUND STRUCTURES

6.1 Types of Structures — There are mainly the following two types of structures:

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_s and the dynamic pressures q caused by blast wind.

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.

6.2 Closed Rectangular Structures — (*See* Fig. 2).

6.2.1 *Front Face* — As the shock wave strikes the vertical face of a structure normal reflection occurs and the pressure on the front face instantaneously increases to the reflected overpressure p_{ro} given by the following equation:

$$p_{ro} = p_{so}\left(2 + \frac{6\,p_{so}}{p_{so} + 7\,p_a}\right) \quad \ldots \quad \ldots \quad \ldots \quad (3)$$

where

p_a = the ambient atmospheric pressure. Taking $p_a = 1$ kg/cm², the value of p_{ro} are given in Table 1.

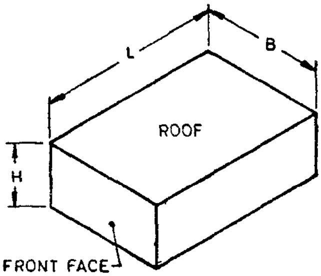


Fig. 2 Above Ground Rectangular Structure

6.2.1.1 The net pressure acting on the front face at any time t is the reflected overpressure p_r or ($p_s + C_d\, q$), whichever is greater:

where

C_d = drag coefficient given in Table 2, and

p_r = the reflected overpressure which drops from the peak value p_{ro} to overpressure ($p_s + C_d\, q$) in clearance time t_c given by:

$$t_c = \frac{3S}{U} \text{ or } t_d \text{ whichever is less} \qquad \ldots \qquad (4)$$

where

S = H or $B/2$ whichever is less (*see* Fig. 2)

U = shock front velocity = $M.a$

where

a = velocity of sound in air which may be taken as 344 m/s at mean sea level at 20°C, and

M = Mach number of the incident pulse given by $\sqrt{1 + 6\, p_{so}/7\, p_a}$. The values of M for various conditions are also tabulated in Table 1.

6.2.1.2 The net average loading on the front face ($B \times H$) as a function of time is shown in Fig. 3A or 3B depending on whether t_c is smaller than or equal to t_d. The pressures p_{ro}, p_{so} and q_o and time t_d are for actual explosion determined according to the scaling laws given in **5.3**.

6.2.2 *Rear Face* — Using the pressures for the actual explosion, the average loading on the rear face ($B \times H$ in Fig. 2) is taken as shown in Fig. 4 where the time has been reckoned 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 back face.

TABLE 2 DRAG COEFFICIENT C_d

(*Clause* 6.2.1.1)

SL No.	SHAPE OF ELEMENT	DRAG COEFFICIENT C_d	REMARKS
(1)	(2)	(3)	(4)
For Closed Rectangular Structures			
i)	Front vertical face	1·0	For above ground structures
ii)	Roof, rear and side faces for		
	q_o = 0 to 1·8 kg/cm²	−0·4	
	q_o = 1·8 to 3·5 kg/cm²	−0·3	
	q_o = 3·5 to 9·0 kg/cm²	−0·2	
iii)	Front face sloping		For semi-buried structures
	4 to 1	Zero	
	1½ to 1	0·4	
For Open, Drag Type Structures			
iv)	Sphere	0·1	—
v)	Cylinder	1·2	This covers steel tubes used as columns, truss members, etc
vi)	Structural shapes	2·0	This covers flats, angles, tees, I sections, etc
vii)	Rectangular projection	1·3	This covers beam projections below or above slabs, cantilever walls standing freely above ground, etc

6.2.3 *Roof and Side Walls* — As for rear face in **6.2.2**, the average pressure *versus* time curve for roof and side walls is given in Fig. 5A, when t_d is greater than the transit time $t_t = \frac{L}{U}$. 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 ($p_{so} + C_d \cdot q_o$) and time t_d as shown in Fig. 5B.

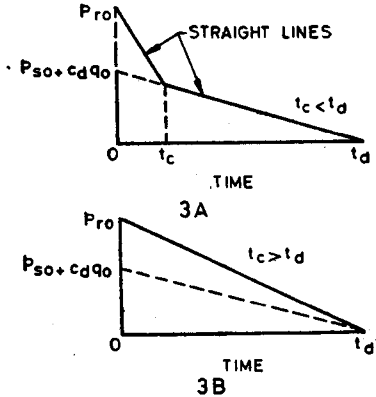


FIG. 3 PRESSURE *versus* TIME FOR FRONT FACE

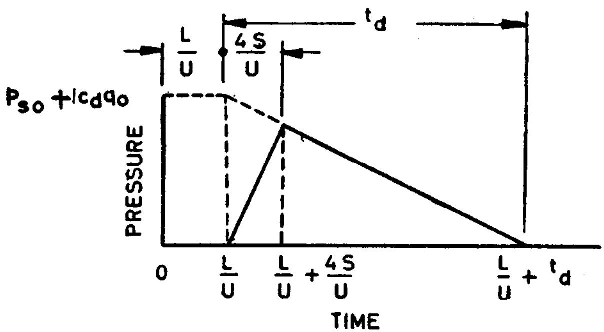


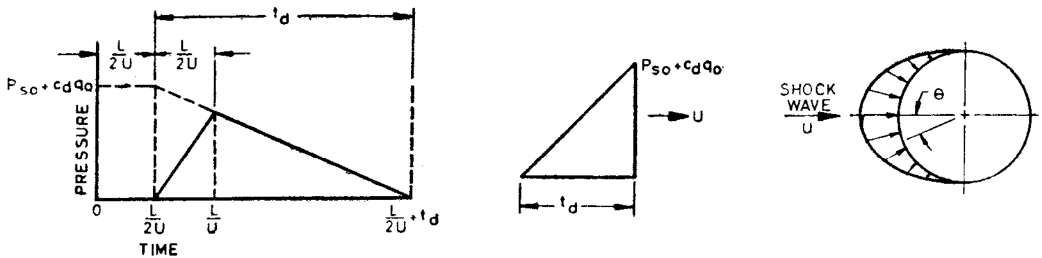
FIG. 4 PRESSURE *versus* TIME FOR REAR FACE

6.2.4 *Overturning of Structure* — The net average 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.

6.2.5 An example of calculation on pressure-time curves on a rectangular above-ground building is given in Appendix A.

6.3 Structures with Openings

6.3.1 *Open or Drag Type Structures* — The net translational pressure on the obstructing areas of elements may be taken as shown in



5A Average Pressure Diagram $t_d > t_t$

5B Moving Pressure Pulse for $t_t > t_d$

5C Transverse Variation of Pressure on Dome

FIG. 5 PRESSURE *versus* TIME FOR ROOF AND SIDE WALLS

Fig. 6 where C_d is the drag coefficient depending upon the shape of the structural element given in Table 2 and t_q is given as follows:

$$t_q = \frac{1}{2}\, t_o \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \qquad (5)$$

NOTE — The pressures p_{so} and time t_o are for actual explosion.

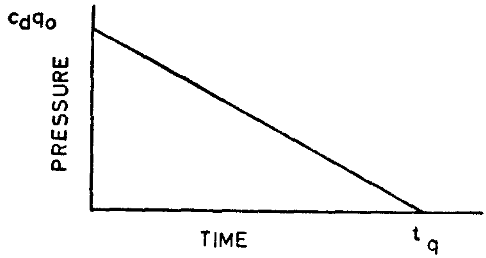


FIG. 6 PRESSURE *versus* TIME FOR OPEN STRUCTURES

6.3.2 *Partly Open Structures* — When the area of openings is more than 50 percent of the area of walls, the structure may be considered drag type (*see* **6.3.1**). When the area of openings is less than 5 percent of the area of walls, the structure may be considered closed type, (*see* **6.2**). For intermediate conditions, direct interpolation may be made between the two conditions of both maximum pressures and time duration.

6.4 Closed Cylindrical Arch-Shape Structures

6.4.1 *Gable Ends* — The loading may be taken same as for front and rear faces of above ground rectangular structure.

6.4.2 *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 can not be assumed. Therefore, the loading on curved surface may be taken as a moving triangular pulse as shown in Fig. 5B.

6.5 Closed Dome Structures — The loading on a domical structure may be taken as a moving triangular pressure pulse as shown in Fig. 5B. The variation of pressure transverse to the direction of propagation of the pulse may be considered symmetrical varying according to Cos θ where the angle θ is measured from longitudinal vertical section of the dome (*see* Fig. 5C).

7. BLAST LOAD ON BELOW-GROUND STRUCTURES

7.1 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 structure is subjected only to the general overpressure p_{so}, the reflected and dynamic pressures being neglected. The semi-buried structure is subjected to partial dynamic pressures besides the general overpressure. Both are acted upon by air-induced ground shock also.

7.2 Buried Rectangular Structures

7.2.1 A rectangular structure is considered buried if it satisfies the condition shown in Fig. 7.

7.2.2 The pressure *versus* time diagrams for the various faces of the structure are shown in Fig. 8. The values of the coefficient of earth pressure K_a appearing in Fig. 8 are given in Table 3.

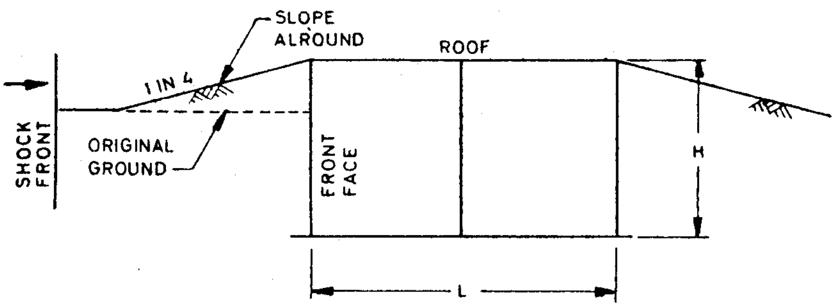


FIG. 7 BURIED RECTANGULAR STRUCTURE

7.3 Buried Arch or Dome Structures

7.3.1 An arch or dome structure is considered buried if it satisfies the condition shown in Fig. 9.

7.3.2 The pressure *versus* time relations at crown and wall are the same as that on the roof and walls of a buried rectangular structure respectively. The pressure on the slope of the arch or dome may gradually be reduced from crown towards the springings such that the pressure at springing becomes equal to that on the wall. If the semi-central angle is less than 45°, the arch or dome may be designed for the same pressure as the roof of a buried rectangular structure.

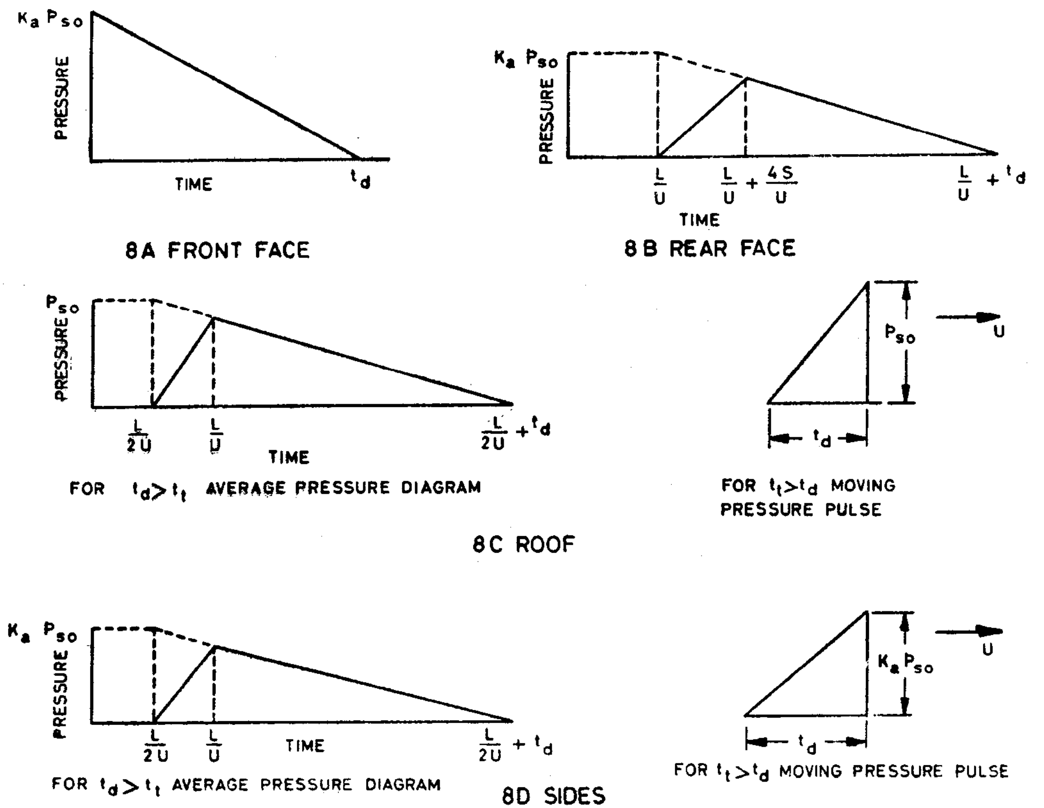


FIG. 8 PRESSURE *versus* TIME DIAGRAMS FOR VARIOUS FACES OF BURIED STRUCTURE

TABLE 3 COEFFICIENT OF EARTH PRESSURE K_a

(*Clause* 7.2.2)

Sl No.	Type of Soil	Coefficient K_a
(1)	(2)	(3)
i)	Cohesionless soil, dry or damp	$\frac{1}{4}$
ii)	Cohesive soil:	
	a) Stiff unsaturated	$\frac{1}{3}$
	b) Medium unsaturated	$\frac{1}{2}$
	c) Soft unsaturated	$\frac{3}{4}$
iii)	Fully saturated soil	1

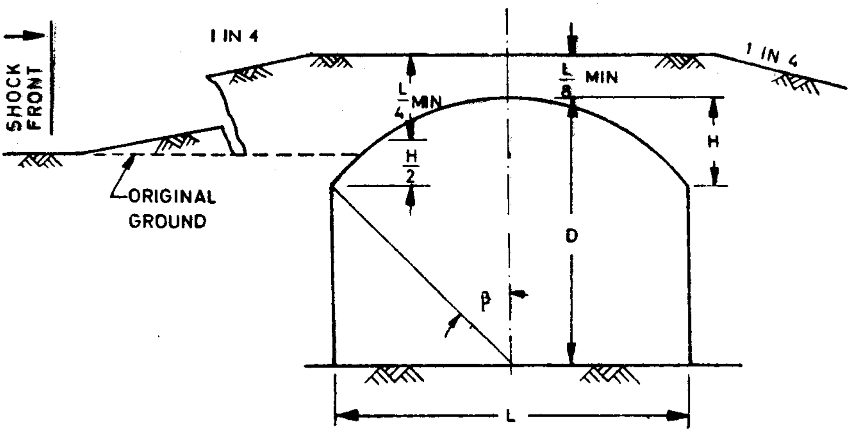


Fig. 9 Buried Arch or Dome Structure

7.4 Semi-Buried Structures

7.4.1 Semi-buried structures are those which do not have the minimum earth cover or the slopes are steeper than those specified in Fig. 7 and 9. These are subjected to dynamic pressures besides the general overpressures.

7.4.2 A minimum slope of $1\frac{1}{2}$ to 1 may be used for the earth cover. The reflected pressure effect may be neglected and dynamic pressure may be considered on the front face with drag coefficient 0·4. For flatter slope the drag coefficient may be reduced making it zero when the slope becomes 4 to 1.

7.4.3 The loading on the earth berm surface may be computed as for the above ground structures.

8. RESPONSE OF STRUCTURAL ELEMENTS

8.1 Significant Factors

8.1.1 The significant factors, on which the response of a structural element subject to blast forces depends, are the pressure *versus* time diagram acting on the element, the effective time period of the element, the resistance *versus* deflection diagram of the element, and the maximum permissible deflection.

8.1.2 When the ratio of time duration t_d or t_o 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.

8.1.3 In most cases, the pressure-time diagram can be idealised without loss of accuracy as a triangular pulse (Fig. 10A) having the same maximum pressure as in the original diagram and the time so adjusted that the area under the curve remains unchanged. However, where the load acting on the front face has the shape of the diagram of Fig. 10B, it may be treated as such (*see* also **8.2.4**).

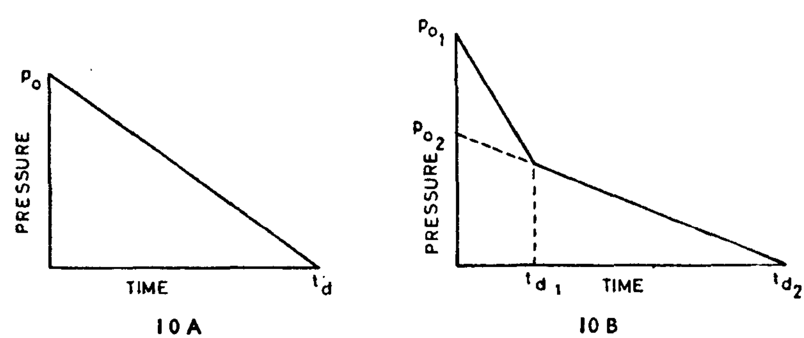


FIG. 10 IDEALIZED PRESSURE TIME DIAGRAMS

8.1.4 For simplicity the resistance *versus* deflection diagram of an element is idealized as elasto-plastic as shown in Fig. 11 by keeping the area under the actual and idealized curves about the same up to the maximum permissible deflection.

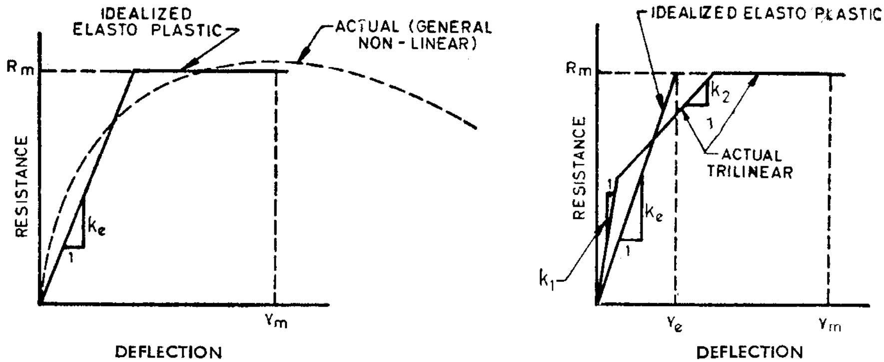


FIG. 11 IDEALIZED RESISTANCE DEFLECTION DIAGRAM

8.1.5 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, greater the permissible deflection, lesser will be the maximum resistance required in the member.

8.2 Elastic and Elasto-Plastic Response

8.2.1 Based on the triangular pressure-time curve shown in Fig. 10 A and elasto-plastic resistance-deflection curve shown in Fig. 11, the ratio of resistance R_m required to the peak overpressure p_o is given in Fig. 12 for various values of t_d/T and μ.

8.2.2 Figure 12 also shows the time t_m at which maximum deflection occurs, as a ratio of time T for various values of t_d/T and μ.

8.2.3 When the time ratio t_d/T is less than 0·1, the ratio k may be computed from the following equation:

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

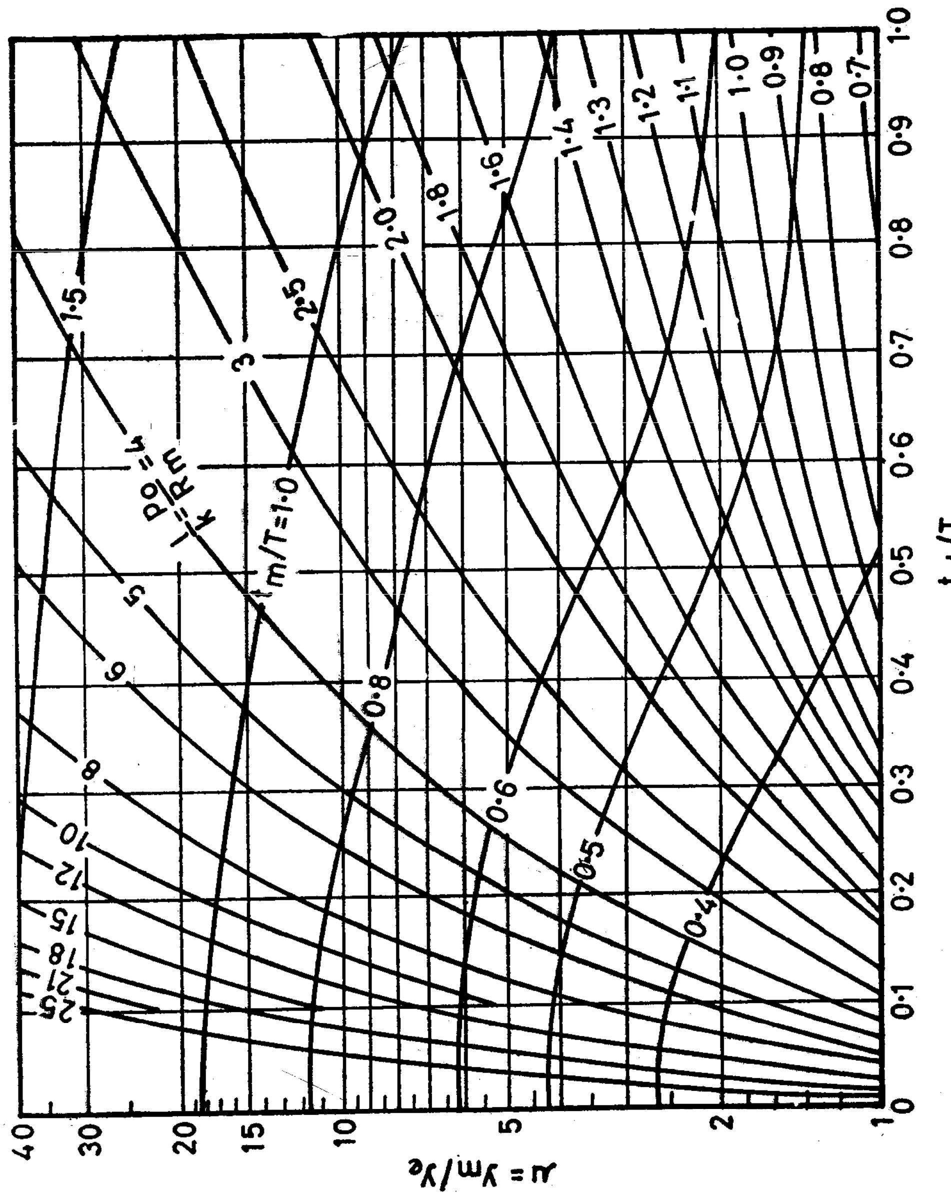
8.2.4 When the pressure-time diagram is given by Fig. 10 B, the following equation shall be satisfied:

$$\frac{P_{o1}}{R_m} k_1 + \frac{P_{o2}}{R_m} k_2 = 1$$

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.


t_d/T
μ = y_m/y_e
1/k = P_o/R_m = 4
t_m/T = 1·0

8.3 Elastic Rebound — After attaining the maximum response, the structure is found to oscillate and may have an elastic rebound equal and opposite to the maximum deflection. Therefore, members shall be designed to have the same strength for the reversal of the effective design load.

9. TIME PERIOD OF STRUCTURAL MEMBERS

9.1 The structural element or frame-work 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.

9.2 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:

$$K_{LM} = K_M / K_L$$

$$K_M = \frac{1}{M_t} \left\{ \sum_{r=1}^{n} M_r \, \phi_r^2 + \int m \, \phi^2_{(x)} \, dx \right\}$$

$$K_L = \frac{1}{P_t} \left\{ \sum_{r=1}^{j} p_r \phi_r + \int p_{(x)} \phi_{(x)} \, dx \right\}$$

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.

NOTE 1 — Values of the factors k_E and K_{LM} for certain structural members are given in Tables 4 to 6.

NOTE 2 — 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 has to be normalized such that $\phi = 1$ at the point with respect to which the effective stiffness k_E is defined.

9.2.1 Two typical examples explaining the evaluation of K_{LM} are given in Appendix B.

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

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

9.4 For elastic analysis (ductility ratio $\mu \leqslant 1{\cdot}0$) of structures, the effective stiffness k_E and load mass factor K_{LM} are to be used as given in Tables 4 to 6. For elasto-plastic analysis k_E is to be used as given in Tables 4 to 6 but value of K_{LM} may be chosen in between the elastic and plastic cases depending upon the ductility factor.

9.4.1 For elasto-plastic design of fixed slabs, the modified value of k_E is to be worked out in accordance with Fig. 11B using the stiffness values of slab in elastic and elasto-plastic cases as given in Table 6 and K_{LM} is to be suitably chosen depending upon the ductility factor.

9.5 For calculating moment of inertia I of reinforced concrete sections, the effective transformed area may be used. The value of modular ratio shall be taken the same as in static design for calculating EI.

10. DYNAMIC STRENGTH OF MATERIALS, DESIGN STRESSES

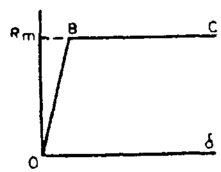
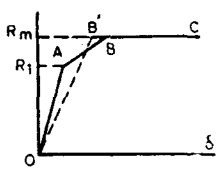
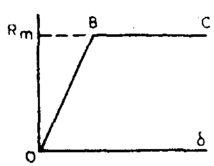
10.1 General — Plastic deformation of the structural elements should be permitted except where the functioning of the structure would be adversely affected by their permanent displacements.

10.1.1 Under rapid rates of straining as associated with blast loading, materials develop higher strengths than in statically loaded members. Under such conditions the dynamic strength may be taken greater than the minimum specified static strength as indicated in **10.2** to **10.5**.

TABLE 4 TRANSFORMATION FACTORS FOR BEAMS AND ONE-WAY SLABS*

(*Note* 1 of *Clause* 9.2)

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
k_E = effective spring constant

Resistance curve, End conditions of Beams or One-Way Slabs	Dynamic Loading	Strain Range	Load-Mass Factor: Concentrated Mass	Load-Mass Factor: Uniform Mass	Maximum Resistance R_m	Effective Spring Constant k_E	Dynamic Reaction
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
 Simply Supported	Uniform $P_t = pL$	Elastic	—	0·78	$8M_p/L$	$384EI/5L^3$	$0{\cdot}39R_m + 0{\cdot}11P_t$
		Plastic	—	0·66	$8M_p/L$	—	$0{\cdot}38R_m + 0{\cdot}12P_t$
	Concentrated at mid-span $P_t = P$	Elastic	1·0	0·49	$4M_p/L$	$48EI/L^3$	$0{\cdot}78R_m - 0{\cdot}28P_t$
		Plastic	1·0	0·33	$4M_p/L$	—	$0{\cdot}75R_m - 0{\cdot}25P_t$
	Concentrated at third points $P/2$ each $P_t = P$	Elastic	0·87	0·60	$6M_p/L$	$56{\cdot}4EI/L^3$	$0{\cdot}62R_m - 0{\cdot}12P_t$
		Plastic	1·0	0·56	$6M_p/L$	—	$0{\cdot}52R_m - 0{\cdot}02P_t$
 Fixed at both ends	Uniform $P_t = pL$	Elastic (OA)	—	0·77	$12M_{ps}/L$	$384EI/L^3$	$0{\cdot}36R_1 + 0{\cdot}14P_t$
		Elastic (OB′)	—	0·78	$(8/L)(M_{ps} + M_{pm})$	$307EI/L^3$	$0{\cdot}39R_m + 0{\cdot}11P_t$
		Plastic	—	0·66	$(8/L)(M_{ps} + M_{pm})$	—	$0{\cdot}38R_m + 0{\cdot}12P_t$
 	Concentrated at mid-span $P_t = P$	Elastic (OB)	1·0	0·37	$(4/L)(M_{ps} + M_{pm})$	$192EI/L^3$	$0{\cdot}71R_m - 0{\cdot}21P_t$
		Plastic	1·0	0·33	$(4/L)(M_{ps} + M_{pm})$	—	$0{\cdot}75R_m - 0{\cdot}25P_t$
Fixed at one end and simply supported at the other	Uniform $P_t = pL$	Elastic (OA)	—	0·78	$(8/L)M_{ps}$	$185EI/L^3$	$(V_1 = 0{\cdot}26R_1 + 0{\cdot}12P_t)$ $(V_2 = 0{\cdot}43R_1 + 0{\cdot}19P_t)$
		Elastic (OB′)	—	0·78	$(4/L)(M_{ps} + 2M_{pm})$	$160EI/L^3$	$0{\cdot}39R_m + 0{\cdot}11P_t \pm M_{ps}/L$
		Plastic	—	0·66	$(4/L)(M_{ps} + 2M_{pm})$	—	$0{\cdot}38R_m + 0{\cdot}12P_t \pm M_{ps}/L$
	Concentrated at mid-span $P_t = P$	Elastic (OA)	1·0	0·43	$(16/3L)M_{ps}$	$107EI/L^3$	$(V_1 = 0{\cdot}54R_1 + 0{\cdot}14P_t)$ $(V_2 = 0{\cdot}25R_1 + 0{\cdot}07P_t)$
		Elastic (OB′)	1·0	0·49	$(2/L)(M_{ps} + 2M_{pm})$	$106EI/L^3$	$0{\cdot}78R_m - 0{\cdot}28P_t \pm M_{ps}/L$
		Plastic	1·0	0·33	$(2/L)(M_{ps} + 2M_{pm})$	—	$0{\cdot}75R_m - 0{\cdot}25P_t \pm M_{ps}/L$

*Taken from 'Design of structures to resist the effect of atomic weapons', U.S. Army Corps of Engineers Manual TM5-856-3, 1957.

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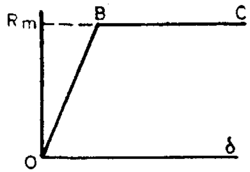
TABLE 5 TRANSFORMATION FACTORS FOR TWO-WAY SLABS — FOUR SIDES, SIMPLE SUPPORTS UNIFORM LOAD

(*Note* 1 of *clause* 9.2)



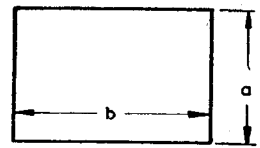
M_{pfa} = Total positive ultimate moment capacity along mid-span section parallel to short edge

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

V_B = Total dynamic reaction along a long edge



Strain Range	a/b	Load Mass Factor K_{LM}	Maximum Resistance R_m	Spring Constant k_E	Dynamic Reactions V_A	V_B
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Elastic (*OB*)	1·0	0·68	$12 (M_{pfa} + M_{pfb})/a$	$252EI/a^2$	$0{\cdot}07P_t + 0{\cdot}18R_m$	$0{\cdot}07P_t + 0{\cdot}18R_m$
	0·9	0·70	$(12M_{pfa} + 11M_{pfb})/a$	$230EI/a^2$	$0{\cdot}06P_t + 0{\cdot}16R_m$	$0{\cdot}08P_t + 0{\cdot}20R_m$
	0·8	0·71	$(12M_{pfa} + 10{\cdot}3M_{pfb})/a$	$212EI/a^2$	$0{\cdot}06P_t + 0{\cdot}14R_m$	$0{\cdot}08P_t + 0{\cdot}22R_m$
	0·7	0·73	$(12M_{pfa} + 9{\cdot}8M_{pfb})/a$	$201EI/a^2$	$0{\cdot}05P_t + 0{\cdot}13R_m$	$0{\cdot}08P_t + 0{\cdot}24R_m$
	0·6	0·74	$(12M_{pfa} + 9{\cdot}3M_{pfb})/a$	$197EI/a^2$	$0{\cdot}04P_t + 0{\cdot}11R_m$	$0{\cdot}09P_t + 0{\cdot}26R_m$
	0·5	0·75	$(12M_{pfa} + 9{\cdot}0M_{pfb})/a$	$201EI/a^2$	$0{\cdot}04P_t + 0{\cdot}09R_m$	$0{\cdot}09P_t + 0{\cdot}28R_m$
Plastic (*BC*)	1·0	0·51	$12 (M_{pfa} + M_{pfb})/a$	—	$0{\cdot}09P_t + 0{\cdot}16R_m$	$0{\cdot}09P_t + 0{\cdot}16R_m$
	0·9	0·51	$(12M_{pfa} + 11M_{pfb})/a$	—	$0{\cdot}08P_t + 0{\cdot}15R_m$	$0{\cdot}09P_t + 0{\cdot}18R_m$
	0·8	0·54	$(12M_{pfa} + 10{\cdot}3M_{pfb})/a$	—	$0{\cdot}07P_t + 0{\cdot}13R_m$	$0{\cdot}10P_t + 0{\cdot}20R_m$
	0·7	0·58	$(12M_{pfa} + 9{\cdot}8M_{pfb})/a$	—	$0{\cdot}06P_t + 0{\cdot}12R_m$	$0{\cdot}10P_t + 0{\cdot}22R_m$
	0·6	0·58	$(12M_{pfa} + 9{\cdot}3M_{pfb})/a$	—	$0{\cdot}05P_t + 0{\cdot}10R_m$	$0{\cdot}10P_t + 0{\cdot}25R_m$
	0·5	0·59	$(12M_{pfa} + 9{\cdot}0M_{pfb})/a$	—	$0{\cdot}04P_t + 0{\cdot}08R_m$	$0{\cdot}11P_t + 0{\cdot}27R_m$

*Taken from 'Design of structures to resist the effects of atomic weapons', U. S. Army Corps of Engineers Manual TM 5-856-3, 1957.

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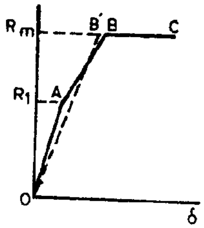
TABLE 6 TRANSFORMATION FACTORS FOR TWO-WAY SLABS—FIXED FOUR SIDES, UNIFORM LOAD*

(*Note* 1 *of Clause* 9.2)



M°_{psb} = Negative ultimate moment capacity per *unit* width at centre of long edge

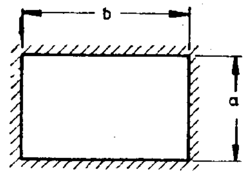
M_{psa} = Total negative ultimate moment capacity along a short edge support

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

V_B = Total dynamic reaction along a long edge



Strain Range	a/b	Load Mass Factor K_{LM}	Maximum Resistance R_m	Spring Constant k_E	Dynamic Reactions	
					V_A	V_B
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Elastic (*OA*)	1·0	0·63	$29{\cdot}2M^{\circ}_{psb}$	$810EI/a^2$	$0{\cdot}10P_t + 0{\cdot}15Rl$	$0{\cdot}10P_t + 0{\cdot}15Rl$
	0·9	0·68	$27{\cdot}4M^{\circ}_{psb}$	$742EI/a^2$	$0{\cdot}09P_t + 0{\cdot}14Rl$	$0{\cdot}10P_t + 0{\cdot}17Rl$
	0·8	0·69	$26{\cdot}4M^{\circ}_{psb}$	$705EI/a^2$	$0{\cdot}08P_t + 0{\cdot}12Rl$	$0{\cdot}11P_t + 0{\cdot}19Rl$
	0·7	0·71	$26{\cdot}2M^{\circ}_{psb}$	$692EI/a^2$	$0{\cdot}07P_t + 0{\cdot}11Rl$	$0{\cdot}11P_t + 0{\cdot}21Rl$
	0·6	0·71	$27{\cdot}3M^{\circ}_{psb}$	$724EI/a^2$	$0{\cdot}06P_t + 0{\cdot}09Rl$	$0{\cdot}12P_t + 0{\cdot}23Rl$
	0·5	0·72	$30{\cdot}2M^{\circ}_{psb}$	$806EI/a^2$	$0{\cdot}05P_t + 0{\cdot}08Rl$	$0{\cdot}12P_t + 0{\cdot}25Rl$
Elasto-Plastic (*OB′*)	1·0	0·67	$(1/a)\,[12(M_{pfa}+M_{psa})+12(M_{pfb}+M_{psb})]$	$252EI/a^2$	$0{\cdot}07P_t + 0{\cdot}18R_m$	$0{\cdot}07P_t + 0{\cdot}18R_m$
	0·9	0·70	$(1/a)\,[12(M_{pfa}+M_{psa})+11(M_{pfb}+M_{psb})]$	$230EI/a^2$	$0{\cdot}06P_t + 0{\cdot}16R_m$	$0{\cdot}08P_t + 0{\cdot}20R_m$
	0·8	0·71	$(1/a)\,[12(M_{pfa}+M_{psa})+10{\cdot}3(M_{pfb}+M_{psb})]$	$212EI/a^2$	$0{\cdot}06P_t + 0{\cdot}14R_m$	$0{\cdot}08P_t + 0{\cdot}22R_m$
	0·7	0·73	$(1/a)\,[12(M_{pfa}+M_{psa})+9{\cdot}8(M_{pfb}+M_{psb})]$	$201EI/a^2$	$0{\cdot}05P_t + 0{\cdot}13R_m$	$0{\cdot}08P_t + 0{\cdot}24R_m$
	0·6	0·74	$(1/a)\,[12(M_{pfa}+M_{psa})+9{\cdot}3(M_{pfb}+M_{psb})]$	$197EI/a^2$	$0{\cdot}04P_t + 0{\cdot}11R_m$	$0{\cdot}09P_t + 0{\cdot}26R_m$
	0·5	0·75	$(1/a)\,[12(M_{pfa}+M_{psa})+9{\cdot}0(M_{pfb}+M_{psb})]$	$201EI/a^2$	$0{\cdot}04P_t + 0{\cdot}09R_m$	$0{\cdot}09P_t + 0{\cdot}28R_m$
Fully Plastic (*BC*)	1·0	0·51	$(1/a)\,[12(M_{pfa}+M_{psa})+12(M_{pfb}+M_{psb})]$	—	$0{\cdot}09P_t + 0{\cdot}16R_m$	$0{\cdot}09P_t + 0{\cdot}16R_m$
	0·9	0·51	$(1/a)\,[12(M_{pfa}+M_{psa})+11(M_{pfb}+M_{psb})]$	—	$0{\cdot}08P_t + 0{\cdot}15R_m$	$0{\cdot}09P_t + 0{\cdot}18R_m$
	0·8	0·54	$(1/a)\,[12(M_{pfa}+M_{psa})+10{\cdot}3(M_{pfb}+M_{psb})]$	—	$0{\cdot}07P_t + 0{\cdot}13R_m$	$0{\cdot}10P_t + 0{\cdot}20R_m$
	0·7	0·58	$(1/a)\,[12(M_{pfa}+M_{psa})+9{\cdot}8(M_{pfb}+M_{psb})]$	—	$0{\cdot}06P_t + 0{\cdot}12R_m$	$0{\cdot}10P_t + 0{\cdot}22R_m$
	0·6	0·58	$(1/a)\,[12(M_{pfa}+M_{psa})+9{\cdot}3(M_{pfb}+M_{psb})]$	—	$0{\cdot}05P_t + 0{\cdot}10R_m$	$0{\cdot}10P_t + 0{\cdot}25R_m$
	0·5	0·59	$(1/a)\,[12(M_{pfa}+M_{psa})+9{\cdot}0(M_{pfb}+M_{psb})]$	—	$0{\cdot}04P_t + 0{\cdot}08R_m$	$0{\cdot}11P_t + 0{\cdot}27R_m$

Range (*OA*) : Moment at centre of long edge just becomes plastic

Range (*OB′*) : Moment at supports and mid-span sections just become plastic

*Taken from ' Design of structures to resist the effects of atomic weapons ', U.S. Army Corps of Engineers Manual TM 5-856-3, 1957.

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10.2 Design Stresses for Structural Steel

10.2.1 The average dynamic yield stress of structural carbon, mild, weldable or rivet steels may be assumed to exceed the minimum specified static yield stress by 25 percent and that of high strength alloy steels by 10 percent.

10.2.2 For elastic design, the resistance R_m of various elements of a framework may be assumed to have just been reached. The dynamic yield stress of steel as given in **10.2.1** shall be used for calculating R_m.

10.2.3 For elasto-plastic design, the ductility ratio may be assumed as follows:

TYPE OF MEMBER	DUCTILITY RATIO, μ
Members of roof trusses	
Slenderness ratio (l/r) = 180	1·0
Slenderness ratio (l/r) $\leqslant$ 60	5·0
Members subjected to bending and direct stresses	
Minor damage	5·0
Moderate damage	10·0
Considerable damage	20·0

NOTE — For intermediate values of l/r, linear interpolation may be done.

10.3 Design Stress for Reinforced Concrete

10.3.1 The dynamic strength of material may be assumed as follows:

a) *Reinforcing Steel* — Dynamic yield stress 25 percent higher than the minimum specified static yield stress.

b) *Concrete* — The dynamic cube compression strength may be assumed to be 25 percent higher than the minimum static cube strength at 28 days.

10.3.1.1 For dynamic shear in reinforced concrete members, no increase over the static shear strength shall be permitted. For dynamic bond stress in reinforced concrete members, an increase of 25 percent may be permitted over the static strength.

10.3.2 In elastic design, for calculating resistance R_m the ultimate flexural strength on the basis of ultimate load theory shall be assumed to have just been reached. For ultimate strength calculations a reference may be made to IS : 456-1964*, and the dynamic strength of materials as given in **10.3.1** should be used.

*Code of practice for plain and reinforced concrete (*second revision*).

10.3.3 For elasto-plastic design of reinforced concrete members the ductility ratio may be assumed as follows, provided that the steel ratio remains less than balanced steel ratio for plastic design:

Members Subjected to Bending and Direct Stresses	Ductility Ratio, μ
Minor damage	$\frac{0{\cdot}04}{Z - Z'}$ but $\not> 5$
Moderate damage	$\frac{0{\cdot}07}{Z - Z'}$ but $\not> 10$
Considerable damage	$\frac{0{\cdot}1}{Z - Z'}$ but $\not> 15$

where

Z' = compression steel ratio, and

Z = tension steel ratio.

10.4 Design Stresses for Masonry or Plain Concrete

10.4.1 The dynamic flexural strength of plain brick and stone masonry may be assumed to be the same as the corresponding static strength. The compressive strength shall be taken 25 precent higher than the corresponding static strength.

10.4.2 For unreinforced brick work the ductility ratio may be limited to 1·5.

10.4.3 For reinforced brickwork, with not less than 0·05 percent steel on each face and not more than balanced percentage, the ductility factors as for reinforced concrete may be used.

10.5 Design Pressures on Foundation Material

10.5.1 The following allowable bearing pressures may be used for design under blast loading:

Rock	Static crushing strength
Granular soil	Static load per unit area for 4·0 cm settlement of the structure
Cohesive soil	$\frac{3}{4}$ of static failure load per unit area determined by quick undrained test.

10.5.2 In the absence of test data on soils as envisaged under **10.5.1**, the design bearing pressure may be taken equal to twice the allowable static bearing pressure.

NOTE — In working out the pressure under foundation, the direct loading of ground due to blast overpressures may be neglected.

10.5.3 For raft foundation, the raft area need not be more than the roof area.

11. LOAD COMBINATIONS FOR DESIGN

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

11.2 Live load on floors shall be considered as per IS : 875-1964* depending upon the class of building. No live load shall be considered on roof at the time of blast.

12. RECOMMENDED VALUES OF BLAST PRESSURES FOR DESIGN

12.1 The design parameters that is the yield of explosion and its distance from the structure will depend upon the importance of the structure and conditions prevailing in a particular time and should be considered by the designer in each specific case.

12.2 For general guidance the buildings may be designed for a bare charge of 100 kg at distances given in Table 7.

TABLE 7 BUILDING DESIGN FOR A CHARGE OF 100 kg

Building Category	Type	Distance, m
A	Residential buildings	40
B	Community buildings, such as schools, offices, cinemas, etc, and industrial buildings with continuous human occupancy	30
C	Buildings provided for accommodating essential service which would be of post bombing importance, such as hospitals, emergency relief stores, power stations, water works, communication centres, etc.	20

12.3 General recommendations for planning blast resistant buildings are given in Appendix C.

*Code of practice for structural safety of buildings: Loading standards (*revised*).

APPENDIX A

(*Clause* 6.2.5)

AN EXAMPLE OF CALCULATION OF PRESSURE-TIME CURVES ON A RECTANGULAR ABOVE GROUND BUILDING

A-1. EXAMPLE

A-1.1 Blast parameters due to the detonation of a 0·1 tonne explosive are evaluated on an above ground rectangular structure, 3 m high, 10 m wide and 8 m long, situated at 30 m from ground zero.

a) *Characteristics of the Blast*

$$\text{Scaled distance } x = \frac{30}{(0{\cdot}1)^{\frac{1}{3}}} = 64{\cdot}65 \text{ m}$$

From Table 1 assuming $p_a = 1{\cdot}00$ kg/cm² and linearly interpolating between 63 m and 66 m for the scaled distance 64·65 m, the pressures are directly obtained:

$p_{so} = 0{\cdot}35$ kg/cm²

$p_{ro} = 0{\cdot}81$ kg/cm²

$q_o = 0{\cdot}042$ kg/cm²

The scaled times t_o and t_d obtained from Table 1 for scaled distance 64·65 m are multiplied by $(0{\cdot}1)^{\frac{1}{3}}$ to get the values of the respective quantities for the actual explosion of 0·1 tonne charge.

$$t_o = 37{\cdot}71\ (0{\cdot}1)^{\frac{1}{3}} = 17{\cdot}5 \text{ milliseconds}$$

$$t_d = 28{\cdot}32\ (0{\cdot}1)^{\frac{1}{3}} = 13{\cdot}15 \text{ milliseconds}$$

$$M = \sqrt{1 + \frac{6}{7}\frac{p_{so}}{p_a}} = 1{\cdot}14$$

$a = 344$ m/s $\qquad U = 392$ m/s $= 0{\cdot}392$ m/millisecond

b) *Pressures on the Building*

Here $H = 3$ m, $B = 10$ m, and $L = 8$ m

Then $S = 3$ m

$$t_c = \frac{3S}{U} = \frac{3 \times 3}{0{\cdot}392} = 23{\cdot}0 \text{ milliseconds} > t_d$$

$$t_t = \frac{L}{U} = \frac{8}{0{\cdot}392} = 20{\cdot}4 \text{ milliseconds} > t_d$$

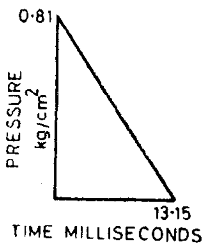
$$t_r = \frac{4S}{U} = \frac{4 \times 3}{0{\cdot}392} = 30{\cdot}6 \text{ milliseconds} > t_d$$

As $t_r > t_d$ no pressure on the back face are considered.

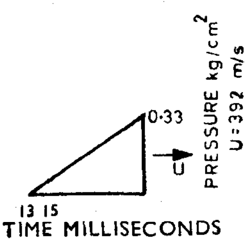
For roof and sides $C_d = -0{\cdot}4$

$$p_{so} + C_d\, q_o = 0{\cdot}35 - 0{\cdot}4 \times 0{\cdot}042 = 0{\cdot}33 \text{ kg/cm}^2$$

The pressure diagrams are as shown below:



Average Front Face Loading



Moving Pressure Pulse on Roof and Side Walls

APPENDIX B

(*Clause* 9.2.1)

TYPICAL EXAMPLES OF EVALUATION OF K_{LM}

B-1. A TYPICAL EXAMPLE OF EVALUATION OF K_{LM} FOR A CANTILEVER COLUMN SUBJECTED TO UNIFORM DISTRIBUTED DYNAMIC LOAD p

B-1.1 The deflected shape of the column in elastic range when p is considered to be acting statically is given by:

$$\phi_{(x)} = 1 - \frac{4}{3}\frac{x}{l} + \frac{1}{3}\left(\frac{x}{l}\right)^4$$

where the deflection at the free end has been taken as unity.

$$K_L = \frac{1}{l}\int_0^l \phi_{(x)}\, dx = \frac{2}{5}$$

If mass is considered to be uniformly distributed along the height of the column:

$$k_M = \frac{1}{l}\int_o^l \phi^2_{(x)}\, dx = \frac{104}{405}\ [\therefore K_{LM} = K_M/K_L = 52/81]$$

Effective stiffness of the cantilever will be defined with respect to the deflection at the free end where $\phi_{(x)} = 1$

$$k_E = \frac{8\ EI}{L^3}$$

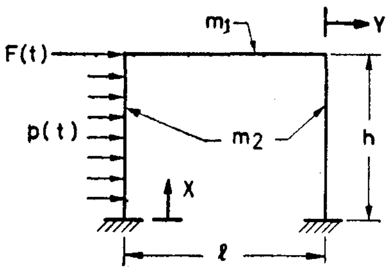
Period of the cantilever is thus obtained as:

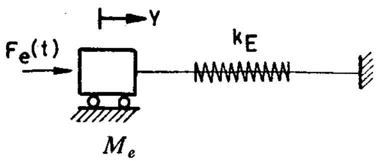
$$T = 2\pi\sqrt{\frac{K_{LM}\ M_t}{k_E}}$$

$$= 1{\cdot}78\ \sqrt{\frac{M_t\ L^3}{EI}}$$

B-2. A TYPICAL EXAMPLE OF EVALUATION OF K_{LM} FOR A SINGLE STOREY FRAME

B-2.1 A single storey rigid frame with distributed masses on the roof and sides is subjected to a concentrated dynamic force $F_{(t)}$ at the roof level plus a distributed dynamic load $p(t)$ on one wall surface. Considering only horizontal motion, the equivalent one degree system is defined such that its displacement is equal to the displacement at the roof. If the walls are assumed to remain straight then the displacement Y_x of a point at a distance x from the base is given by:





$$Y_x = \frac{x}{h}\ Y \quad ; \quad \phi(x) = \frac{x}{h}$$

The mass factor K_M is given by the following equation:

$$k_M = \frac{1}{M_t}\left[\ m_1 l + 2\int_o^h m_2\left(\frac{x}{h}\right)^2 dx\ \right]$$

$$= \left(m_1 l + \frac{2}{3} m_2 h \right) / \left(m_1 l + 2\, m_2 h \right)$$

where m_1 and m_2 are the masses per unit length (along the frame) of the roof and walls respectively.

The equivalent load is:

$$F_e(t) = F(t) + \int_0^h p(t) \left(\frac{x}{h} \right) dx$$

$$= F(t) + \tfrac{1}{2}\, p(t)\, h$$

and the load factor is:

$$K_L = \frac{F(t) + \frac{1}{2}\, p(t)\, h}{F(t) + p(t)\, h}, \text{ then } K_{LM} = K_M / K_L$$

The effective stiffness k_E will be given by the total load [$F(t)$ and $p(t)\ h$] causing unit displacement at the top of the frame. If $F(t)$ and $p(t)$ have different time variation, numerical analysis is necessary.

APPENDIX C

(*Clause* 12.3)

GENERAL RECOMMENDATIONS FOR PLANNING BLAST RESISTANT BUILDINGS

C-1. SIZE OF ROOMS

C-1.1 Small size of rooms generally confine the blast damage to a limited area of the structure, because of the screening action of the partition walls.

C-2. CORRIDORS

C-2.1 Long narrow corridors should be avoided as they tend to increase the extent of damage along the length of the corridors because of 'multiple reflections'.

C-3. PROJECTIONS

C-3.1 All slender projections like, parapets and balconies specially those made of brittle materials should be avoided as far as possible.

C-4. CHIMNEYS

C-4.1 Masonry chimneys on factory buildings and boiler houses are a potential hazard and should be avoided.

C-5. ROOFING AND CLADDING MATERIALS

C-5.1 Brittle roofing materials, such as tiles and corrugated asbestos sheets are especially prone to blast damage. When corrugated galvanized iron sheets are used for roofing and/or cladding, particular attention should be paid to the fixtures fastening the corrugated galvanized iron sheets to the framework.

C-6. USE OF TIMBER AND OTHER INFLAMMABLE MATERIALS

C-6.1 These are especially prone to catch fire in a strafing or incendiary attack and should be best avoided in strategic structures where such attacks might be expected.

C-7. ELECTRIC WIRING

C-7.1 Conduit wiring is preferable to open wiring, as in case of large movement of the walls the conduit will give an added protection to the wiring inside and prevent them from getting cut thus preventing fire hazards due to short circuits.

C-8. GLASS PANES

C-8.1 The most widespread damage due to blast is the breaking of glass panes. The splinters from shattered glass window are dangerous to personnel safety. It is preferable to use non-splintering type glass panes wherever their use cannot be avoided.

C-9. DOORS

C-9.1 Doors should be designed for the front face load.

C-10. WALL THICKNESSES AGAINST FLYING SPLINTERS

C-10.1 For protection against splinters from bombs with equivalent bare charges exploding at a distance of 15 m, the wall thicknesses given in Table 8 will be adequate.

TABLE 8 MINIMUM WALL THICKNESSES AGAINST FLYING SPLINTERS

Material of Wall	Wall Thickness, cm	
	For Bomb with Equivalent Bare Charge of 50 kg	For Bomb with Equivalent Bare Charge of 100 kg
(1)	(2)	(3)
Reinforced concrete	30	38
Plain concrete or brickwork	34	45

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