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Structural Elements Response In Blast Loading

[1]Manish Nigam, [2]Dr. Rajendra Kumar Srivastava, [3]Awadhesh Kumar Singh
[1] Associate Professor, [2] Retd. Engineer in Chief, [3]Assistant Engineer
[1] PSIT College of Engineering, Kanpur, U.P. (INDIA), 209305
[2] Uttar Pradesh, Public Works Department
[3] U.P. PWD, Kanpur Dehat

ABSTRACT- *The Prime objective of this paper is to describe the design of blast load-resisting structural elements for providing sufficient ductility to enable the element to deflect by an amount consistent with the degree of damage permitted; this will entail an initial design based upon extensive flexural plastic deformation. In so deforming , the element should not fail prematurely due to other load effects say shear or locally instability. Unless the element is to be subjected to repeated blast loading for example in a test facility the design should be based on the ultimate limit state. Simple supports should be avoided wherever possible and joints between elements should be carefully detailed to facilitate load transfer.*

Keywords- *Blast loading, blast load resisting structures, explosives, plastic deformation, structural response ,blast resistance deflection curve.*

I. INTRODUCTION

The blast loading for which resistance is to be provided is likely to be an extreme event, and as such has a low probability of occurrence. Thus, it is appropriate to set the partial safety factor for blast loading to a value of unity. Dead loads, storage and other permanent loads should also be assigned a partial safety factor of unity. The magnitude of imposed loads and wind loads acting simultaneously with the blast load are likely to be only a small proportion of their respective design values. A factor of 033 may be applied to the design values of the variable imposed and wind loads when acting in combination with the blast load, as recommended in BS 5950: Part 1 and BS 8110: Part 1 . When other loads are present at the time of the blast loading they may be assumed to act constantly throughout the application of the blast load. The effect of these other loads generally will be to reduce the effective resistance of an element. However, where mass is associated with loads there may be a beneficial effect as a result of the inertial effects of these loads. The design should generally be based upon the characteristic strength of materials — that is with a partial safety factor of unity, unless there is evidence to show that the mean strengths of a particular material are generally higher than the specified minimum. For example, the characteristic yield stress for grade 50 or lower structural steel may be increased by 10% in design calculations involving blast loading. Under the action of rapidly applied loads the rate of strain application increases and this may have a marked influence on the mechanical properties of structural materials. In comparison with the mechanical properties under static loading the effects may be summarized as follows:

a) The yield stress of structural steel or steel reinforcement bars, f_y, increases significantly to the dynamic yield stress, f_{dy}
b) The ultimate tensile strength of structural steel or steel reinforcement bars, f. in which account is taken of strain hardening effects, increases slightly to the dynamic ultimate strength, f_{du}.
c) The compressive strength of concrete, f_{cu}, increases significantly to the dynamic compressive strength, f_{dcu}.
d) The modulus of elasticity of both steel and concrete remains insensitive to the rate of loading.

e) The elongation at failure of structural steel is relatively insensitive to the rate of loading.

The factor by which the static stress is enhanced in order to calculate the dynamic stress is known as the dynamic increase factor (DIF). Typical values of DIF for structural steel and reinforced concrete are given in Table 1. The dynamic stress to be used in the design of reinforced concrete

Table 1 Dynamic increase factors (DJF) for design of reinforced concrete and structural steel elements

Type of stress	Concrete	Reinforcing bars		Structural steel	
	f_{dcu} / f_{cu}	f_{dy} / f_y	f_{du} / f_u	f_{dy} / f_y *	f_{du} / f_u
Bending	1.25	1.20	1.05	1.20	1.05
Shear	1.00	1.10	1.00	1.20	1.05
Compression	1.15	1.10	–	1.10	–

1.1 Deformation limits

The controlling criterion in the design of blast-resistant structural elements is normally a limit on the deformation or deflection of the element. In this way the degree of damage sustained by the element may be controlled. The damage level that may be tolerated in any particular situation will depend on what is to be protected, for example, the structure itself, the occupants of a building or equipment within the building. There are two methods by which limiting element deformations may be specified: by using the support rotation, U (see Fig. 5.1) and the ductility ratio

$$\mu = \frac{total\ deflection}{deflection\ at\ elastic\ \lim it} \times \frac{X_m}{X_E}$$

In general, deformations in reinforced concrete elements are expressed in terms of support rotations whilst ductility ratios are used for structural steel elements. For the protection of personnel and equipment through the attenuation of blast pressures and to shield them from the effects of primary and secondary fragments and falling portions of the structure, recommended deformation limits are given under protection category 1 in Table 2. For the protection of structural elements themselves from collapse under the action of blast loading, the recommended deformation limits are given under protection category 2 in Table 5.2. It should be noted that these limits imply extensive plastic deformation of the elements and the need for subsequent repair or replacement before they may be re-used. For situations where re-use is required without repair, deformations should be maintained within the elastic range, i.e. $\mu \leq 1$. This latter design condition is likely to lead to massive and consequently costly construction. In addition to these considerations for individual elements there remains, of: course, a requirement for the overall structure to remain stable in the event of being subject to blast loading.

Table 2. Deformation limits

	Protection category			
	1		2	
	θ	μ	θ	μ
Reinforced concrete beams and slab	2°*	Not	4° †	Not applicable

Structural steel beams and plates	2°	10	12°	20

Structural response of steelwork

Structural steel can generally be considered as exhibiting a linear stress–strain relationship up to the yield point, beyond which it can strain substantially without appreciable increase in stress. This yield plateau / extends to a ductility ratio, μ, of between 10 and 15. Beyond this range strain hardening occurs and after reaching a maximum stress – known as the tensile strength – a drop in stress accompanies further elongation and precedes fracture at a strain of approximately 20–30%. Typical static and dynamic stress—strain curves for steel are shown in Fig. 5.5. Structural steels of strengths higher than grade 50 generally exhibit smaller elongations at rupture and should be used with caution when very large ductility's are a prerequisite of design. The design plastic moment, M_p, for steel elements with $\mu \leq 3$ is given by $M_p = f_{ds}(Z+S)/2$ Where Z and S are the elastic and plastic section moduli, respectively for $\mu > 3$ $M_p = f_{ds}S$

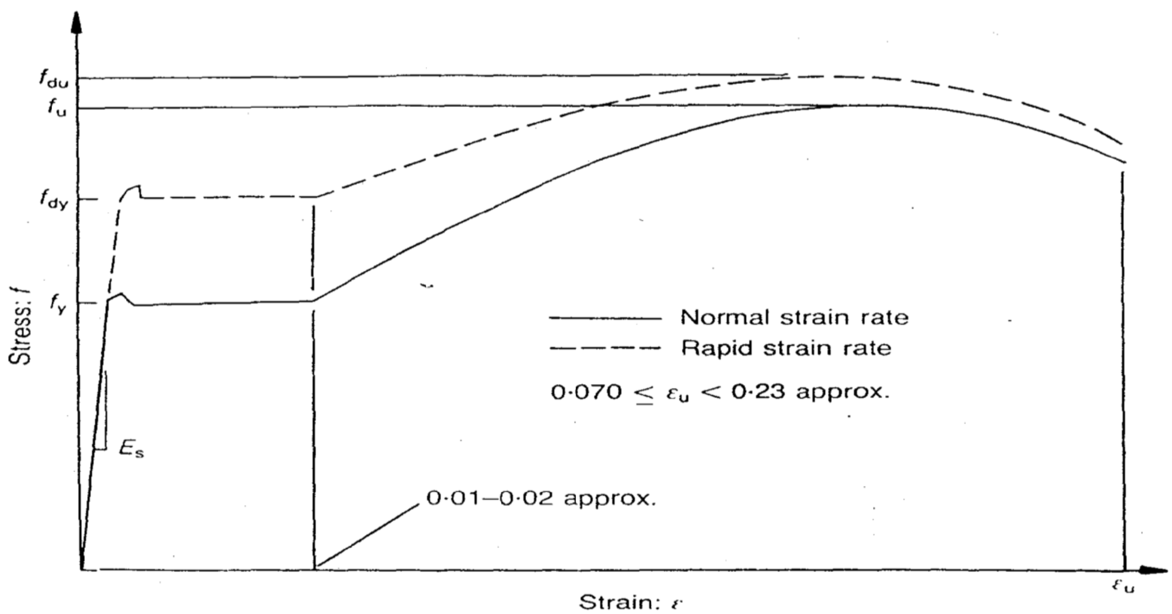

Fig. 1

Reinforced concrete is relatively massive and, as such, is more appropriate than steel to resist the close-in effects of large explosions in the impulsive regime. Steelwork is better suited to resist relatively low pressures of a quasi-static nature. The massive nature of reinforced concrete implies 'stocky' sections whose ultimate capacity is reasonably predictable. The slender nature of structural steel sections can cause local instability and unpredictable ultimate capacities There are two other significant differences between the materials. First, the rebound in concrete structures is small because cracking causes internal damping. In steel, the rebound can be quite large, particularly for short duration loads on relatively flexible elements. Therefore, steel structures must be designed to support significant reversals of loading. Second, in reinforced concrete, separate reinforcing steel is provided to resist flexure, shear and torsion. In steel, complex stress combinations occur which are difficult to predict and which can potentially cause distress. Stress concentrations at welds and notches must also be carefully considered if the full strength of the section is to be realized.

Table 3 : Dynamic design stresses for reinforced concrete

Type of stress	Protection category	Dynamic design stress	
		Concrete	Reinforcing bars
Bending	1 2	f_{du} f_{dcu}	f_{dy} $f_{dy} + (f_{du} - f_{dy})/4$
Shear	1 2	f_{du} f_{dcu}	f_{dy} f_{dy}
Compression	1 and 2	f_{dcu}	f_{dy}

determined using plastic theory, modified to account for static loads present at the time of the blast loading, X_E is the deflection at the limit of elastic behaviour, K_E is the elastic stiffness and X_m is the maximum permitted deflection corresponding to the limiting support rotation, θ, given in Table 2 for the appropriate protection category.

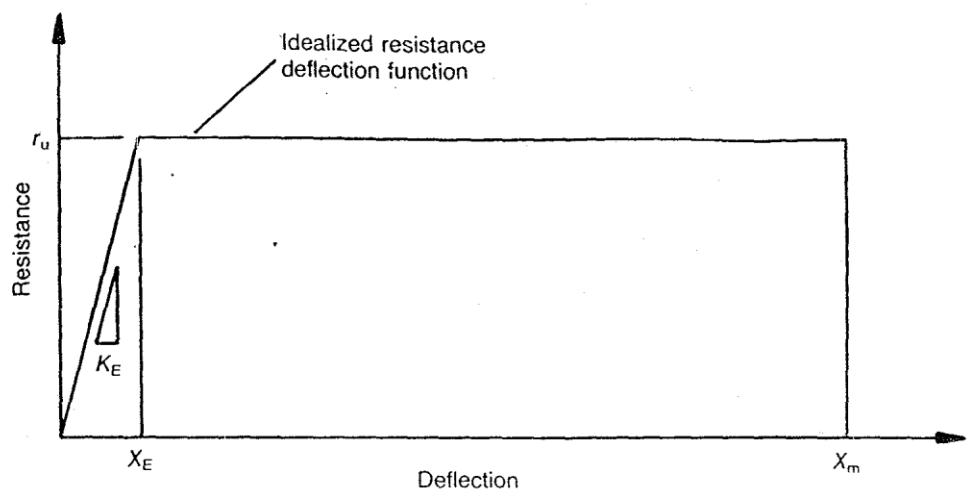

Fig.2. **idealized resistance–deflection curve**

II. OBSERVATIONS AND STUDY

2.1 Response to impulse

Idealization of blast load. The blast load may be idealized to a triangular pressure—time function with zero rise time as shown in Fig. 5.7. In the impulsive regime, the duration of the applied load, td, is short in relation to the response time, tm, of the element (the time for the element to attain a deflection X_m), such that $t_m / t_d \geq 3$. The loading is assumed to be uniformly distributed and is represented by the specific impulse, i. Design for flexure.

(a) Design objective. To provide flexural strength and ductility so that the kinetic energy delivered by the impulsive load may be resisted by the strain energy developed by the member in deflecting. to Xm.

(b) Basic impulse equation. For support rotations, $\theta < 5°$, the elastic part of the idealized resistance—deflection function must be considered such that

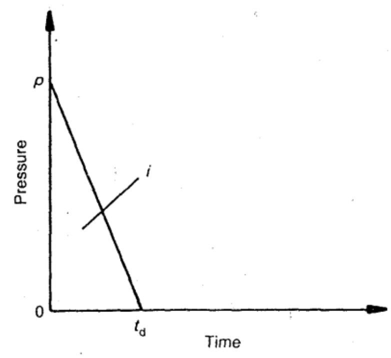

Fig. 3. idealization of blast load

$$\frac{i^2}{2K_{LM}m} = \frac{r_u X_E}{2} + r_u\left(X_m - X_E\right)$$

where K_{LM} is the appropriate load—mass factor obtained from Table B.1 (in Appendix B) and m is the unit mass of the element.

(c) Design steps.

Step 1. Define the resistance—deflection function in terms of:

(i) $r_u = f(M_P, L)$ (see Table B.3), where

$$Mp = \frac{A_s f_{ds}}{b} d_c$$

for a type 2 section and

$$p_s = \frac{A_s}{bd_c}$$

(ii) $X_m = f(\theta)$

(iii) $K_E = f\,(E,I,L)$ (see Table B.4), where $I = \rho_s f b_{ds} d_c^2$ (see Fig. 5.8) and E is the modulus of elasticity of concrete.

(iv) $X_E = r_U/K_E$.

Step 2. Determine K_{LM} (see Table B.1) and m = ρ_{dc}, where p is the density of concrete.

Step 3. Solve for d based on assumed value of 5.

Step 4. Calculate tm i/re, hence tm ltd and check whether appropriate design procedure has been used, i.e. quasi-static/dynamic or impulse.

Design for shear. After the flexural design of the element has been completed, the required quantity of shear reinforcement must be determined. The ultimate shear is developed when the resistance reaches the ultimate value, r_u, and hence the shear reinforcement is a function of the resistance of the element and not of the applied load. There are two critical locations where shear must be considered in the design of reinforced concrete elements. The ultimate shear stress, v_u, is calculated at a distance d_c from the supports to check the diagonal tension stress and to provide shear reinforcement in the form of stirrups, as necessary. The direct shear force or the ultimate support shear, V_s, is calculated at the face of the support to determine the required quantity of diagonal bars.

(a) Ultimate shear stresses. Values of ultimate shear stresses, v_u, at distance d from the face of the support are given in Table 5

(b) Shear capacity of concrete'. The capacity of the concrete, v_u, may be taken from BS 8110: Part 1 with the partial safety factor, γ_m = 1.25, removed.

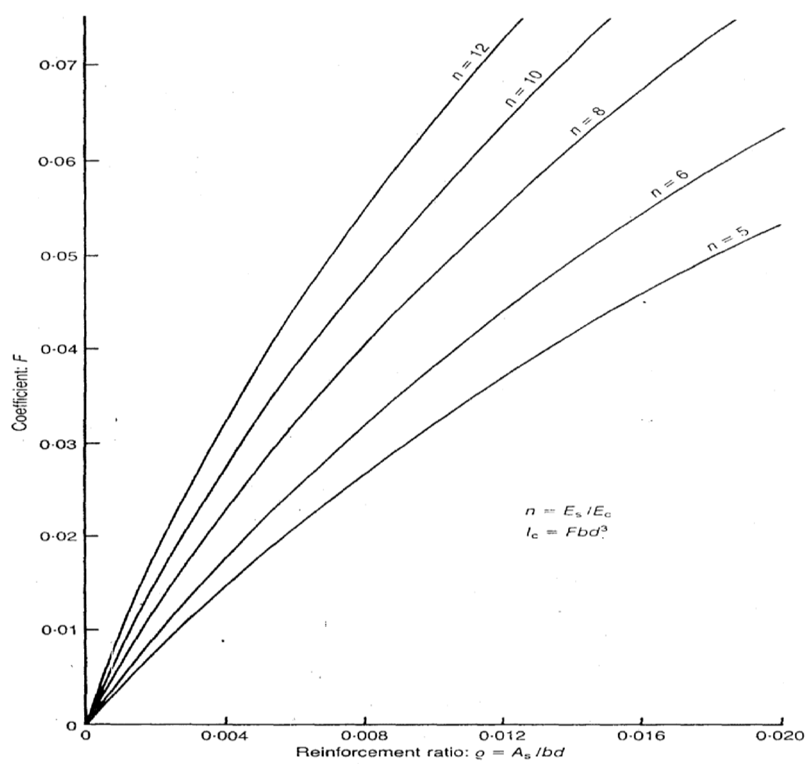

Fig. 4. Coefficient for moment of inertia of cracked sections with equal reinforcement on opposite faces

Design of shear reinforcement. Whenever the ultimate shear stress, v_u, exceeds the shear capacity of the concrete, v_c, shear reinforcement must be provided to carry the excess. The required area of stirrups, A_v, is calculated from

$$A_v = \frac{(v_u - v_c)bs}{f_{ds}}$$

where s is the spacing of stirrups in the direction parallel to the longitudinal reinforcement. For type 2 sections, the minimum design stress, $(v_u - v_c)$, to be used in the calculation of shear reinforcement is $0.85v_c$.

(d) Ultimate support shears. Values of ultimate support shears, V_s, at the supports are given in Table B.6.

(e) Direct shear capacity of concrete. For type 2 sections where the design support rotation, θ, exceeds 2°, the ultimate direct shear capacity of the concrete, V_d, is zero and diagonal bars are required to take all direct shear.

(f) Design of diagonal bars. The required area of diagonal bars at 45°, Ad, is determined from

$$A_d = V_s b / f_{ds}$$

Response to quasi-static/dynamic loading

Idealization of blast load. The blast load may be idealized into a triangular pressure–time function with zero rise time as shown in Fig. 5.7 or to other idealizations for which response charts based on SDOF analyses are available. These may include square pulses with zero rise time, gradually applied loads, or triangular pulses with a finite rise time (see Appendix C). In the quasi-static/dynamic response regimes, the duration of the applied load, td, is long in relation to the response time of the element, t_d, such that $t_m/t_d < 3$. The loading is assumed to be uniformly distri/ buted and is represented by the pressure, p, which varies with tithe, t.

Design for flexure.

(a) Design objective. To provide flexural strength and ductility such that the work done by the applied blast load may be resisted by the strain energy developed by the member in deflecting to X_m.

(b) Design steps.

Step 1. Define resistance–deflection function in terms of:

(i) $r_u = f(M_p, L)$ (see Table B.3),

Where

$$Mp = \frac{A_s f_{ds}}{b}(d - 0.45x)$$ for a type 1 section and

$$x = \frac{A_s f_{ds}}{0.6 b f_{dc}}$$

is the depth from the compression face to the neutral axis and d is the effective depth of the tension reinforcemert and $p_s = (A_s/bd)$.

(ii) X_m f(θ).

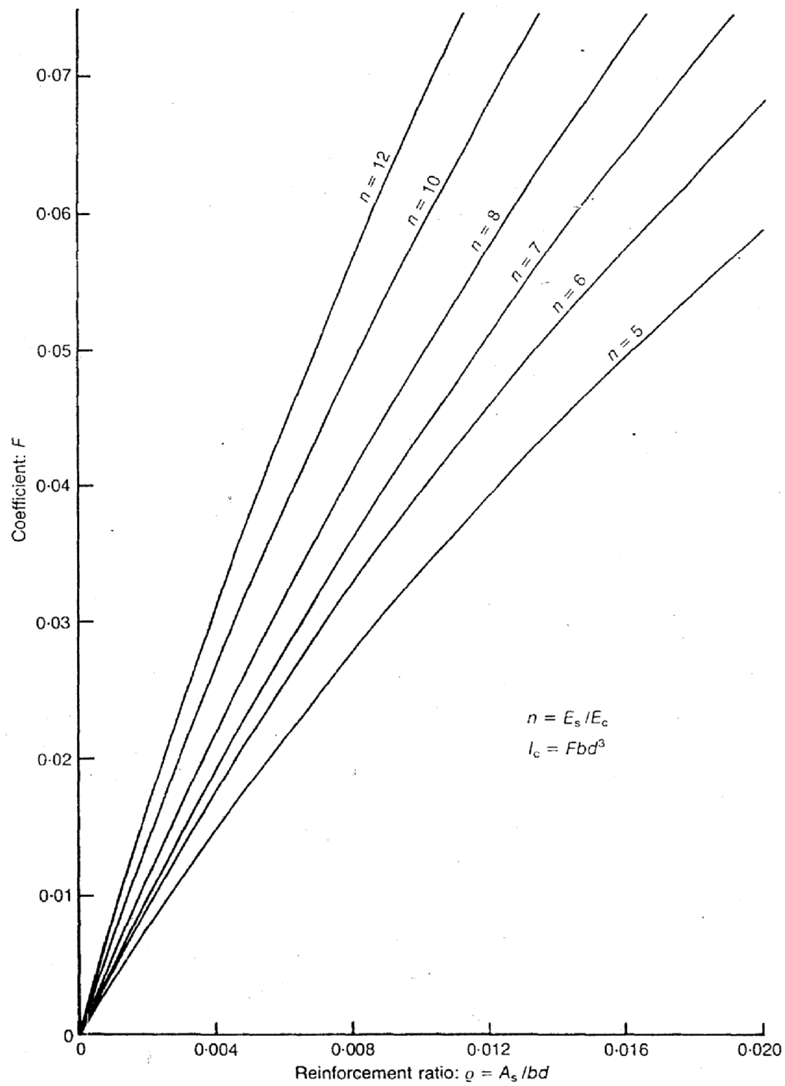

Fig. 5. Coefficient for moment of inertia of cracked sections with tension reinforcement only

(iii) K_E f(E, I, L) (see Table B.4), where I = Fbd^3 (see Fig. 5.9).

(iv) $X_E = r_U / K_E$.

Step 2. Calculate natural period of element

$$T = 2\pi\sqrt{\frac{K_{LM} m}{K_E}}$$

where K_{LM} is the appropriate load—mass factor from Table B.1 and m is the unit mass of the element.

Step 3. Refer to appropriate SDOF response chart in Appendix C for an elasto-plastic system under idealized load to obtain:

(i) $\mu = X_m/X_E$, hence X_m and θ

(ii) t_m / t_d and hence check whether the appropriate design procedure has been used, i.e. quasi-static/dynamic or impulse.

Design for shear. The design for shear of type 1 reinforced concrete elements responding in the pressure–time regime is similar to that for type 2 elements responding to impulse with the following exceptions:

(a) Ultimate shear stresses.

(i) Unless specifically required as shear reinforcement, stirrups are not required in slabs for which $\theta \leq 1^{o}$.

(ii) The values of ultimate shear stresses should be calculated at distance, d, from the supports, rather than d .

(b) Ultimate support shears.

(i) For type 1 sections where $\theta \leq 2°$, the ultimate direct shear force that can be resisted by the concrete, V_d, is given by $V_d = 0.l5 f_{dc} bd$

(ii) The corresponding area of 45° diagonal bars, A_d, required becomes

$$Ad = \frac{V_s b - V_d}{f_{ds}}$$

III . RESULTS AND INTERPRETATIONS OF DATA

3.1 Dynamic reactions

When a reinforced concrete element is loaded dynamically, the loads transferred to the supports are known as the dynamic reactions. The magnitude of these reactions, which may be used as the basis of design of supporting elements, is a function of both the total resistance, K, and the total load, P, applied to the element, both of which vary with time. They may be expressed generally in the form $V = \alpha R + \beta P$. ,provides values of α and β for differing support and loading arrangements. It should be noted that the procedures described in this chapter are largely concerned with the design of elements. This is likely to be the critical design consideration when dealing with relatively large quantities of explosive at close proximity to a building structure. In situations where the overall stability of the building becomes critical, the designer will need to consider the provision of stabilizing elements such as shear walls. The external loading on complete structures and the associated blast wave–structure interaction is fully described

3.2 Introduction to the design of structural steel elements to resist blast loading

Design stresses

Table 4 summarizes the relevant dynamic stresses, f_{ds} and f_{dv} for flexure and shear, respectively, to be used in the design of structural steel e1eients.

Table 4 Dynamic design stresses for structural steel

Type of stress	Protection category	Dynamic design stress
Bending	1 2	$f_{ds} = f_{dy}$ $f_{ds}\ f_{dv} + (f_{du} - f_{dy})/4$
Shear	I and 2	$f_{dv} = 0.55\ f_{ds}$

3.3 Idealization of structural response

The structural response of a steel element subjected to flexure may be represented by the idealized resistance–deflection function where r is the unit ultimate dynamic resistance determined using plastic theory modified to account for static loads present at the time of the blast loading; X_E is the deflection at the limit of elastic behaviour; K_E is the elastic stiffness and X_m is the maximum permitted deflection corresponding to the more critical of the limiting support rotation, θ, or ductility ratio, μ, given in Table 2 for the appropriate protection category.

Response to quasi-static/dynamic loading

Idealization of blast load. The blast load may be idealized into a triangular pressure—time function with zero rise time as illustrated or to other idealizations for which response charts based upon SDOF analyses are available

3.4 Design for flexure.

(a) Design objective. To provide flexural strength and ductility such that the work done by the applied blast load may be resisted by the strain energy developed by the member in deflecting to X_m.

(b) Design steps.

Step 1. Carry out preliminary design assuming an equivalent static ultimate resistance as defined in Table 5.

(i) Determine the required resistance, r, using Table 5.

(ii) Determine the required plastic moment of resistance, $M_p = f(r_u, L)$

Table 5. Equivalent static ultimate resistance for. preliminary design of steel elements in flexure

Protection category	Equivalent static ultimate resistance
1 2	$r_u = 1.0\ \rho max$ $r_u = 0.5\ \rho max$

(iii) Select a steel member using appropriate relationship between M_ρ, f_{ds}, S and Z.

Step 2. Calculate the natural period of the element using the following equation

$$T = 2\pi\sqrt{\frac{K_{LM} m}{K_E}}$$

where KM is the appropriate load–mass factor from Table B.1 and m is the unit mass of the element.

Step 3. Refer to appropriate SDOF response chart in Appendix C for an elasto-plastic system under idealized load to obtain

(i) $\mu = X_m/X_E$, hence X_m and θ, and

(ii) t_m/t_d and hence check whether the appropriate design procedure has been used, i.e. quasi-static/dynamic.

Check for shear and secondary effects.

(a) Ultimate support shear. Values of ultimate support shears, V.

(b) Ultimate shear capacity. The ultimate shear capacity is given by $V_p = f_{dv} A_w$, where A_w is the area of the web.

IV . CONCLUSIONS

Local buckling and web stiffeners. In order to ensure that a steel beam will attain fully plastic behaviour and hence the required ductility at plastic hinge locations, it is necessary that the elements of a beam section meet the normal minimum thickness requirements sufficient to prevent a local buckling failure. Similarly, web stiffeners should be employed at locations of concentrated loads and reactions to provide a gradual transfer of forces to the web. Lateral bracing. Members subjected to bending about their strong axes may be susceptible to lateral-torsional buckling in the direction of their weak axes if their compression flange .is not laterally braced. In order for a plastically designed member to reach its collapse mechanism, lateral supports must be provided at and adjacent to the location of plastic hinges. In designing such bracing due consideration should be given to the possibility of rebound which induces stress reversal.

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