CHAPTER 5. STRUCTURAL STEEL DESIGN

CHAPTER 5

STRUCTURAL STEEL DESIGN

INTRODUCTION

5-1. Purpose

The purpose of this manual is to present methods of design for protective construction used in facilities for development, testing, production, storage, maintenance, modification, inspection, demilitarization, and disposal of explosive materials.

5-2. Objective

The primary objectives are to establish design procedures and construction techniques whereby propagation of explosion (from one structure or part of a structure to another) or mass detonation can be prevented and to provide protection for personnel and valuable equipment.

The secondary objectives are to:

(1) Establish the blast load parameters required for design of protective structures.
(2) Provide methods for calculating the dynamic response of structural elements including reinforced concrete, and structural steel.
(3) Establish construction details and procedures necessary to afford the required strength to resist the applied blast loads.
(4) Establish guidelines for siting explosive facilities to obtain maximum cost effectiveness in both the planning and structural arrangements, providing closures, and preventing damage to interior portions of structures because of structural motion, shock, and fragment perforation.

5-3. Background

For the first 60 years of the 20th century, criteria and methods based upon results of catastrophic events were used for the design of explosive facilities. The criteria and methods did not include a detailed or reliable quantitative basis for assessing the degree of protection afforded by the protective facility. In the late 1960's quantitative procedures were set forth in the first edition of the present manual, "Structures to Resist the Effects of Accidental Explosions". This manual was based on extensive research and development programs which permitted a more reliable approach to current and future design requirements. Since the original publication of this manual, more extensive testing and development programs have taken place. This additional research included work with materials other than reinforced concrete which was the principal construction material referenced in the initial version of the manual.

Modern methods for the manufacture and storage of explosive materials, which include many exotic chemicals, fuels, and propellants, require less space for a given quantity of explosive material than was previously needed. Such concentration of explosives increases the possibility of the propagation of accidental explosions. (One accidental explosion causing the detonation of other explosive materials.) It is evident that a requirement for more accurate design techniques is essential. This manual describes rational design methods to provide the required structural protection.

These design methods account for the close-in effects of a detonation including the high pressures and the nonuniformity of blast loading on protective structures or barriers. These methods also account for intermediate and far-range effects for the design of structures located away from the explosion. The dynamic response of structures, constructed of various materials, or combination of materials, can be calculated, and details are given to provide the strength and ductility required by the design. The design approach is directed primarily toward protective structures subjected to the effects of a high explosive detonation. However, this approach is general, and it is applicable to the design of other explosive environments as well as other explosive materials as mentioned above.

The design techniques set forth in this manual are based upon the results of numerous full- and small-scale structural response and explosive effects tests of various materials conducted in conjunction with the development of this manual and/or related projects.

5-4. Scope

It is not the intent of this manual to establish safety criteria. Applicable documents should be consulted for this purpose. Response predictions for personnel and equipment are included for information.

In this manual an effort is made to cover the more probable design situations. However, sufficient general information on protective design techniques has been included in order that application of the basic theory can be made to situations other than those which were fully considered.

This manual is applicable to the design of protective structures subjected to the effects associated with high explosive detonations. For these design situations, the manual will apply for explosive quantities less than 25,000 pounds for close-in effects. However, this manual is also applicable to other situations such as far- or intermediate-range effects. For these latter cases the design procedures are applicable for explosive quantities in the order of 500,000 pounds which is the maximum quantity of high explosive approved for aboveground storage facilities in the Department of Defense manual, "Ammunition and Explosives Safety Standards", DOD 6055.9-STD. Since tests were primarily directed toward the response of structural steel and reinforced concrete elements to blast overpressures, this manual concentrates on design procedures and techniques for these materials. However, this does not imply that concrete and steel are the only useful materials for protective construction. Tests to establish the response of wood, brick blocks, and plastics, as well as the blast attenuating and mass effects of soil are contemplated. The results of these tests may require, at a later date, the supplementation of these design methods for these and other materials.

Other manuals are available to design protective structures against the effects of high explosive or nuclear detonations. The procedures in these manuals will quite often complement this manual and should be consulted for specific applications.

Computer programs, which are consistent with procedures and techniques contained in the manual, have been approved by the appropriate representative of the US Army, the US Navy, the US Air Force and the Department of Defense Explosives Safety Board (DDESB). These programs are available through the following repositories:

(1) Department of the Army
Commander and Director
U.S. Army Engineer
Waterways Experiment Station
Post Office Box 631
Vicksburg, Mississippi 39180-0631
Attn: WESKA

(2) Department of the Navy
Commanding Officer
Naval Civil Engineering Laboratory
Port Hueneme, California 93043
Attn: Code L51

(3) Department of the Air Force
Aerospace Structures
Information and Analysis Center
Wright Patterson Air Force Base
Ohio 45433
Attn: AFFDL/FBR

If any modifications to these programs are required, they will be submitted for review by DDESB and the above services. Upon concurrence of the revisions, the necessary changes will be made and notification of the changes will be made by the individual repositories.

5-5. Format

This manual is subdivided into six specific chapters dealing with various aspects of design. The titles of these chapters are as follows:

Chapter 1	Introduction
Chapter 2	Blast, Fragment, and Shock Loads
Chapter 3	Principles of Dynamic Analysis
Chapter 4	Reinforced Concrete Design
Chapter 5	Structural Steel Design
Chapter 6	Special Considerations in Explosive Facility Design

When applicable, illustrative examples are included in the Appendices.

Commonly accepted symbols are used as much as possible. However, protective design involves many different scientific and engineering fields, and, therefore, no attempt is made to standardize completely all the symbols used. Each symbol is defined where it is first used, and in the list of symbols at the end of each chapter.

CHAPTER CONTENTS

5-6. General

This chapter contains procedures and guidelines for the design of blast-resistant steel structures and steel elements. Light construction and steel framed acceptor structures provide an adequate form of protection in a pressure range of 10 psi or less. However, if fragments are present, light-gage construction may only be partially appropriate. The use of structural steel frames in combination with precast concrete roof and wall panels (Chapter VI) will provide a measure of fragment protection at lower pressure ranges. Containment structures or steel elements of containment structures, such as blast doors, ventilation closures, fragments shields, etc. can be designed for almost any pressure range. This chapter covers detailed procedures and design techniques for the blast-resistant design of steel elements and structures subjected to short-duration, high-intensity blast loading. Provisions for inelastic, blast-resistant design will be consistent with conventional static plastic design procedures. Steel elements such as beams, beam columns, open-web joists, plates and cold-formed steel panels are considered. In addition, the design of steel structures such as rigid frames, and braced frames are presented as they relate to blast-resistant design. Special considerations for blast doors, penetration of fragments into steel, and unsymmetrical bending are also presented.

STEEL STRUCTURES IN PROTECTIVE DESIGN

5-7. Differences Between Steel and Concrete Structures in Protective Design

Qualitative differences between steel and concrete protective structures are summarized below:

(1) In close-in high-impulse design situations where a containment structure is utilized, a massive reinforced concrete structure, rather than a steel structure, is generally employed in order to limit deflections and to offer protection against the effects of primary and secondary fragments. However, elements of containment structures such as blast doors, ventilation closures, etc., are generally designed using structural steel. Fragment protection is usually accomplished by increasing the element thickness to resist fragment penetration or by providing supplementary fragment protection. In some cases, structural steel can be used in the design of containment cells. However, explosive charge weights are generally low; thereby preventing brittle modes of failure (Section 5-18.3) due to high pressure intensity.

(2) Structural steel shapes are considerably more slender, both in terms of the overall structure and the components of a typical member cross section. As a result, the effect of overall and local instability upon the ultimate capacity is an important consideration in steel design. Moreover, in most cases, plate elements and structures will sustain large deformations in comparison to those of more rigid concrete elements.

(3) The amount of rebound in concrete structures is considerably reduced by internal damping (cracking) and is essentially eliminated in cases where large deformations or incipient failure are permitted to occur. In structural steel, however, a larger response in rebound, up to 100 percent, can be obtained for a combination of short duration load and a relatively flexible element. As a result, steel structures require that special provisions be made to account for extreme responses of comparable magnitude in both directions.

(4) The treatment of stress interaction is more of a consideration in steel shapes since each element of the cross section must be considered subject to a state of combined stresses. In reinforced concrete, the provision of separate steel reinforcement for flexure, shear and torsion enables the designer to consider these stresses as being carried by more or less independent systems.

(5) Special care must be taken in steel design to provide for connection integrity up to the point of maximum response. For example, in order to avoid premature brittle fracture in welded connections, the welding characteristics of the particular grade of steel must be considered and the introduction of any stress concentrations at joints and notches in main elements must be avoided.

(6) If fragments are involved, special care must be given to brittle modes of failure as they affect construction methods. For example, fragment penetration depth may govern the thickness of a steel plate.

5-8. Economy of Design of Protective Structures in the Inelastic Range

The economy of facility design generally requires that blast-resistant structures be designed to perform in the inelastic response range during an accident. In order to ensure the structure's integrity throughout such severe conditions, the facility designer must be cognizant of the various possible failure modes and their inter-relationships. The limiting design values are dictated by the attainment of inelastic deflections and rotations without complete collapse. The amount of inelastic deformation is dependent not only upon the ductility characteristics of the material, but also upon the intended use of the structure following an accident as well as the protection required. In order for the structure to maintain such large deformations, steps must be taken to prevent premature failure by either brittle fracture or instability (local or overall). Guidelines and criteria for dealing with these effects are presented in the body of this chapter.

5-9. Applications of Steel Elements and Structures In Protective Design

The design procedures and applications of this chapter are directed toward steel acceptor- and donor-type structures.

Acceptor-type structures are removed from the immediate vicinity of the detonation. These include typical frame structures with beams, columns and beam-columns composed of standard structural shapes, and built-up sections. In many cases, the relatively low blast pressures suggest the use of standard building components such as open-web joists, prefabricated wall panels and roof decking detailed as required to carry the full magnitude of the dynamic loads. Another economical application can be the use of entire pre-engineered buildings, strengthened locally, to adapt their designs to low blast pressures (up to 2 psi) with short duration. For guidelines on the blast evaluation of pre-engineered buildings, see "Special Provisions for Pre-engineered Buildings", Chapter VI.

Donor-type structures, which are located in the immediate vicinity of the detonation may include steel containment cells or steel components of reinforced concrete containment structures such as blast doors or ventilation and electrical closure plates. In some cases, the use of suppressive shielding to control or confine the hazardous blast, fragment, and flame effects of detonations may be an economically feasible alternative. A brief review of suppressive shield design and criteria is outlined in Section 6-23 to 6-26 of Chapter VI. The high blast pressures encountered in these structures suggest the use of large plates or built-up sections with relatively high resistances. In some instances, fragment impact or pressure leakage is permitted.

5-10. Application of Dynamic Analysis

The first step in a dynamic design entails the development of a trial design considering facility requirements, available materials, and economy with members sized by a simple preliminary procedure. The next step involves the performance of a dynamic analysis to determine the response of the trial design to the blast and the comparison of the maximum response with the deformation limits specified in this chapter. The final design is then determined by achieving an economical balance between stiffness and resistance such that the calculated response under the blast loading lies within the limiting values dictated by the operational requirements of the facility.

The dynamic response calculation involves either a single-degree-of-freedom analysis using the response charts in Chapter 3, or, in more complex structures, a multi-degree-of-freedom analysis using available dynamic elasto-plastic frame programs.

A single-degree-of-freedom analysis may be performed for the design analysis of either a given structural element or of an element for which a preliminary design has been performed according to procedures given in this chapter. Since this type of dynamic analysis is described fully with accompanying charts and tables in Chapter 3, it will not be duplicated herein. In principle, the structure or structural element is characterized by an idealized, bilinear, elasto-plastic resistance function and the loading is treated as an idealized triangular (or bilinear) pulse with zero rise time (Chapter 3). Response charts are presented in Chapter 3 for determining the ratio of the maximum response to the elastic response and the time to reach maximum response for the initial response. The equations presented for the dynamic reactions are also applicable to this chapter.

Multi-degree-of-freedom, nonlinear dynamic analyses of braced, and unbraced rigid frames can be performed using programs available through the repositories listed in Section 5-4 and through the reports listed in the bibliography at the end of this chapter.

PROPERTIES OF STRUCTURAL STEEL

5-11. General

Structural steel is known to be a strong and ductile building material. The significant engineering properties of steel are strength expressed in terms of yield stress and ultimate tensile strength, ductility expressed in terms of percent elongation at rupture, and rigidity expressed in terms of modulus of elasticity. This section covers the mechanical properties of structural steel subjected to static loading and dynamic loading. Recommended dynamic design stresses for bending and shear are then derived. Structural steels that are admissible in plastic design are listed.

5-12. Mechanical Properties

5-12.1. Mechanical Properties Under Static Loading, Static Design Stresses

Structural steel generally can be considered as exhibiting a linear stress-strain relationship up to the proportional limit, which is either close to or identical to the yield point. Beyond the yield point, it can stretch substantially without appreciable increase in stress, the amount of elongation reaching 10 to 15 times that needed to reach yield, a range that is termed "the yield plateau". Beyond that range, strain hardening occurs, i.e., additional elongation is associated with an increase in stress. After reaching a maximum nominal stress called "the tensile strength", a drop in the nominal stress accompanies further elongation and precedes fracture at an elongation (at rupture) amounting to 20 to 30 percent of the specimen's original length (see Figure 5-1). It is this ability of structural steel to undergo sizable permanent (plastic) deformations before fracturing, i.e., its ductility, that makes steel a construction material with the required properties for blast resistant design.

Some high strength structural steels do not exhibit a sharp, well defined yield plateau, but rather show continuous yielding with a curved stress-strain relation. For those steels, it is generally accepted to define a quantity analogous to the yield point, called "the yield stress", as that stress which would produce a permanent strain of 0.2 percent or a total unit elongation of 0.4 to 0.5 percent. Although such steels usually have a higher yield stress than those steels which exhibit definite yield and tensile stresses, their elongation at rupture is generally smaller. Therefore, they should be used with caution when large ductilities are a prerequisite of design.

Blast-resistant design is commonly associated with plastic design since protective structures are generally designed with the assumption that economy can be achieved when plastic deformations are permitted. The steels to be used should at least meet the requirements of the American Institute of Steel Construction (AISC) Specification in regard to the adequacy for plastic design.

Since the average yield stress for structural steels having a specified minimum yield stress of 50 ksi or less is generally higher than the specified minimum, it is recommended that the minimum design yield stress, as specified by the AISC specification, be increased by 10 percent. That is, the average yield stress to be used in a blast resistant design shall be 1.1 times the minimum yield stress for these steels. This increase, which is referred to as the increase factor (a), should not be applied to high strength steels since the average increase may be less than 5 percent.

The minimum yield stress, f_y, and the tensile stress, f_u, (minimum) for structural steel shapes and plates which conform to the American Society for Testing and Materials (ASTM) Specification are listed in Table 5-1. All are admissible in plastic design except for ASTM A514 which exhibits the smallest reserve in ductility since the minimum tensile stress is only 10 percent higher than the minimum yield stress. However, elastic dynamic design may require the use of this steel or its boiler plate equivalent, as in ASTM A517.

5-12.2. Mechanical Properties Under Dynamic Loading, Dynamic Increase Factors

The effects of rapid loading on the mechanical behavior of structural steel have been observed and measured in uniaxial tensile stress tests. Under rapidly applied loads, the rate of strain increases and this has a marked influence on the mechanical properties of structural steel.

Considering the mechanical properties under static loading as a basis, the effects of increasing strain rates are illustrated in Figure 5-1 and can be summarized as follows:

(1) The yield point increases substantially to the dynamic yield stress value. This effect is termed the dynamic increase factor for yield stress.
(2) The modulus of elasticity in general will remain insensitive to the rate of loading.
(3) The ultimate tensile strength increases slightly. However, the percentage increase is less than that for the yield stress. This effect is termed the dynamic increase factor for ultimate stress.
(4) The elongation at rupture either remains unchanged or is slightly reduced due to increased strain rate.

In actual members subjected to blast loading, the dynamic effects resulting from the rapid strain rates may be expressed as a function of the time to reach yielding. In this case, the mechanical behavior depends on both the loading regime and the response of the system which determines the dynamic effect felt by the particular material.

For members made of ASTM A36 and A514 steels, studies have been made to determine the percentage increase in the yield stress as a function of strain rate. Design curves for the dynamic increase factors (DIF) for yield stresses of A36 and A514 structural steel are illustrated in Figure 5-2. Even though ASTM A514 is not recommended for plastic design, the curve in Figure 5-2 may be used for dynamic elastic design.

The strain rate, assumed to be a constant from zero strain to yielding, may be determined according to Equation 5-1:

$$\dot{\epsilon} = f_{ds}/E_s t_E \qquad \text{5-1}$$

where

$\dot{\epsilon}$ = average strain rate in the elastic range of the steel (in/in/sec)

t_E = time to yield (sec)

f_{ds} = dynamic design stress (Section 5-13)

Dynamic increase factors for yield stresses in various pressure levels in the bending, tension, and compression modes are listed in Table 5-2. The values for bending assume a strain rate of 0.10 in/in/sec in the low design pressure range and 0.30 in/in/sec in the high pressure design range. For tension and compression members, the DIF values assume the strain rates are 0.02 in/in/sec in the low design pressure range and 0.05 in/in/sec in the high design pressure range. Lower strain rates are selected for the tension and compression members since they are likely to carry the reaction of a beam or girder which may exhibit a significant rise time, thereby increasing the time to reach yield in the tension or compression mode.

On the basis of the above, the dynamic increase factors for yield stresses summarized in Table 5-2 are recommended for use in dynamic design. However, a more accurate representation may be derived using Figure 5-2 once the strain rate has been determined.

Steel protective structures and members are generally not designed for excessive deflections, that is, deflections associated with elongations well into the strain-hardening region (see Figure 5-1). However, situations arise where excessive deflections may be tolerated and will not lead to structural failure or collapse. In this case, the ultimate stresses and associated dynamic increase factors for ultimate stresses must be considered. Table 5-3 lists the dynamic increase factors for ultimate stresses of steels. Unlike the dynamic increase factors for yield stress, these values are independent of the pressure ranges.

5-13. Recommended Dynamic Design Stresses

5-13.1. General

The yield point of steel under uniaxial tensile stress is generally used as a base to determine yield stresses under other loading states namely, bending, shear and compression, or tension. The design stresses are also functions of the average strength increase factor, a, and the dynamic increase factor, c.

5-13.2. Dynamic Design Stress for Ductility Ratio $\mu \leq 10$

To determine the plastic strength of a section under dynamic loading, the appropriate dynamic yield stress, f_{dy}, must be used. For a ductility ratio (see Section 5-16.3) $\mu \leq 10$, the dynamic design stress, f_{ds}, is equal to the dynamic yield stress, f_{dy}. In general terms, the dynamic yield stress, f_{dy}, shall be equal to the product of the dynamic increase factor, c, the average yield strength increase factor, a, (see Section 5-12.1) and the specified minimum yield stress of the steel. The dynamic design stress, f_{ds}, for bending, tension, and compression shall be:

$$f_{ds} = f_{dy} = c \times a \times f_y \qquad 5\text{-}2$$

where

f_{dy} = dynamic yield stress

c = dynamic increase factor on the yield stress (Figure 5-2 or Table 5-2)

a = average strength increase factor (= 1.1 for steels with a specified minimum yield stress of 50 ksi or less; = 1.0 otherwise)

f_y = static yield stress from Table 5-1

5-13.3. Dynamic Design Stress for Ductility Ratio, μ > 10

Where excessive deflections or ductility ratios may be tolerated, the dynamic design stress can be increased to account for deformations in the strain-hardening region. In this case, for $\mu > 10$, the dynamic design stress, f_{ds}, becomes

$$f_{ds} = f_{dy} + (f_{du} - f_{dy})/4 \qquad 5\text{-}3$$

where:

f_{dy} = dynamic yield stress from Equation 5-2

f_{du} = dynamic ultimate stress equal to the product of f_u from Table 5-1 and the value of c from Table 5-3 or Figure 5-2

It should be noted that the average strength increase factor, a, does not apply to f_{du}.

5-13.4. Dynamic Design Stress for Shear

The dynamic design stress for shear shall be:

$$f_{dv} = 0.55\ f_{ds} \qquad 5\text{-}4$$

where f_{ds} is from Equation 5-2 or 5-3.

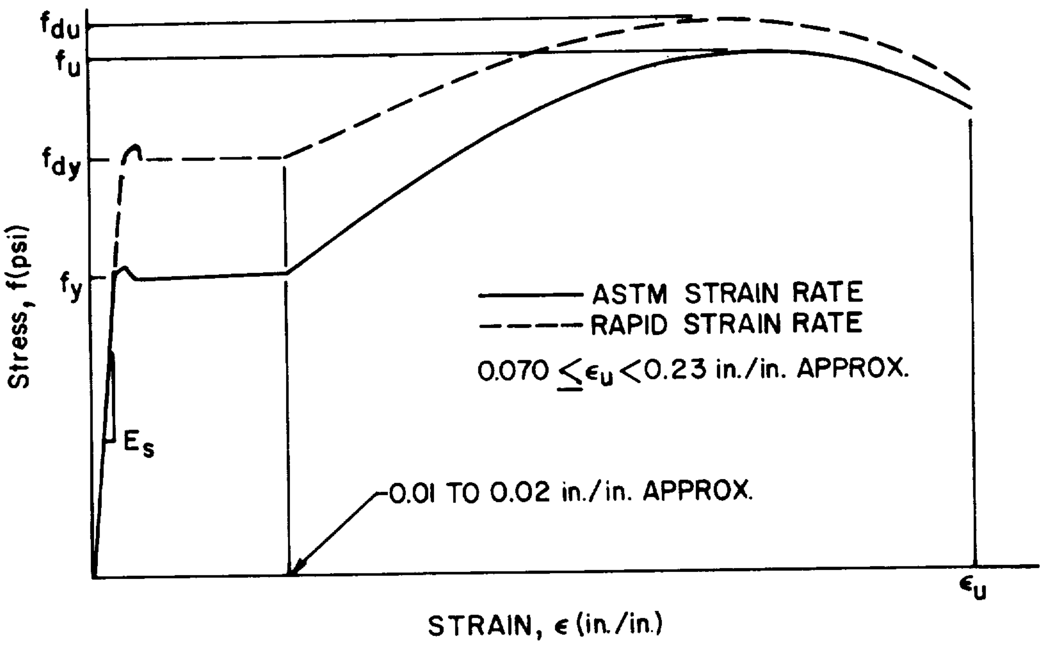


Figure 5-1 Typical stress-strain curves for steel

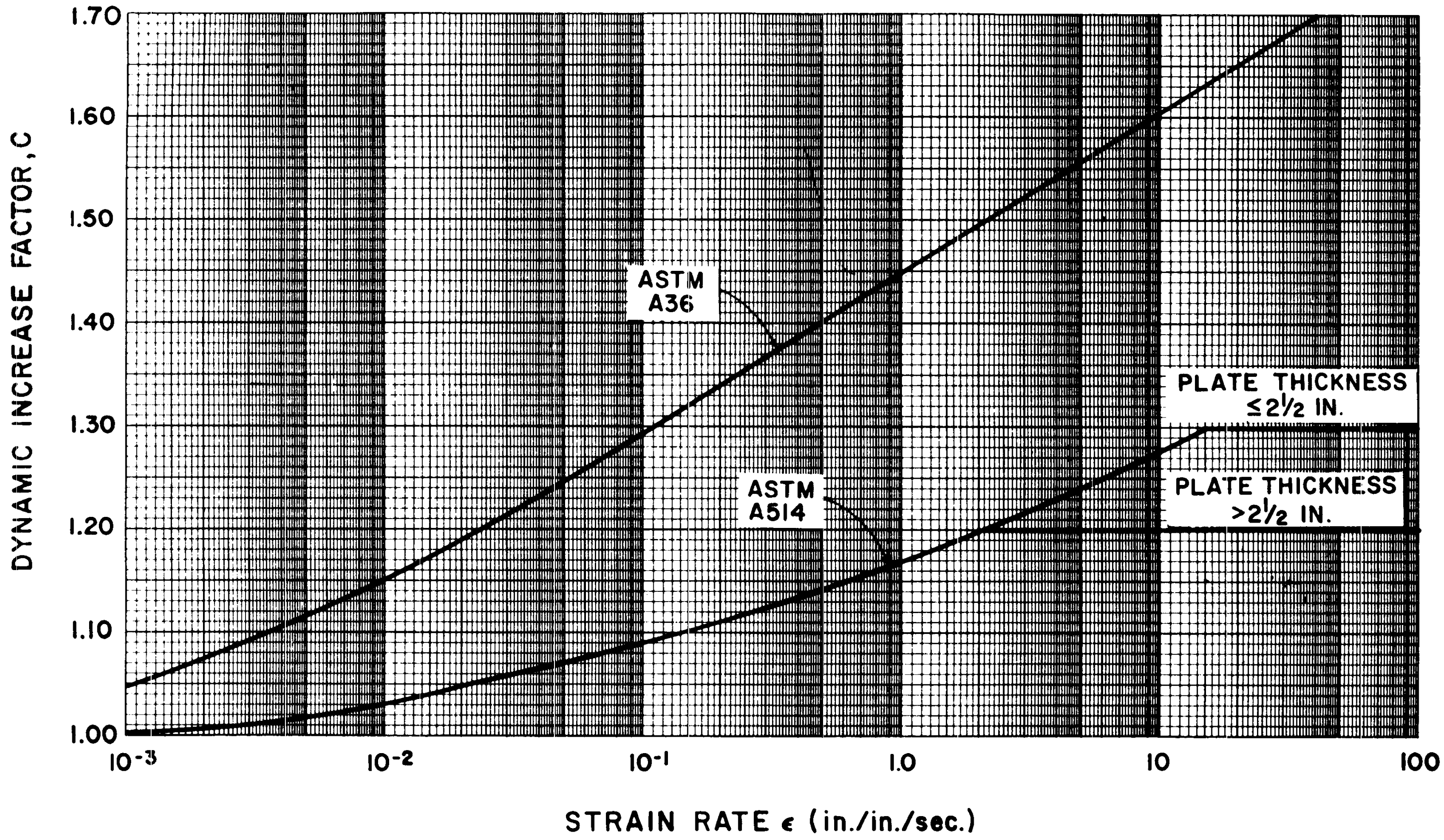


Figure 5-2 Dynamic increase factors for yield stresses at various strain rates for ASTM A-36 and A-514 steels

Table 5-1 Static Design Stresses for Materials

Material (ASTM)	f_y min (ksi)	f_u min (ksi)
A36	36	58
A529	42	60
A441	40	60
	42	63
	46	67
	50	70
A572	42	60
	50	65
	60	75
	65	80
A242	42	63
	46	67
	50	70
A588	42	63
	46	67
	50	70
A514	90	100
	100	110

Table 5-2 **Dynamic Increase Factor, c, for Yield Stress of Structural Steels**

Material	Bending		Tension or Compression	
	Low Pressure ($\dot{\epsilon}$ = 0.10 in/in/sec)	High Pressure ($\dot{\epsilon}$ = 0.30)	Low Pressure ($\dot{\epsilon}$ = 0.02)	High Pressure ($\dot{\epsilon}$ = 0.05)
A36	1.29	1.36	1.19	1.24
A588	1.19*	1.24*	1.12*	1.15*
A514	1.09	1.12	1.05	1.07

*Estimated

Table 5-3 **Dynamic Increase Factor, c, for Ultimate Stress of Structural Steels**

Material	c
A36	1.10
A588	1.05*
A514	1.00

*Estimated

DYNAMIC RESPONSE OF STEEL STRUCTURES IN THE PLASTIC RANGE

5-14. Plastic Behavior of Steel Structures

Although plastic behavior is not generally permissible under service loading conditions, it is quite appropriate for design when the structure is subjected to a severe blast loading only once or at most a few times during its existence. Under blast pressures, it will usually be uneconomical to design a structure to remain elastic and, as a result, plastic behavior is normally anticipated in order to utilize more fully the energy-absorbing capacity of blast-resistant structures. Plastic design for flexure is based on the assumption that the structure or member resistance is fully developed with the formation of near totally plastified sections at the most highly stressed locations. For economical design, the structure should be proportioned to assure its ductile behavior up to the limit of its load-carrying capacity. The structure or structural element can attain its full plastic capacity provided that premature impairment of strength due to secondary effects, such as brittle fracture or instability, does not occur.

Structural resistance is determined on the basis of plastic design concepts, taking into account dynamic yield strength values. The design proceeds with the basic objective that the computed deformations of either the individual members or the structure as a whole, due to the anticipated blast loading, should be limited to prescribed maximum values consistent with safety and the desired post accident condition.

5-15. Relationship Between Structure Function and Deformations

5-15.1. General

Deformation criteria are specified in detail for two categories of structures, namely, acceptor-type structures in the low pressure range and structures in the high pressure range which may either be acceptor- or donor-type. A description of the two categories of structures follow.

5-15.2. Acceptor-type Structures in the Low Pressure Ranges

The maximum deformations to be specified in this category are consistent with maintaining structural integrity into the plastic range while providing safety for personnel and equipment. The type of structure generally associated with this design category may be constructed of one or two stories with braced or rigid frames. Main members consisting of columns and main beams should be fabricated from hot rolled steel while secondary members, consisting of purlins or girts which span the frame members, can be hot-rolled I-shapes and channels or cold-formed Z-shapes and channels. The structure skin shall consist of cold-formed siding and decking spanning between the wall girts or roof purlins.

5-15.3. Acceptor- or Donor-type Structures in the High Pressure Range

The deformation criteria specified in this category cover the severe conditions associated with structures located close-in to a blast. In cases where the design objective is the containment of an explosion the deformations should be limited. In other cases where prevention of explosion propagation or of missile generation is required, the structure may be allowed to approach incipient failure, and deformations well into the strain hardening region may be permitted for energy absorption. In general, plate elements and curved plate-type structures fall under these categories.

5-16. Deformation Criteria

5-16.1. General

The deformation criteria presented in this chapter will be consistent with designing the structure for one accident. However, if it is desirable for a structure to sustain two or three "incidents" in its lifetime, the designer may limit design deformations so that, in its post accident condition, the structure is suitable for repair and reuse.

The deformation criteria for beams (including purlins, spandrels and girts) are presented in Section 5-16.5. The criteria for frames, including sidesway, are presented in Section 5-16.6 and that for plates are given in Section 5-16.7. Special consideration is given to the deformation criteria for open-web joists (Section 5-33) and cold-formed metal decking (Section 5-34). Deformation criteria are summarized in Section 5-35.

5-16.2. Structural Response Quantities

In order to restrict damage to a structure or element which is subjected to the effects of accidental explosion, limiting values must be assigned to appropriate response quantities. Generally speaking, two different types of values are specified, namely, limits on the level of inelastic dynamic response and limits on the maximum deflections and rotations.

For elements which can be represented as single-degree-of-freedom systems such as beams, floor and wall panels, open-web joists, and plates, the appropriate quantities are taken as the maximum ductility ratio and the maximum rotation at an end support.

For systems such as frame structures which can be represented by multi-degree-of-freedom systems, the appropriate quantities are taken as the sidesway deflection and individual frame member rotations.

5-16.3. Ductility Ratio, μ

Following the development in Chapter 3 of this manual, the ductility ratio, μ, is defined as the ratio of the maximum deflection (X_m) to the equivalent elastic deflection (X_E) corresponding to the development of the limiting resistance on the bilinear resistance diagram for the element. Thus, a ductility ratio of 3 corresponds to a maximum dynamic response three times the equivalent elastic response.

In the case of individual beam elements, ductility ratios as high as 20 can be achieved provided that sufficient bracing exists. Subsequent sections of this chapter cover bracing requirements for beam elements. In the case of plate elements, ductility ratios are important insomuch as the higher ductility ratios permit the use of higher design stresses.

Support rotations, as discussed in the next paragraph, provide the basis for beam and plate design. For a beam element, the ductility ratio must be checked to determine whether the specified rotation can be reached without premature buckling of the member. A similar provision shall apply to plates even though they may undergo larger ductility ratios in the absence of premature buckling.

5-16.4. Support Rotation, θ

The end rotation, θ, and the associated maximum deflection, X_m, for a beam are illustrated in Figure 5-3. As shown, θ is the angle between the chord joining the supports and the point on the element where the deflection is a maximum.

5-16.5. Limiting Deformations for Beams

A steel beam element may be designed to attain large deflections corresponding to 12 degrees support rotation. To assure the integrity of the beam element, it must be adequately braced to permit this high level of ductile behavior. In no case, however, shall the ductility ratio exceed 20.

A limiting support rotation of 2 degrees, and a limiting ductility ratio of 10 (whichever governs) are specified as reasonable estimates of the absolute magnitude of the beam deformation where safety for personnel and equipment is required. These deformations are consistent with maintaining structural integrity into the plastic range. Adequate bracing shall be present to assure the corresponding level of ductile behavior.

The interrelationship between the various parameters involved in the design of beams is readily described in the idealized resistance-deflection curve shown in Figure 5-4. In the figure, the values shown for the ductility ratio, μ, and the support rotation θ, are arbitrary. For example, the deflection corresponding to a 2-degree support rotation can be greater than that corresponding to a ductility ratio of 10.

5-16.6. Application of Deformation Criteria to a Frame Structure

In the detailed analysis of a frame structure, representation of the response by a single quantity is not possible. This fact combined with the wide range and time-varying nature of the end conditions of the individual frame members makes the concept of ductility ratio intractable. Hence, for this case, the response quantities referred to in the criteria are the sidesway deflection of each story and the end rotation, θ, of the individual members with reference to a chord joining the member ends, as illustrated in Figure 5-3. In addition, in lieu of a ductility ratio criterion, the amount of inelastic deformation is restricted by means of a limitation on the individual member rotation. For members which are not loaded between their ends, such as an interior column, 0 is zero and only the sidesway criteria must be considered. The maximum member end rotation, as shown in Figure 5-3, shall be 2 degrees. The maximum sidesway deflection is limited to 1/25 of the story height.

These response quantities, sidesway deflection, and end rotation are part of the required output of various computer programs which perform an inelastic, multi-degree-of-freedom analysis of frame structures. These programs are available through the repositories listed in Section 5-4 and several reports listed in the bibliography at the end of this chapter. The designer can use the output of these programs to check the sidesway deflection of each story and the maximum rotation of each member.

5-16.7. Limiting Deformations for Plates

Plates and plate-type structures can undergo large deformations with regard to support rotations and ductility ratios. The effect of overall and local instability upon the ultimate capacity is considerably more important to structural steel shapes than to plates. Depending upon the functional requirements for a plate, the following deformation criteria should be considered in the design of a plate:

(1) Large deflections at or close to incipient failure.

(2) Moderate deflections where the structure is designed to sustain two or three "incidents" before being nonreusable.

(3) Limited deflections where performance of the structure is critical during the blast as in the case of a blast door designed to contain pressure and/or fire leakage.

(4) Elastic deflections where the structure must not sustain permanent deflections, as in the case of an explosives test chamber.

This is a partial list of design considerations for plates. It can be seen that the designer must establish deformation criteria based on the function of the plate or plate system.

A plate or plate-type structure may undergo a support rotation, as illustrated in Figure 3-22 of Chapter 3, of 12 degrees. The corresponding allowable ductility ratio shall not exceed 20. It should be noted that higher design stresses can be utilized when the ductility ratio exceeds 10 (See Section 5-13.3).

A limiting support rotation of 2 degrees is specified as a reasonable estimate of the absolute magnitude of the plate support rotation where safety for personnel and equipment in an acceptor-type structure is required. As in the deformation criteria for beams, the ductility ratio shall not exceed 10.

Two edge conditions may govern the deformation of plates in the plastic region. The first occurs when opposite edges are not built-in. In this case, elastic plate deflection theory and yield-line theory apply. The second involves tension-membrane action which occurs when at least two opposite edges are clamped. In this case, tensile-membrane action can occur before the plate element reaches a support rotation of 12 degrees. Tensile-membrane action of built-in plates is not covered in this chapter. However, the designer can utilize yield-line theory for limited plate deflection problems.

The interrelationship between the various parameters involved in the design of plates is readily described in the idealized resistance-deflection curve shown in Figure 5-5. The figure shows the values for the ductility ratio, μ, and the support rotation, θ, are arbitrary. For example, the deflection corresponding to a 2-degree support rotation can be greater than that corresponding to a ductility ratio of 10.

5-17. Rebound

Another aspect of dynamic design of steel structures subjected to blast loadings is the occurrence of rebound. Unlike the conditions prevailing in reinforced concrete structures where rebound considerations may not be of primary concern, steel structures will be subjected to relatively large stress reversals caused by rebound and will require lateral bracing of unstayed compression flanges which were formerly in tension. Rebound is more critical for elements supporting light dead loads and subjected to blast pressures of short duration. It is also a primary concern in the design of reversal bolts for blast doors.

5-18. Secondary Modes of Failure

5-18.1. General

In the process of designing for the plastic or ductile mode of failure, it is important to follow certain provisions in order to avoid premature failure of the structure, i.e., to ensure that the structure can develop its full plastic resistance.

These secondary modes of failure can be grouped in two main categories:

(1) Instability modes of failure

(2) Brittle modes of failure

5-18.2. Instability Modes of Failure

In this category, the problem of structural instability at two levels is of concern, namely, overall buckling of the structural system as a whole, and buckling of the component elements.

Overall buckling of framed structures can occur in two essentially different manners. In the first case, the load and the structure are symmetric; deformations remain also symmetric up to a critical value of the load for which a sudden change in configuration will produce instant anti-symmetry, large sidesway and displacement, and eventually a failure by collapse if not by excessive deformations. This type of instability can also occur in the elastic domain before substantial deformation or any plastification has taken place. It is called "instability by bifurcation".

In the second case, the loading or the structure or both are nonsymmetric. With the application of the load, sidesway develops progressively. In such cases, the vertical loads acting through the sidesway displacements, commonly called "the P-Δ effect", create second order bending moments that magnify the deformation. Because of rapidly increasing displacements, plastic hinges form, thereby decreasing the rigidity of the structure and causing more sidesway. This type of instability is related to a continuous deterioration of the stiffness leading to an early failure by either a collapse mechanism or excessive sidesway.

Frame instability need not be explicitly considered in the plastic design of one- and two-story unbraced frames provided that the individual columns and girders are designed according to the beam-column criteria of Section 5-37.

For frames greater than two stories, bracing is normally required according to the AISC provisions for plastic design in order to ensure the overall stability of the structure. However, if an inelastic dynamic frame analysis is performed to determine the complete time-history of the structural response to the blast loading, including the P-Δ effects, it may be established, in particular cases, that lateral bracing is not necessary in a frame greater than two stories. As mentioned previously, computer programs which perform an inelastic, multi-degree-of-freedom analysis of frame structures may be employed for such an analysis.

Buckling of an element in the structure (e.g., a beam, girder, or column) can occur under certain loading and end conditions. Instability is of two types, namely, buckling of the member as a whole (e.g., lateral torsional buckling) and local buckling at certain sections, including flange buckling and web crippling.

Provisions for plastic design of beams and columns are presented in a separate section of this chapter.

5-18.3. Brittle Modes of Failure

Under dynamic loading, there is an enhanced possibility that brittle fracture can develop under certain conditions. Since this type of failure is sudden in nature and difficult to predict, it is very important to diminish the risk of such premature failure.

The complexity of the brittle fracture phenomena precludes a complete quantitative definition. As a result, it is impossible to establish simple rules for design. Brittle fracture will be associated with a loss in flexural resistance.

Brittle fractures are caused by a combination of adverse circumstances that may include a few, some, or all of the following:

(1) Local stress concentrations and residual stresses

(2) Poor welding

(3) The use of a notch sensitive steel

(4) Shock loading or rapid strain rate

(5) Low temperatures

(6) Decreased ductility due to strain aging

(7) The existence of a plane strain condition causing a state of triaxial tension stresses, especially in thick gusset plates, thick webs and in the vicinity of welds

(8) Inappropriate use of some forms of connections

The problem of brittle fracture is closely related to the detailing of connections, a topic that will be treated in a separate section of this chapter.

However, there are certain general guidelines to follow in order to minimize the danger of brittle fracture:

(1) Steel material must be selected to conform with the condition anticipated in service.

(2) Fabrication and workmanship should meet high standards, e.g., sheared edges and notches should be avoided, and material that has been severely cold-worked should be removed.

(3) Proportioning and detailing of connections should be such that free movement of the base material is permitted, stress concentrations and triaxial stress conditions are avoided, and adequate ductility is provided.

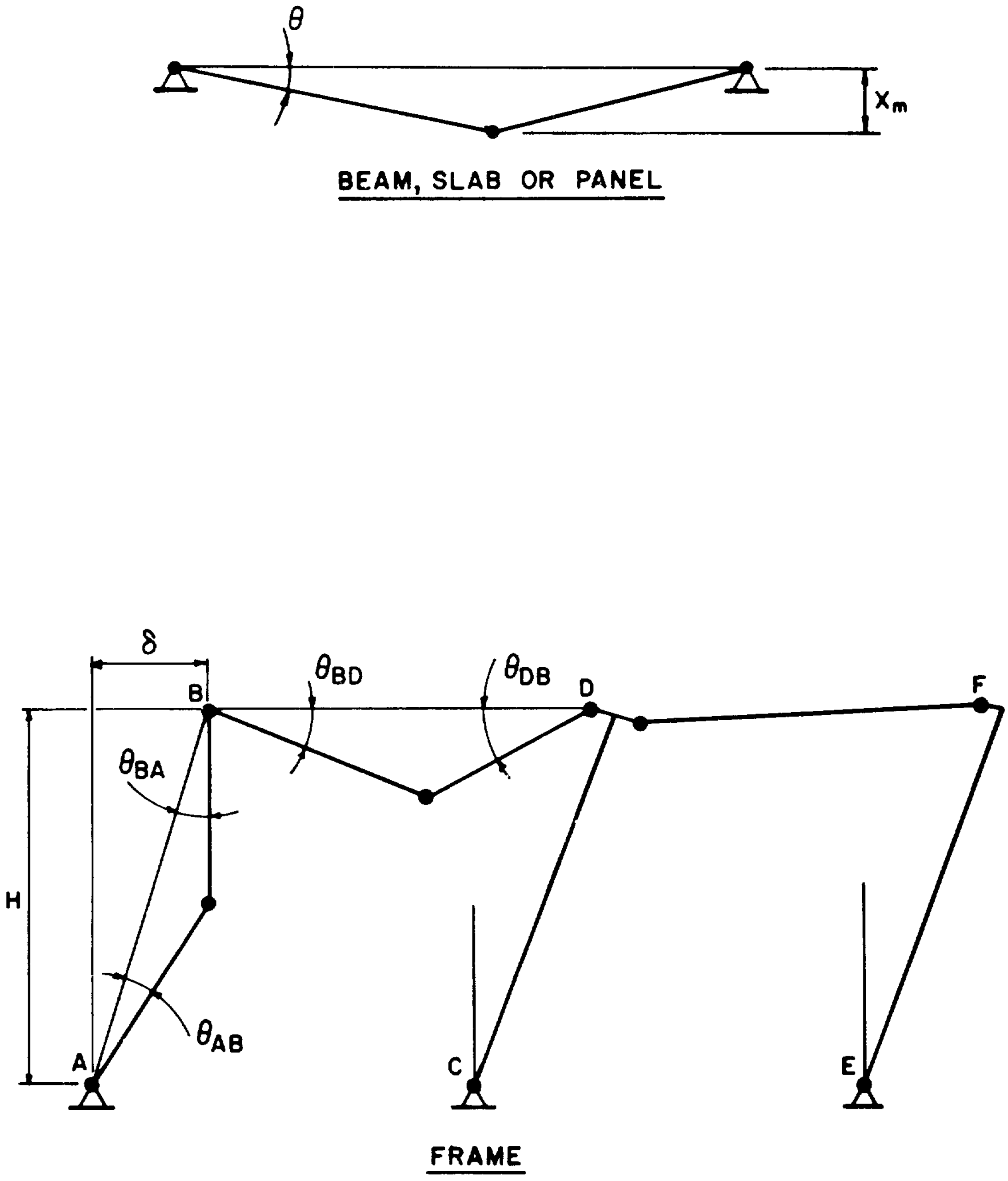


Figure 5-3 Member end rotations for beams and frames

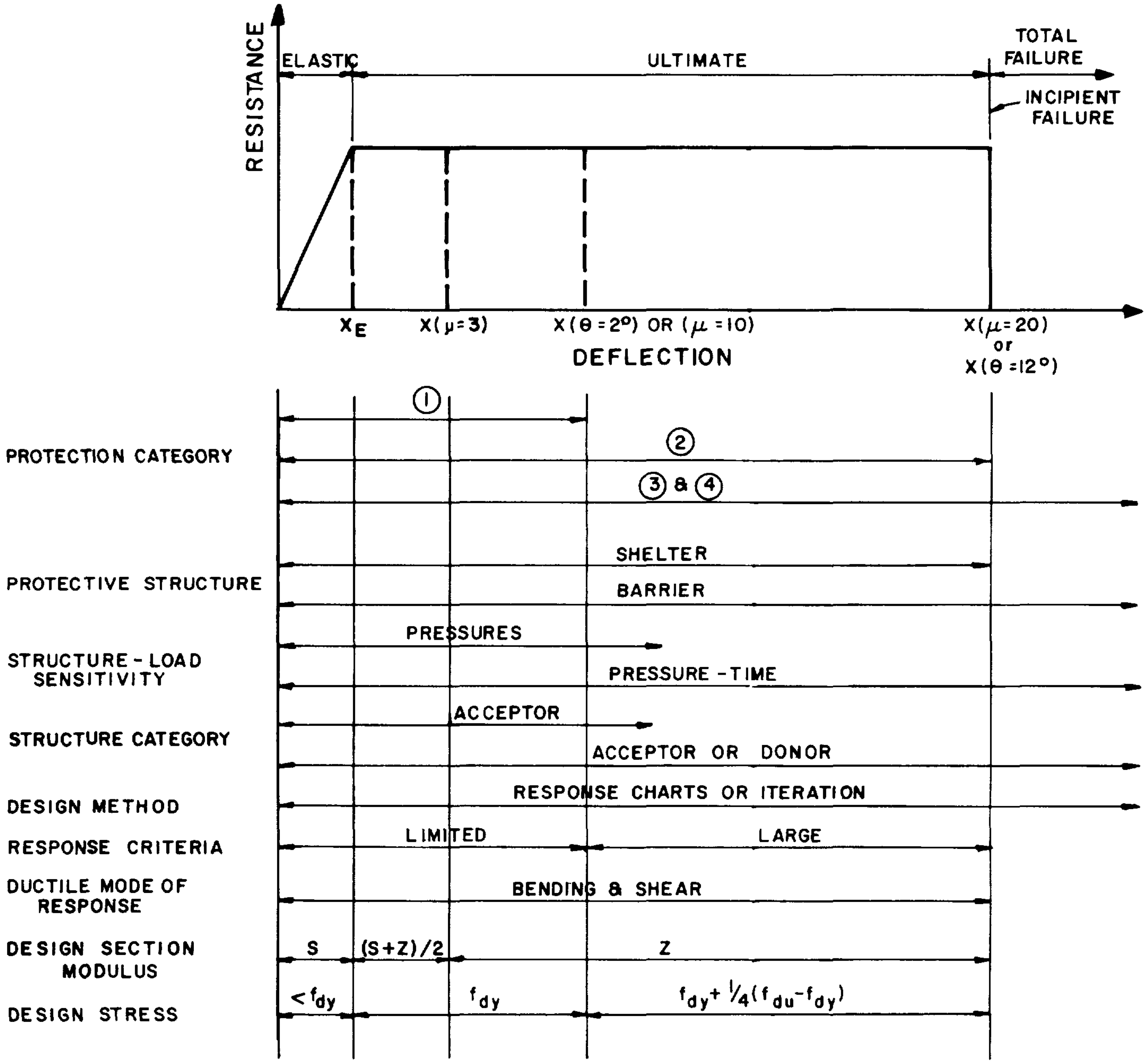


Figure 5-4 Relationships between design parameters for beams

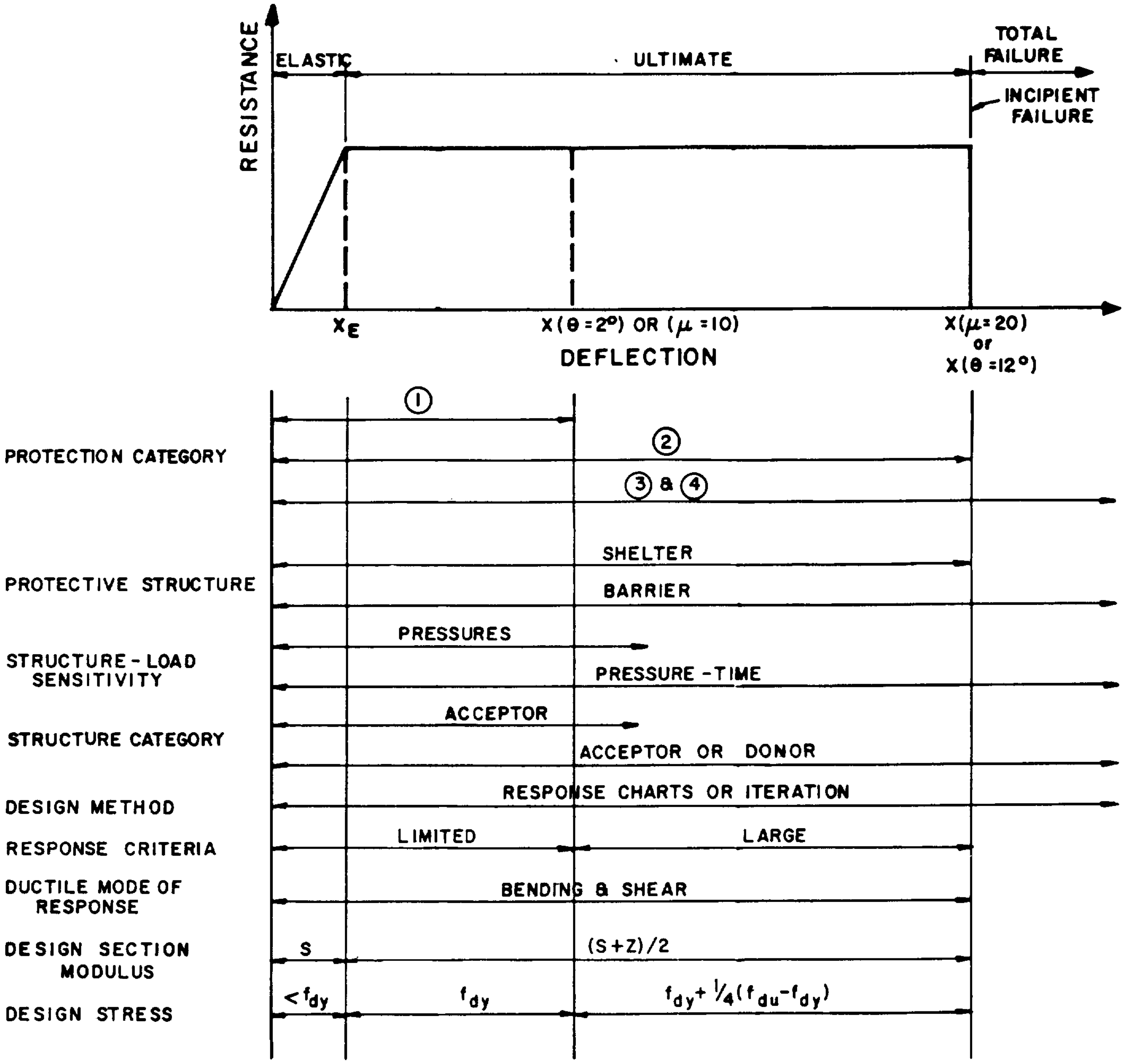


Figure 5-5 Relationships between design parameters for plates

DESIGN OF SINGLE SPAN AND CONTINUOUS BEAMS

5-19. General

The emphasis in this section is on the dynamic plastic design of structural steel beams. Design data have been derived from the static provisions of the AISC Specification with necessary modifications and additions for blast design. It should be noted that all provisions on plastic design in the AISC Specification apply, except as modified in this chapter.

The calculation of the dynamic flexural capacity of beams is described in detail. The necessary information is presented for determining the equivalent bilinear resistance-deflection functions used in evaluating the basic flexural response of beams. Also presented are the supplementary considerations of adequate shear capacity and local and overall stability which are necessary for the process of hinge formation, moment redistribution and inelastic hinge rotation to proceed to the development of a full collapse mechanism.

5-20. Dynamic Flexural Capacity

5-20.1. General

The ultimate dynamic moment resisting capacity of a steel beam is given by

$$M_{pu} = f_{ds} Z \qquad \text{5-5}$$

where f_{ds} is the dynamic design stress (as described in Section 5-13) of the material and Z is the plastic section modulus. The plastic section modulus can be calculated as the sum of the static moments of the fully yielded elements of the equal cross section areas above and below the neutral axis, i.e.:

$$Z = A_c m_1 + A_t m_2 \qquad \text{5-6}$$

Note: $A_c m_1 = A_t m_2$ for a doubly-symmetric section

where
- A_c = area of cross section in compression
- A_t = area of cross section in tension
- m_1 = distance from neutral axis to the centroid of the area in compression
- m_2 = distance from neutral axis to the centroid of the area in tension

For standard I-shaped sections (S, W, and M shapes), the plastic section modulus is approximately 1.15 times the elastic modulus for strong axis bending and may be obtained from standard manuals on structural steel design.

It is generally assumed that a fully plastic section offers no additional resistance to load. However, additional resistance due to strain hardening of the material is present as the deformation continues beyond the yield level of the beam. In the analysis of structural steel beams, it is assumed that the plastic hinge formation is concentrated at a section. Actually, the plastic region extends over a certain length that depends on the type of loading (concentrated or distributed) on the magnitude of the deformation, and on the shape factor of the cross section. The extent of the plastic hinge has no substantial influence on the ultimate capacity; it has, however, an influence on the final magnitude of the deflection. For all practical purposes, the assumption of a concentrated plastic hinge is adequate.

In blast design, although strains well into the strain-hardening range may be tolerated, the corresponding additional resistance is generally not sufficient to warrant analytical consideration since excessive support rotation and/or ductility ratios of beams, which are susceptible to local flange or lateral torsional buckling, are not recommended.

5-20.2. Moment-curvature Relationship for Beams

Figure 5-6 shows the stress distribution at various stages of deformation for a plastic hinge section. Theoretically, the beam bends elastically until the outer fiber stress reaches f_{ds} and the yield moment designated by M_y is attained (Figure 5-6a). As the moment increases above M_y, the yield stress progresses inward from the outer fibers of the section towards the neutral axis as shown in Figure 5-6b. As the moment approaches the fully plastic moment, a rectangular stress distribution as shown in Figure 5-6c is approached. The ratio between the fully plastic moment to the yield moment is the shape factor, f, for the section, i.e., the ratio between the plastic and elastic section modulii.

A representative moment-curvature relationship for a simply-supported steel beam is shown in Figure 5-7. The behavior is elastic until the yield moment My is reached. With further increase in load, the curvature increases at a greater rate as the fully plastic moment value, M_2, is approached. Following the attainment of M_2, the curvature increases significantly, with only a small increase in moment capacity.

For design purposes, a bilinear representation of the moment-curvature relationship is employed as shown by the dashed lines in Figure 5-7. For beams with a moderate design ductility ratio ($\mu \leq 3$), the design moment $M_p = M_1$. For beams with a larger design ductility ratio ($\mu > 3$), the design moment $M_p = M_2$.

5-20.3. Design Plastic Moment, M_p

The equivalent plastic design moment shall be computed as follows:

For beams with ductility ratios less than or equal to 3:

$$M_p = f_{ds} (S + Z)/2 \qquad 5\text{-}7$$

where S and Z are the elastic and plastic section modulii, respectively.
For beams with ductility ratios greater than 3 and beam columns:

$$M_p = f_{ds}Z \qquad 5\text{-}8$$

Equation 5-7 is consistent with test results for beams with moderate ductilities. For beams which are allowed to undergo large ductilities, Equation 5-8, based upon full plastification of the section, is considered reasonable for design purposes.

It is important to note that the above pertains to beams which are supported against buckling. Design provisions for guarding against local and overall buckling of beams during plastic deformation are discussed in Sections 5-24, 5-25, and 5-26.

5-21. Resistance and Stiffness Functions

5-21.1. General

The single-degree-of-freedom analysis which serves as the basis for the flexural response calculation requires that the equivalent stiffness and ultimate resistance be defined for both single-span beams and continuous beams. The ultimate resistance values correspond to developing a full collapse mechanism in each case. The equivalent stiffnesses correspond to load-deflection relationships that have been idealized as bilinear functions with initial slopes so defined that the areas under the idealized load deflection diagrams are equal to the areas under the actual diagrams at the point of inception of fully plastic behavior of the beam. This concept is covered in Section 3-13 of Chapter 3.

5-21.2. Single-span Beams

Formulas for determining the stiffness and resistance for one-way steel beam elements are presented in Tables 3-1 and 3-8 of Chapter 3. The values of M in Table 3-1 represents the plastic design moment, M_p. For example, the value of r_u for the fixed-simple, uniformly loaded beam becomes $r_u = 12\ M_p/L^2$.

5-21.3. Multi-span Beams

The beam relationships for defining the bilinear resistance function for multi-span continuous beams under uniform loading are summarized below. These expressions are predicated upon the formation of a three-hinge mechanism in each span. Maximum economy normally dictates that the span lengths and/or member sizes be adjusted such that a mechanism forms simultaneously in all spans.

It must be noted that the development of a mechanism in a particular span of a continuous beam assumes compatible stiffness properties at the end supports. If the ratio of the length of the adjacent spans to the span being considered is excessive (say, greater than three). it may not be possible to reach the limit load without the beam failing by excessive deflection.

For uniformly distributed loading on equal spans or spans which do not differ in length by more than 20 percent, the following relationships can be used to define the bilinear resistance function:

Two-span continuous beam:

$$R_u = r_u bL = 12\ M_p/L \qquad 5\text{-}9$$

$$K_E = 163\ EI/L^3 \qquad 5\text{-}10$$

Exterior span of continuous beams with three or more spans:

$$R_u = r_u bL = 11.7 M_p/L \qquad 5\text{-}11$$

$$K_E = 143EI/L^3 \quad \text{5-12}$$

Interior span of continuous beam with three or more spans:

$$R_u = r_u bL = 16.0M_p/L \quad \text{5-13}$$

$$K_E = 300EI/L^3 \quad \text{5-14}$$

For design situations which do not meet the required conditions, the bilinear resistance function may be developed by the application of the basic procedures of plastic analysis.

5-22. Design for Flexure

5-22.1. General

The design of a structure to resist the blast of an accidental explosion consists essentially of the determination of the structural resistance required to limit calculated deflections to within the prescribed maximum values as outlined in Section 5-35. In general, the resistance and deflection may be computed on the basis of flexure provided that the shear capacity of the web is not exceeded. Elastic shearing deformations of steel members are negligible as long as the depth to span ratio is less than about 0.2 and hence, a flexural analysis is normally sufficient for establishing maximum deflections.

5-22.2. Response Charts

Dynamic response charts for one-degree-of-freedom systems in the elastic or elasto-plastic range under various dynamic loads are given in Chapter 3. To use the charts, the effective natural period of vibration of a structural steel beam must be determined. The procedures for determining the natural period of vibration for one-way elements are outlined in Section 3-17.4 of Chapter 3. Equation 5-15 can be used to determine the natural period of vibration for any system for which the total effective mass, M_e, and equivalent elastic stiffness, K_E are known:

$$T_N = 2\pi\ (M_e\ /K_E)^{1/2} \quad \text{5-15}$$

5-22.3. Preliminary Dynamic Load Factors

For preliminary flexural design of beams situated in low pressure range, it is suggested that an equivalent static ultimate resistance equal to the peak blast pressure be used for those beams designed for 2 degrees support rotation. For large support rotations, a preliminary dynamic load factor of 0.5 is recommended. Since the duration of the loading for low pressure range will generally be the same or longer than the period of vibration of the structure, revisions to this preliminary design from a dynamic analysis will usually not be substantial. However, for structures where the loading environment pressure is such that the load duration is short as compared with the period of vibration of the structure, this procedure may result in a substantial overestimate of the required resistance.

5-22.4. Additional Considerations in Flexural Design

Once a dynamic analysis is performed on the single span or continuous beam, the deformations must be checked with the limitations set in the criteria. The provisions for local buckling, web crippling and lateral bracing must be met. The deformation criteria for beam elements including purlins, spandrels and girts are summarized in Section 5-35.

The rebound behavior of the structure must not be overlooked. The information required for calculating the elastic rebound of structures is contained in Figure 3-268 of Chapter 3. The provisions for local buckling and lateral bracing, as outlined in subsequent sections of this chapter, shall apply in the design for rebound.

5-23. Design for Shear

Shearing forces are of significance in plastic design primarily because of their possible influence on the plastic moment capacity of a steel member. At points where large bending moments and shear forces exist, the assumption of an ideal elasto-plastic stress-strain relationship indicates that during the progressive formation of a plastic hinge, there is a reduction of the web area available for shear. This reduced area could result in an initiation of shear yielding and possibly reduce the moment capacity.

However, it has been found experimentally that I-shaped sections achieve their fully plastic moment capacity provided that the average shear stress over the full web area is less than the yield stress in shear. This result can basically be attributed to the fact that I-shaped sections carry moment predominantly through the flanges and shear predominantly through the web. Other contributing factors include the beneficial effects of strain hardening and the fact that combinations of high shear and high moment generally occur at locations where the moment gradient is steep.

The yield capacity of steel beams in shear is given by:

$$V_p = f_{dv} A_w \tag{5-16}$$

where V_p is the shear capacity, f_{dv} is the dynamic yield stress in shear of the steel (Equation 5-4), and A_w is the area of the web. For I-shaped beams and similar flexural members with thin webs, only the web area between flange plates should be used in calculating A_w.

For several particular load and support conditions, equations for the support shears, V, for one-way elements are given in Table 3-9 of Chapter 3. As discussed above, as long as the acting shear V does not exceed V_p, I-shaped sections can be considered capable of achieving their full plastic moment. If V is greater than V_p, the web area of the chosen section is inadequate and either the web must be strengthened or a different section should be selected.

However, for cases where the web is being relied upon to carry a significant portion of the moment capacity of the section, such as rectangular cross section beams or built-up sections, the influence of shear on the available moment capacity must be considered as treated in Section 5-31.

5-24. Local Buckling

In order to ensure that a steel beam will attain fully plastic behavior and attain the desired ductility at plastic hinge locations, it is necessary that the elements of the beam section meet minimum thickness requirements sufficient to prevent a local buckling failure. Adopting the plastic design requirements of the AISC Specification, the width-thickness ratio for flanges of rolled I- and W-shapes and similar built-up single web shapes that would be subjected to compression involving plastic hinge rotation shall not exceed the following values:

f_y (ksi)	$b_f/2t_f$
36	8.5
42	8.0
45	7.4
50	7.0
55	6.6
60	6.3
65	6.0

where f_y is the specified minimum static yield stress for the steel (Table 5-1), b_f is the flange width, and t_f is the flange thickness.

The width-thickness ratio of similarly compressed flange plates in box sections and cover plates shall not exceed $190/(f_y)^{1/2}$. For this purpose, the width of a cover plate shall be taken as the distance between longitudinal lines of connecting rivets, high-strength bolts, or welds.

The depth-thickness ratio of webs of members subjected to plastic bending shall not exceed the value given by Equation 5-17 or 5-18 as applicable.

$$\frac{d}{t_w} = \frac{412}{f_y}\left[1 - 1.4\frac{P}{P_y}\right] \quad \text{when} \quad \frac{P}{P_y} \leq 0.27 \qquad \text{5-17}$$

$$\frac{d}{t_w} = \frac{257}{f_y} \quad \text{when} \quad \frac{P}{P_y} > 0.27 \qquad \text{5-18}$$

where

P = the applied compressive load

P_y = the plastic axial load equal to the cross sectional area times the specified minimum static yield stress, f_y

These equations which are applicable to local buckling under dynamic loading have been adopted from the AISC provisions for static loading. However, since the actual process of buckling takes a finite period of time, the member must accelerate laterally and the mass of the member provides an inertial force retarding this acceleration. For this reason, loads that might otherwise cause failure may be applied to the members for very short durations if they are removed before the buckling has occurred. Hence, it is appropriate and conservative to apply the criteria developed for static loads to the case of dynamic loading of relatively short duration.

These requirements on cross section geometry should be adhered to in the design of all members for blast loading. However, in the event that it is necessary to evaluate the load-carrying capacity of an existing structural member which does not meet these provisions, the ultimate capacity should be reduced in accordance with the recommendations made in the Commentary and Appendix C of the AISC Specification.

5-25. Web Crippling

Since concentrated loads and reactions along a short length of flange are carried by compressive stresses in the web of the supporting member, local yielding may occur followed by crippling or crumpling of the web. Stiffeners bearing against the flanges at load points and fastened to the web are usually employed in such situations to provide a gradual transfer of these forces to the web.

Provisions for web stiffeners, as given in Section 1.15.5 of the AISC Specification, should be used in dynamic design. In applying these provisions, f_y should be taken equal to the specified static yield strength of the steel.

5-26. Lateral Bracing

5-26.1. General

Lateral bracing support is often provided by floor beams, joists or purlins which frame into the member to be braced. The unbraced lengths (l_{cr}, as defined in Sections 5-26.2 and 5-26.3) are either fixed by the spacing of the purlins and girts or by the spacing of supplementary bracing.

When the compression flange is securely connected to steel decking or siding, this will constitute adequate lateral bracing in most cases. In addition, inflection points (points of contraflexure) can be considered as braced points.

Members built into a masonry wall and having their web perpendicular to this wall can be assumed to be laterally supported with respect to their weak axis of bending. In addition, points of contraflexure can be considered as braced points, if necessary.

Members subjected to bending about their strong axis may be susceptible to lateral-torsional buckling in the direction of the weak axis if their compression flange is not laterally braced. Therefore, in order for a plastically designed member to reach its collapse mechanism, lateral supports must be provided at the plastic hinge locations and at a certain distance from the hinge location. Rebound, which constitutes stress reversals, is an important consideration for lateral bracing support.

5-26.2. Requirements for Members with $\mu \leq 3$

Since members with the design ductility ratios less than or equal to three undergo moderate amounts of plastic deformation, the bracing requirements are somewhat less stringent.

For this case, the following relationship may be used:

$$1/r_T = \left[(102 \times 10^3 C_b)/f_{ds}\right]^{1/2} \qquad 5\text{-}19$$

where

- l = distance between cross sections braced against twist or lateral displacement of the compression flange
- r_T = radius of gyration of a section comprising the compression flange plus one-third of the compression web area taken about an axis in the plane of the web
- C_b = bending coefficient defined in Section 1.5.1.4.5 of the AISC Specification

5-26.3. Requirements for Members with $\mu > 3$

In order to develop the full plastic moment, M_p for members with design ductility ratios greater than three, the distance from the brace at the hinge location to the adjacent braced points should not be greater than l_{cr} as determined from either Equation 5-20 or 5-21, as applicable:

$$\beta \frac{l_{cr}}{r_y} = \frac{1375}{f_{ds}} + 25 \text{ when } 1.0 \geq \frac{M}{M_p} > -0.5 \qquad 5\text{-}20$$

$$\beta \frac{l_{cr}}{r_y} = \frac{1375}{f_{ds}} \text{ when } -0.5 \geq \frac{M}{M_p} \geq -1.0 \qquad 5\text{-}21$$

where

- r_y = the radius of gyration of the member about its weak axis
- M = the lesser of the moments at the ends of the unbraced segment
- M/M_p = the end moment ratio. The moment ratio is positive when the segment is bent in reverse curvature and negative when bent in single curvature.
- β = critical length correction factor (See Figure 5-8)

The critical length correction factor, β, accounts for the fact that the required spacing of bracing, l_{cr}, decreases with increased ductility ratio. For example, for a particular member with r_y = 2 in. and f_{ds} = 51 ksi and using the equation for M/M_p = 0, we get l_{cr} = 71.7 in. for μ = 6 and l_{cr} = 39.7 in. for μ = 20.

5-26.4. Requirements for Elements Subjected to Rebound

The bracing requirements for nonyielded segments of members and the bracing requirements for members in rebound can be determined from the following relationship:

$$f = 1.67 \left[2/3 - \frac{f_{ds} \, (1/r_T)^2}{1530 \times 10^3 \, C_b} \right] f_{ds} \qquad 5\text{-}22$$

where

f = the maximum bending stress in the member, and in no case greater than f_{ds}

When f equals f_{ds}, this equation reduces to the $1/r_T$ requirement of Equation 5-19.

5-26.5. Requirements for Bracing Members

In order to function adequately, the bracing member must meet certain minimum requirements on axial strength and axial stiffness. These requirements are quite minimal in relation to the properties of typical framing members.

Lateral braces should be welded or securely bolted to the compression flange and, in addition, a vertical stiffener should generally be provided at bracing points where concentrated vertical loads are also being transferred. Plastic hinge locations within uniformly loaded spans do not generally require a stiffener. Examples of lateral bracing details are presented in Figure 5-9.

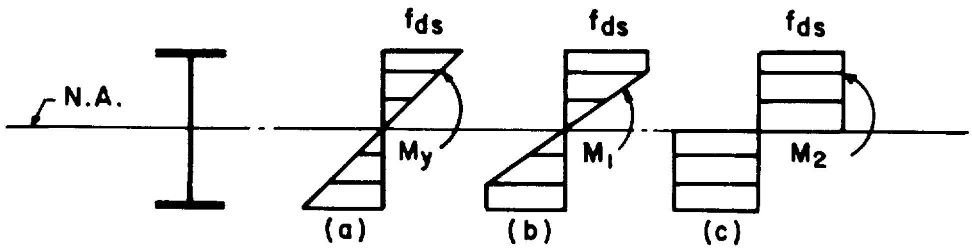


Figure 5-6 Theoretical stress distribution for pure bending at various stages of dynamic loading

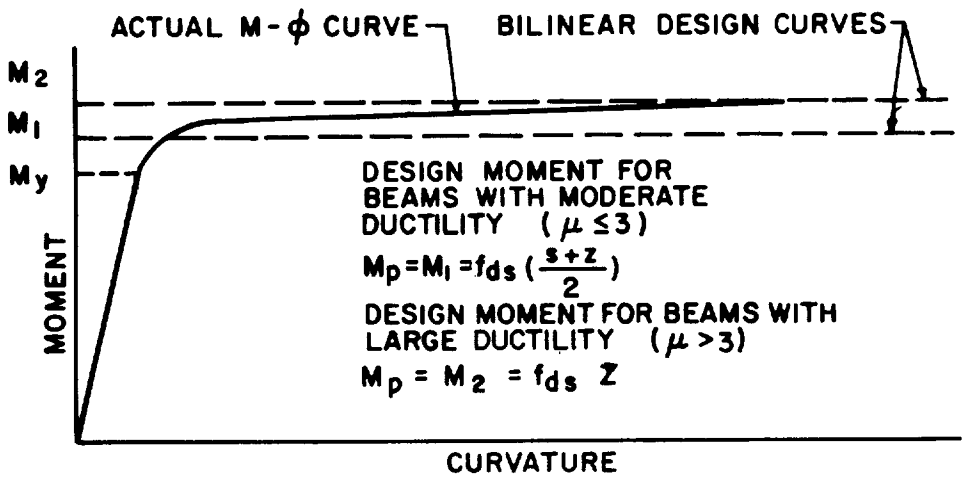


Figure 5-7 Moment-curvature diagram for simple-supported, dynamically loaded, I-shaped beams

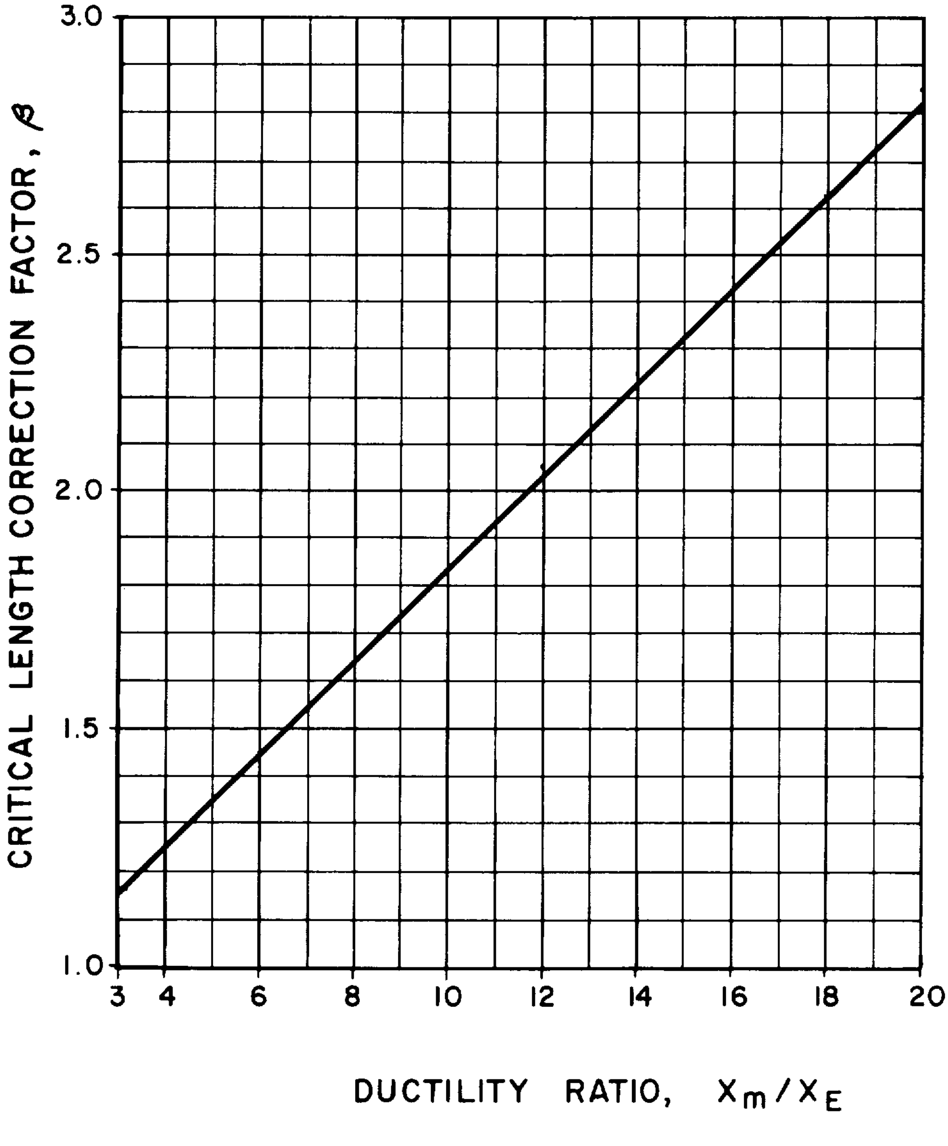


Figure 5-8 Values of β for use in equations 5-20 and 5-21

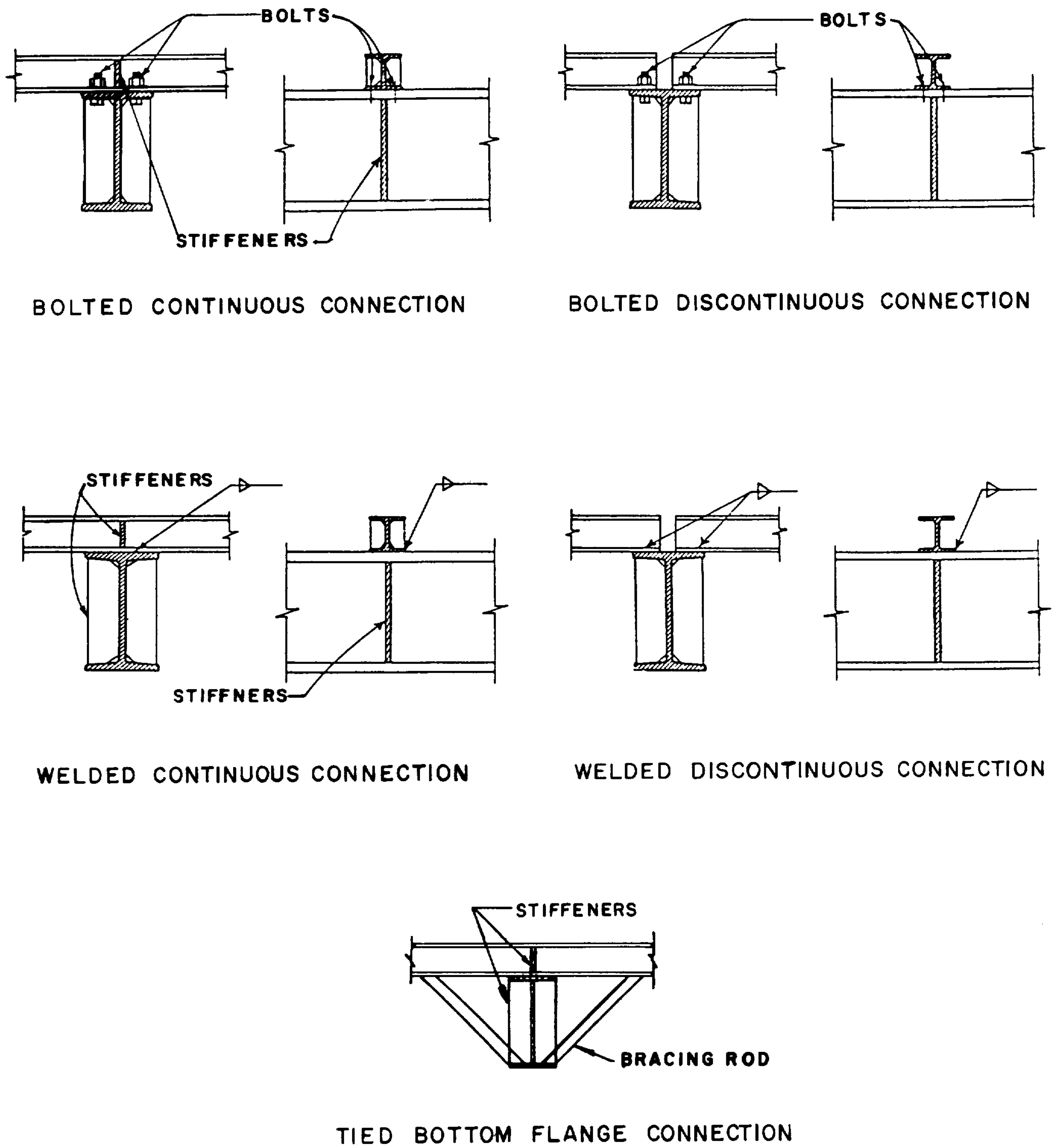


Figure 5-9 Typical lateral bracing details

DESIGN OF PLATES

5-27. General

The emphasis in this section is on the dynamic plastic design of plates. As in the case for simply supported and continuous beams, design data have been derived from the static provisions of the AISC Specification with necessary modifications and additions for blast design.

This section covers the dynamic flexural capacity of plates, as well as the necessary information for determining the equivalent bilinear resistance-deflection functions used in evaluating the flexural response of plates. Also presented is the supplementary consideration of adequate shear capacity at negative yield lines.

5-28. Dynamic Flexural Capacity

As is the case for standard I-shaped sections, the ultimate dynamic moment-resisting capacity of a steel plate is a function of the elastic and plastic modulii and the dynamic design stress. For plates or rectangular cross section beams, the plastic section modulus is 1.5 times the elastic section modulus.

A representative moment-curvature relationship for a simply-supported steel plate is shown in Figure 5-10. The behavior is elastic until a curvature corresponding to the yield moment M_y is reached. With further increase in load, the curvature increases at a greater rate as the fully plastic moment value, M_2, is approached. Following the attainment of M_2, the curvature increases while the moment remains constant.

For plates and rectangular cross section beams, M_2 is 50 percent greater than M_y, and the nature of the transition from yield to the fully plastic condition depends upon the plate geometry and end conditions. It is recommended that a capacity midway between M_y and M_2 be used to define the plastic design moment, M_p (M_1 in Figure 5-11), for plates and rectangular cross section beams. Therefore, for plates with any ductility ratio, Equation 5-7 applies.

5-29. Resistance and Stiffness Functions

Procedures for determining stiffness and resistance factors for one- and two-way plate elements are outlined in Chapter 3. These factors are based upon elastic deflection theory and the yield-line method, and are appropriate for defining the stiffness and ultimate load-carrying capacity of ductile structural steel plates. In applying these factors to steel plates, the modulus of elasticity should be taken equal to 29,600,000 psi. For two-way isotropic steel plates, the ultimate unit positive and negative moments are equal in all directions; i.e.

$$M_{vn} = M_{vp} = M_{hn} = M_{hp} = M_p$$

where M_p is defined by Equation 5-7 and the remaining values are ultimate unit moment capacities as defined in Section 3-9.3 of Chapter 3. Since the stiffness factors were derived for plates with equal stiffness properties in each direction, they are not applicable to the case of orthotropic steel plates, such as stiffened plates, which have different stiffness properties in each direction.

5-30. Design for Flexure

The procedure for the flexural design of a steel plate is essentially the same as the design of a beam. As for beams, it is suggested that preliminary dynamic load factors listed in Table 5-4 be used for plate structures. With the stiffness and resistance factors from Section 5-29 and taking into account the influence of shear on the available plate moment capacity as defined in Section 5-31, the dynamic response and rebound for a given blast loading may be determined from the response charts in Chapter 3. It should be noted that for $\mu > 10$, the dynamic design stress, incorporating the dynamic ultimate stress, f_{du}, may be used (see Equation 5-2).

5-31. Design for Shear

In the design of rectangular plates, the effect of simultaneous high moment and high shear at negative yield lines upon the plastic strength of the plate may be significant. In such cases, the following interaction formula describes the effect of the support shear, V, upon the available moment capacity, M:

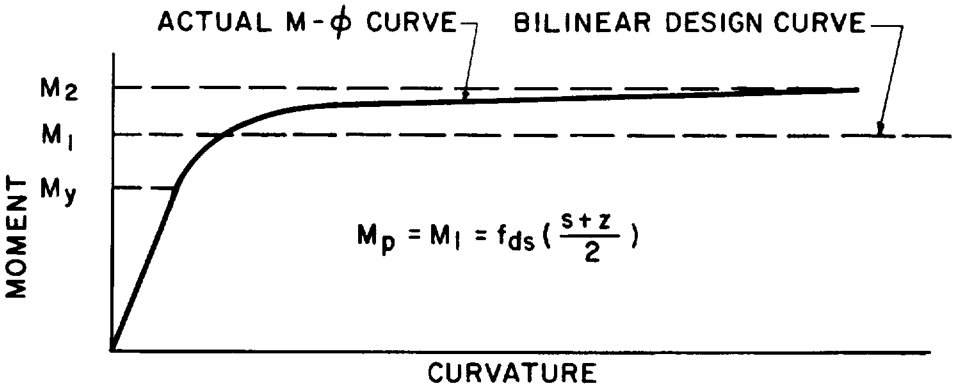
$$M/M_p = 1 - (V/V_p)^4 \qquad \text{5-23}$$

where M_p is the fully plastic moment capacity in the absence of shear calculated from Equation 5-7, and V_p is the ultimate shear capacity in the absence of bending determined from Equation 5-16 where the web area, A_w, is taken equal to the total cross sectional area at the support.

For two-way elements, values for the ultimate support shears which are applicable to steel plates are presented in Table 3-10 of Chapter 3.

It should be noted that, due to the inter-relationship between the support shear, V, the unit ultimate flexural resistance, r_u, of the two-way element, and the fully plastic moment resistance, M_p, the determination of the resistance of steel plates considering Equation 5-23, is not a simple calculation. Fortunately, the number of instances when negative yield lines with support shears are encountered for steel plates will be limited. Moreover, in most applications, the V/V_p ratio is such that the available moment capacity is at least equal to the plastic design moment for plates (Equation 5-7).

It is recommended that for a V/V_p ratio on negative yield lines less than 0.67, the plastic design moment for plates, as determined from Equation 5-7, should be used in design. However, if V/V_p is greater than 0.67, the influence of shear on the available moment capacity must be accounted for by means of Equation 5-23.



NOTE: SEE FIGURE 5-7 FOR M_y, M_1 AND M_2

Figure 5-10 Moment-curvature diagram for dynamically loaded plates and rectangular cross-section beams

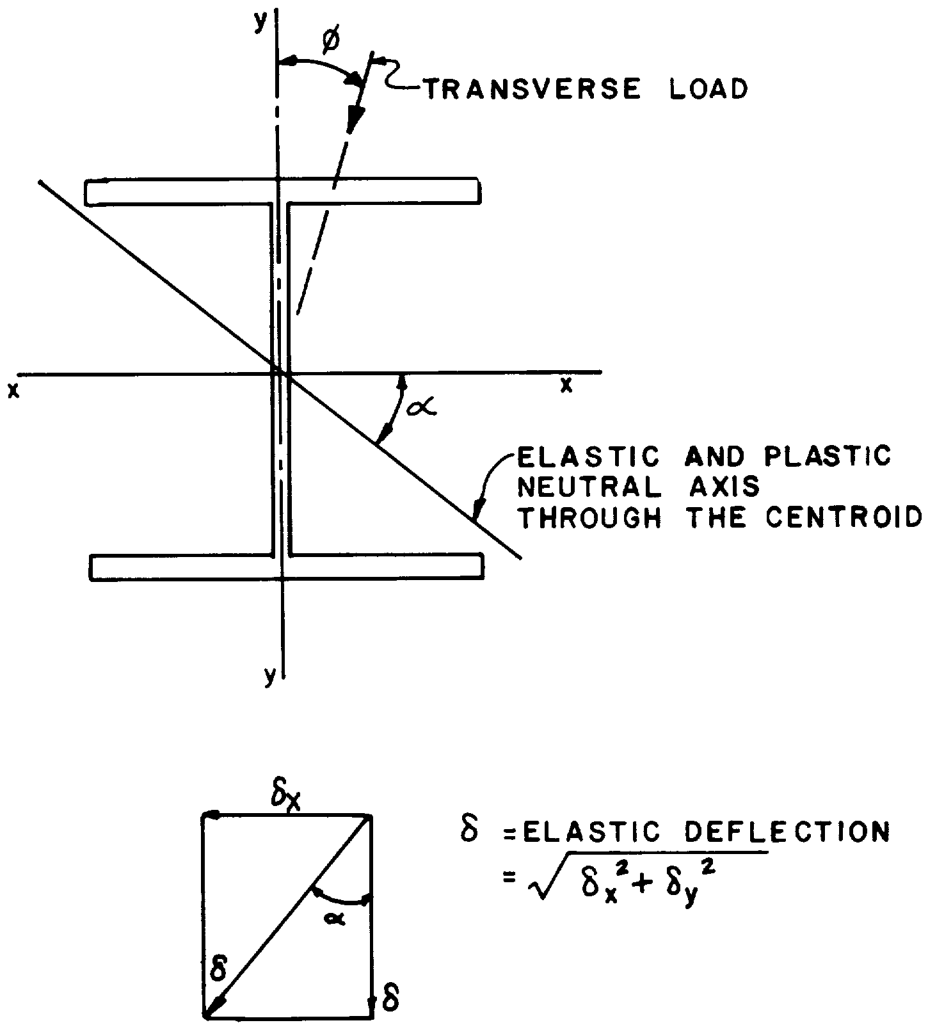


Figure 5-11 Biaxial bending of a doubly-symmetric section

Table 5-4 **Preliminary Dynamic Load Factors for Plates**

Deflection Magnitude	Deformation*		DLF
	θ max	μ max	
Small	2	5	1.0
Moderate	4	10	0.8
Large	12	20	0.6

* Whichever governs

SPECIAL CONSIDERATIONS, BEAMS

5-32. Unsymmetrical Bending

5-32.1. General

In blast design, the number of situations where unsymmetrical bending occurs is limited and where encountered, it can be treated without serious economic penalty. Due to the fact that blast overpressure loads act normal to the surfaces of a structure, the use of doubly-symmetric cross sections for purlins and girts (e.g., hot-rolled S- and W-sections or cold-formed channels used back-to-back) is generally recommended. In such cases, the deformation criteria for flexural members in Section 5-22 apply.

Unsymmetrical bending occurs when flexural members are subjected to transverse loads acting in a plane other than a principal plane. With this type of loading, the following are applicable:

(1) The member's neutral axis is not perpendicular to the plane of loading.

(2) Stresses cannot in general be calculated by means of the simple bending formula (M_c/I).

(3) The bending deflection does not coincide with the plane of loading but is perpendicular to the inclined neutral axis.

(4) If the plane of loads does not pass through the shear center of the cross section, bending is also accompanied by twisting.

Doubly-symmetric S, W, and box sections acting as individual beam elements and subjected to biaxial bending, i.e., unsymmetrical bending without torsion, can be treated using the procedures outlined in the following sections.

5-32.2. Elastic and Plastic Section Modulus

The inclination of the elastic and plastic neutral axis through the centroid of the section can be calculated directly from the following relationship (see Figure 5-11):

$$\tan \alpha = (I_x / I_y) \tan \phi \qquad \text{5-24}$$

where α = angle between the horizontal principal plane and the neutral axis

ϕ = angle between the plane of the load and the vertical principal plane

and x and y refer to the horizontal and vertical principal axes of the cross-section.

The equivalent elastic section modulus may be evaluated from the following equation:

$$S = (S_x S_y)/(S_y \cos \phi + S_x \sin \phi) \qquad \text{5-25}$$

where S_x = elastic section modulus about the x-axis

S_y = elastic section modulus about the y-axis

The plastic section modulus can be calculated using Equation 5-6. With these values of the elastic and plastic section modulii, the design plastic moment capacity can be determined from Equation 5-7.

5-32.3. Equivalent Elastic Stiffness

In order to define the stiffness and bilinear resistance function, it is necessary to determine the elastic deflection of the beam. This deflection may be calculated by resolving the load into components acting in the principal planes of the cross section. The elastic deflection, δ, is calculated as the resultant of the deflections determined by simple bending calculations in each direction (see Figure 5-11). The equivalent elastic deflection on the bilinear resistance function X_E, may then be determined by assuming that the elastic stiffness is valid up to the development of the design plastic moment capacity, M_p.

5-32.4. Lateral Bracing and Recommended Design Criteria

The bracing requirements of Section 5-26 may not be totally adequate to permit a biaxially loaded section to deflect into the inelastic range without premature failure. However, for lack of data, the provisions of Section 5-26 on lateral bracing may be used if the total member end rotation corresponding to the total deflection due to the inclined load is limited to 2 degrees. In addition, the actual details of support conditions and/or bracing provided to such members by the other primary and secondary members of the frame must be carefully checked to ensure that the proper conditions exist to permit deflections in the inelastic range.

5-32.5. Torsion and Unsymmetrical Bending

The inelastic behavior of sections subjected to unsymmetrical bending, with twisting, is not totally known at present. Consequently, the use of sections with the resultant load not passing through the shear center is not recommended in plastic design of blast-resistant structures, unless torsional constraints are provided for the elements. In actual installations, however, the torsional constraint offered to a purlin or girt by the flexural rigidity of the floor, roof, or wall panels to which it is attached may force the secondary member to deflect in the plane of loading with little or no torsional effects. Under such conditions or when some other means of bracing is provided to prevent torsional rotation in both the loading and rebound phases of the response, such unsymmetrically loaded members may be capable of performing well in the plastic range. However, because of the limited data presently available, there is insufficient basis for providing practical design guidelines in this area. Hence, if a case involving unsymmetrical bending with torsion cannot be prevented in design, the maximum ductility ratio should be limited to 1.0.

Furthermore, special precautions may have to be taken to restrict the torsional-flexural distortions that can develop under unsymmetrical loading, thereby reducing the flexural capacity of the member.

5-33. Steel Joists and Joist Girders (Open-web Steel Joists)

Open-web joists are commonly used as load-carrying members for the direct support of roof and floor deck in buildings. The design of joists for conventional loads is covered by the "Standard Specifications, Load Tables and Weight Tables for Steel Joists and Joist Girders", adopted by the Steel Joist Institute. For blast design, all the provisions of this Specification are in force, except as modified herein.

These joists are manufactured using either hot-rolled or cold-formed steel. H-Series joists are composed of 50-ksi steel in the chord members and either 36-ksi or 50-ksi steel for the web sections. LH Series, DLH Series and joist girders are composed of joist chords or web sections with a yield strength of at least 36 ksi, but not greater than 50 ksi.

Standard load tables are available for simply supported, uniformly loaded joists supporting a deck and so constructed that the top chord is braced against lateral buckling. These tables indicate that the capacity of a particular joist may be governed by either flexural or shear (maximum end reaction) considerations. As discussed previously, it is preferable in blast applications to select a member whose capacity is controlled by flexure rather than shear, which may cause abrupt failure.

The tabulated loads include a check on the bottom chord as an axially loaded tensile member and the design of the top chord as a column or beam column. The width-thickness ratios of the unstiffened or stiffened elements of the cross section are also limited to values specified in the Standard Specifications.

The dynamic ultimate capacity of open-web joists may be taken equal to 1.7 a x c times the load given in the joist tables. This value represents the safety factor of 1.7 multiplied by a dynamic increase factor, c, and the average strength increase factor, a (see Section 5-12).

The adequacy of the section in rebound must be evaluated. Upon calculating the required resistance in rebound, r^{-}/r_u, using the rebound chart in Chapter 3 (Figure 3-268), the lower chord must be checked as a column or beam column. If the bottom chord of a standard joist is not adequate in rebound, the chord must be strengthened either by reducing the unbraced length or by increasing the chord area. The top chord must be checked as an axial tensile member, but in most circumstances, it will be adequate.

The bridging members required by the joist specification should be checked for both the initial and rebound phase of the response to verify that they satisfy the required spacing of compression flange bracing for lateral buckling.

The joist tables indicate that the design of some joists is governed by shear, that is, failure of the web bar members in tension or compression near the supports. In such cases, the ductility ratio for the joist should not exceed unity. In addition, the joist members near the support should be investigated for the worst combination of slenderness ratio and axial load under load reversal.

For hot-rolled members not limited by shear considerations, design ductility ratios up to the values specified in Section 5-35 can be used. The design ductility ratio of joists with light gauge chord members should be limited to 1.0.

The top and bottom chords should be symmetrical about a vertical axis. If double angles or bars are used as chord members, the components of each chord should be fastened together so as to act as a single member.

SPECIAL CONSIDERATIONS, COLD-FORMED STEEL PANELS

5-34. Blast Resistant Design of Cold-Formed Steel Panels

5-34.1. General

Recent studies on cold-formed panels have shown that the effective width relationships for cold-formed light gauge elements under dynamic loading do not differ significantly from the static relationships. Consequently, the recommendations presently contained in the AISI Specifications are used as the basis for establishing the special provisions needed for the design of cold-formed panels subjected to blast loads. Some of the formulas of the Specification have been extended to comply with ultimate load conditions and to permit limited performance in the inelastic range.

Two main modes of failure can be recognized, one governed by bending and the other by shear. In the case of continuous members, the interaction of the two influences plays a major role in determining the behavior and the ultimate capacity. Due to the relatively thin webs encountered in cold-formed members, special attention must also be paid to crippling problems. Basically, the design will be dictated by the capacity in flexure but subject to the constraints imposed by shear resistance and local stability.

5-34.2. Resistance in Flexure

The material properties of the steel used in the production of cold-formed steel panels conforms to ASTM A446. This standard covers three grades (a, b, and c) depending on the yield point. Most commonly, panels are made of steel complying with the requirements of grade a, with a minimum yield point of 33 ksi and an elongation of rupture of 20 percent for a 2-inch gauge length. However, it is generally known that the yield stress of the material used in the manufacture of cold-formed panels generally exceeds the specified minimum yield stress by a significant margin; therefore, it is recommended that a design minimum yield stress of 40 ksi (corresponding to an average strength increase factor of a = 1.21) be used unless the actual yield stress of the material is known. For grades b and c which exhibit higher minimum yield points, an average strength increase factor of 1.21 is also recommended.

In calculating the dynamic yield stress of cold-formed steel panels, it is recommended that a dynamic increase factor, c, of 1.1 be applied irrespective of actual strain rate and, consequently, the value of the dynamic design stress to be used is

$$f_{ds} = a \times c \times f_y = 1.21 \times 1.1 \times f_y = 1.33\ f_y \qquad 5\text{-}26$$

and hence, f_{ds} equals 44 ksi for the particular case of $f_y = 33$ ksi.

Ultimate design procedures, combined with the effective width concept, are used in evaluating the strength of cold-formed light gauge elements. Thus, a characteristic feature of cold-formed elements is the variation of their section properties with the intensity of the load. As the load increases beyond the level corresponding to the occurrence of local buckling, the effective area of the compression flange is reduced; as a result, the neutral axis moves toward the tension flange with the effective properties of the cross section such as A (area), I (moment of inertia) and S (section modulus), decreasing with load increase. The properties of the panels, as tabulated by the manufacturer, are related to different stress levels. The value of S referred to that of the effective section modulus at ultimate and the value of I related to a service stress level of 20 ksi. In the case of panels fabricated from hat sections and a flat sheet, two section modulii are tabulated, S^+ and S^-, referring to the effective section modulus for positive and negative moments, respectively. Consequently, the following ultimate moment capacities are obtained:

$$M_{up} = f_{ds}S^+ \qquad 5\text{-}27$$

$$M_{un} = f_{ds}S^- \qquad 5\text{-}28$$

where M_{up} = ultimate positive moment capacity for a 1-foot width of panel

M_{un} = ultimate negative moment capacity for a 1-foot width of panel

It should be noted that in cases where tabulated section properties are not available, the required properties may be calculated based upon the relationships in the AISI Design Specification.

As for any single-span flexural element, the panel may be subjected to different end conditions, either simply supported or fixed. The fixed-fixed condition is seldom found in practice since this situation is difficult to achieve in actual installations. The simply fixed condition is found because of symmetry in each span of a two-span continuous panel. For multi-span members (three or more), the response is governed by that of the first span which is generally characterized by a simply supported condition at one support and a partial moment restraint at the other. Three typical cases can, therefore, be considered:

(1) Simply supported at both ends (single span).

(2) Simply supported at one end and fixed at the other (two equal span continuous member).

(3) Simply supported at one end and partially fixed at the other (first span of an equally spaced multi-span element).

The resistance of the panel is a function of both the strength of the section and the maximum moment in the member.

The ability of the panel to sustain yielding of its cross section produces significant moment re-distribution in the continuous member which results in an increase of the resistance of the panel.

The behavior of cold-formed steel sections in flexure is nonlinear as shown in Figure 5-12. To simulate the bilinear approximation to the resistancedeflection curve, a factor of 0.9 is applied to the peak resistance. Therefore, for design purposes, the recommended resistance formula for a simply supported, single-span panel is given by:

$$r_u = 0.9 \times 8\ M_{up}/L^2 = 7.2\ M_{up}/L^2 \qquad 5\text{-}29$$

where r_u is the resistance per unit length of panel, and L is the clear or effective span length.

The recommended resistance formula for a simply-fixed, single-span panel or first span of an equally spaced continuous panel is given by:

$$r_u = 0.9 \times 4\ (M_{un} + 2M_{up})/L^2 = 3.6\ (M_{un} + 2M_{up})/L^2 \qquad 5\text{-}30$$

5-34.3. **Equivalent Elastic Deflection**

As previously mentioned, the behavior of cold-formed sections in flexure is nonlinear as shown in Figure 5-12. A bilinear approximation of the resistance-deflection curve is assumed for design. The equivalent elastic deflection X_E is obtained by using the following equation:

$$X_E = (\beta r_u L^4)/EI_{20} \qquad 5\text{-}31$$

where β is a constant which depends on the support conditions and whose values are as follows:

$\beta = 0.0130$ for a simply supported element

$\beta = 0.0062$ for simply fixed or continuous elements

I_{20} is defined as the effective moment of inertia of the section at a service stress of 20 ksi. The value of I_{20} is generally tabulated as a section property of the panel. The value of r_u is obtained from Equation 5-29 or 5-30.

5-34.4. **Design for Flexure**

When performing a one-degree-of-freedom analysis of the panel's behavior, the properties of the equivalent system can be evaluated by using a load-mass factor, $K_{LM} = 0.74$, which is an average value applicable to all support conditions. The natural period of vibration for the equivalent single-degree system is thus obtained by substituting into Equation 5-15:

$$T_N = 2\pi\ (0.74\ mL/K_E)^{1/2} \qquad 5\text{-}32$$

where $m =$ w/g is the unit mass of the panel

$K_E =$ $r_u L/X_E$ is the equivalent elastic stiffness of the system

5-34.5. **Recommended Ductility Ratios**

Figure 5-12 illustrates the nonlinear character of the resistance-deflection curve and the recommended bilinear approximation. The initial slope of the actual curve is fairly linear until it enters a range of marked nonlinearity and, finally, a point of instability. However, excessive deflections cause the decking to act as a membrane in tension (solid curve) and, consequently, a certain level of stability sets in. It should be noted that, in order to use the procedure outlined in this section, care must be taken to adequately connect the ends of the decking so that it can achieve the desired level of tension-membrane action. A discussion of connectors at end panels is presented in Section 5-48. When tension-membrane action is not present, increased deflection will result in a significant dropoff in resistance as illustrated by the dotted curve in Figure 5-12.

Two limits of deformation are assigned, depending on end-anchorage condition of the panel. For panels having nominal end anchorage, that is, where tension-membrane action is minimal, the maximum deflection of the panel is X_o, as illustrated in Figure 5-12, and is defined by:

$$X_o = 1.75\ X_E \qquad 5\text{-}33$$

For panels with sufficient end anchorage to permit tension-membrane action, the maximum deflection of the panel is X_m, as illustrated in Figure 5-12, and is defined by:

$$X_m = 6.0\ X_E \qquad 5\text{-}34$$

5-34.6. Recommended Support Rotations

In order to restrict the magnitude of rotation at the supports, limitations are placed on the maximum deflections X_o and X_m as follows:

For elements where tension-membrane action is not present:

$$X_o = L/92 \text{ or } \theta_{max} = 1.25 \text{ degrees} \qquad 5\text{-}35$$

For elements where tension-membrane action is present:

$$X_m = L/92 \text{ or } \theta_{max} = 4 \text{ degrees} \qquad 5\text{-}36$$

5-34.7. Rebound

Appropriate dynamic response charts for one-degree-of-freedom systems in the elastic or elasto-plastic range under various dynamic loads are given in Chapter 3. The problem of rebound should be considered in the design of decking due to the different section properties of the panel, depending on whether the hat section or the flat sheet is in compression. Figure 5-13 presents the maximum elastic resistance in rebound as a function of T/T_N. While the behavior of the panel in rebound does not often control, the designer should be aware of the problem; in any event, there is a need for providing connections capable of resisting uplift or pull-out forces due to load reversal in rebound.

5-34.8. Resistance in Shear

Webs with h/t in excess of 60 are in common use among cold-formed members and the fabrication process makes it impractical to use stiffeners. The design web stresses must, therefore, be limited to ensure adequate stability without the aid of stiffeners, thereby preventing premature local web failure and the accompanying loss of load-carrying capacity.

The possibility of web buckling due to bending stresses exists and the critical bending stress is given by Equation 5-37:

$$f_{cr} = 640{,}000/(h/t)^2 \le f_y \qquad 5\text{-}37$$

By equating f_{cr} to 32 ksi, which is a stress close to the yielding of the material, a value h/t = 141 is obtained. Since it is known that webs do not actually fail at these theoretical buckling stresses due to the development of post-buckling strength, it can be safely assumed that webs with h/t ≤ 150 will not be susceptible to flexural buckling. Moreover, since the AISI recommendations prescribe a limit of h/t = 150 for unstiffened webs, this type of web instability need not be considered in the design.

Panels are generally manufactured in geometrical proportions which preclude web-shear problems when used for recommended spans and minimum support-bearing lengths of 2 to 3 inches. In blast design, however, because of the greater intensity of the loading, the increase in required flexural resistance of the panels requires shorter spans.

In most cases, the shear capacity of a web is dictated by instability due to either

(1) Simple shear stresses
(2) Combined bending and shearing stresses

For the case of simple shear stresses, as encountered at end supports, it is important to distinguish three ranges of behavior depending on the magnitude of h/t. For large values of h/t, the maximum shear stress is dictated by elastic buckling in shear and for intermediate h/t values, the inelastic buckling of the web governs; whereas for very small values of h/t, local buckling will not occur and failure will be caused by yielding produced by shear stresses. This point is illustrated in Figure 5-14 for f_{ds} = 44 ksi. The provisions of the AISI Specification in this area are based on a safety factor ranging from 1.44 to 1.67 depending upon h/t. For blast-resistant design, the recommended design stresses for simple shear are based on an extension of the AISI provisions to comply with ultimate load conditions. The specific equations for use in design for f_{ds} = 44, 66 and 88 ksi are summarized in Tables 5-5a, 5-6a, and 5-7a, respectively.

At the interior supports of continuous panels, high bending moments combined with large shear forces are present, and webs must be checked for buckling due to these forces. The interaction formula presented in the AISI Specification is given in terms of the allowable stresses rather than critical stresses which produce buckling. In order to adapt this interaction formula to ultimate load conditions, the problem of inelastic buckling under combined stresses has been considered in the development of the recommended design data.

In order to minimize the amount and complexity of design calculations, the allowable dynamic design shear stresses at the interior support of a continuous member have been computed for different depth-thickness ratios for f_{ds} = 44, 66, and 88 ksi, and tabulated in Tables 5-5b, 5-6b, and 5-7b, respectively.

5-34.9. Web Crippling

In addition to shear problems, concentrated loads or reactions at panel supports, applied over relatively short lengths, can produce load intensities that can cripple unstiffened thin webs. The problem of web crippling is rather complicated for theoretical analysis because it involves the following:

(1) Nonuniform stress distribution under the applied load and the adjacent portions of the web

(2) Elastic and inelastic stability of the web element

(3) Local yielding in the intermediate region of load application

(4) The bending produced by the eccentric load (or reaction) when it is applied on the bearing flange at a distance beyond the curved transition of the web

The AISI recommendations have been developed by relating extensive experimental data to service loads with a safety factor of 2.2 which was established taking into account the scatter in the data. For blast design of cold-formed panels, it is recommended that the AISI values be multiplied by a factor of 1.50 in order to relate the crippling loads to ultimate conditions with sufficient provisions for scatter in test data.

For those sections that provide a high degree of restraint against rotation of their webs, the ultimate crippling loads are given as follows:

Allowable ultimate end support reaction

$$Q_u = 1.5\ f_{ds} t^2\ [4.44 + 0.558\ (N/t)^{1/2}] \tag{5-38}$$

Allowable ultimate interior support reaction

$$Q_u = 1.5\ f_{ds} t^2\ [6.66 + 1.446\ (N/t)^{1/2}] \tag{5-39}$$

where Q_u = ultimate support reaction

f_{ds} = dynamic design stress

N = bearing length (in.)

t = web thickness (in)

The charts in figures 5-15 and 5-16 present the variation of Q_u as a function of the web thickness for bearing lengths from 1 to 5 inches for $f_d s$ = 44 ksi for end and interior supports, respectively. It should be noted that the values reported in the charts relate to one web only, the total ultimate reaction being obtained by multiplying Q_u by the number of webs in the panel.

For design, the maximum shear forces and dynamic reactions are computed as a function of the maximum resistance in flexure. The ultimate load-carrying capacity of the webs of the panel must then be compared with these forces. As a general comment, the shear capacity is controlled for simply supported elements and by the allowable design shear stresses at the interior supports for continuous panels.

In addition, it can be shown that the resistance in shear governs only in cases of relatively very short spans. If a design is controlled by shear resistance, it is recommended that another panel be selected since a flexural failure mode is generally preferred.

5-35 Summary of Deformation Criteria for Structural Elements

Deformation criteria are summarized in Table 5-8 for frames, beams and other structural elements including cold-formed steel panels, open-web joists and plates.

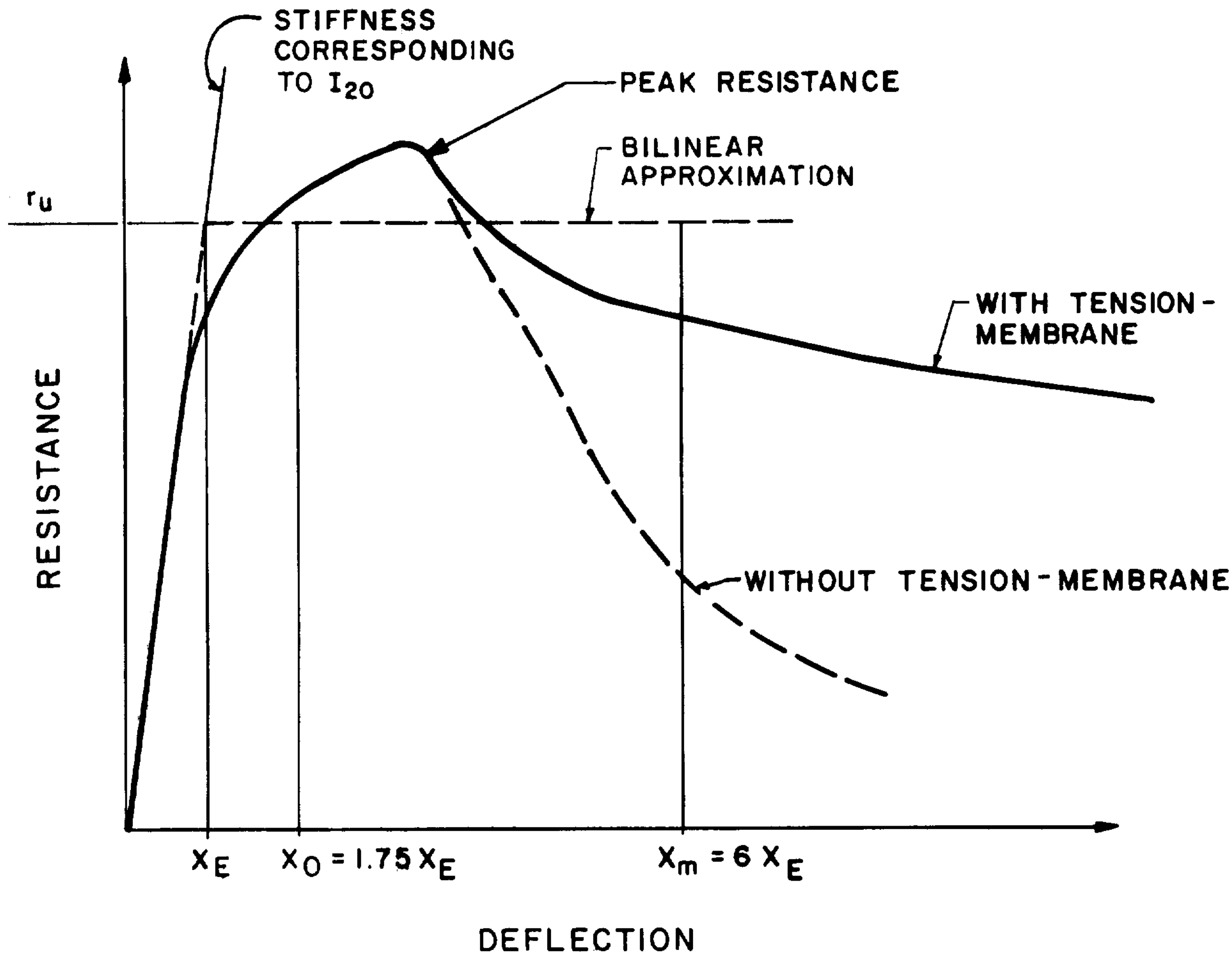


Figure 5-12 Resistance-deflection curve for a typical cold-formed section

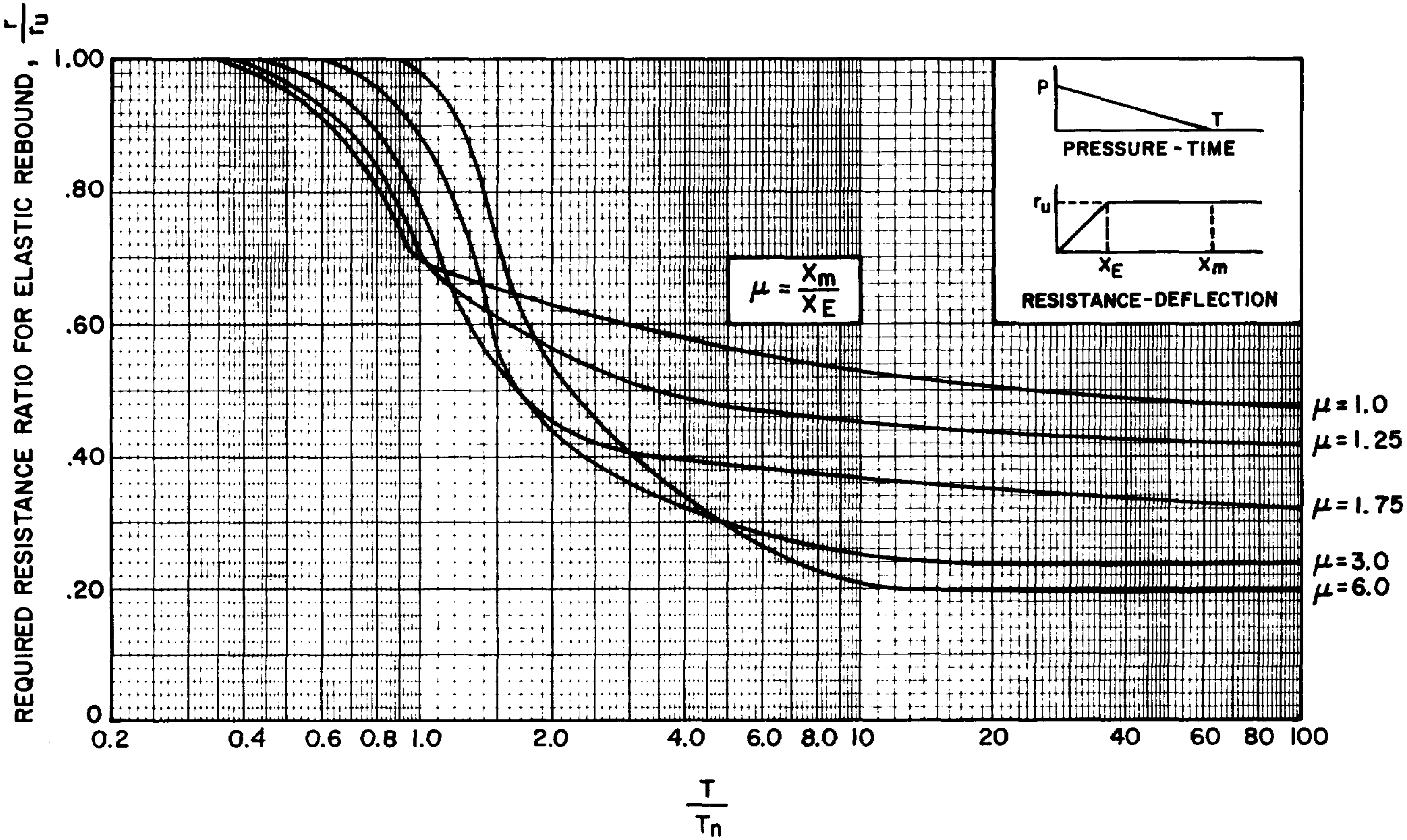


Figure 5-13 Elastic rebound of single-degree-of-freedom system

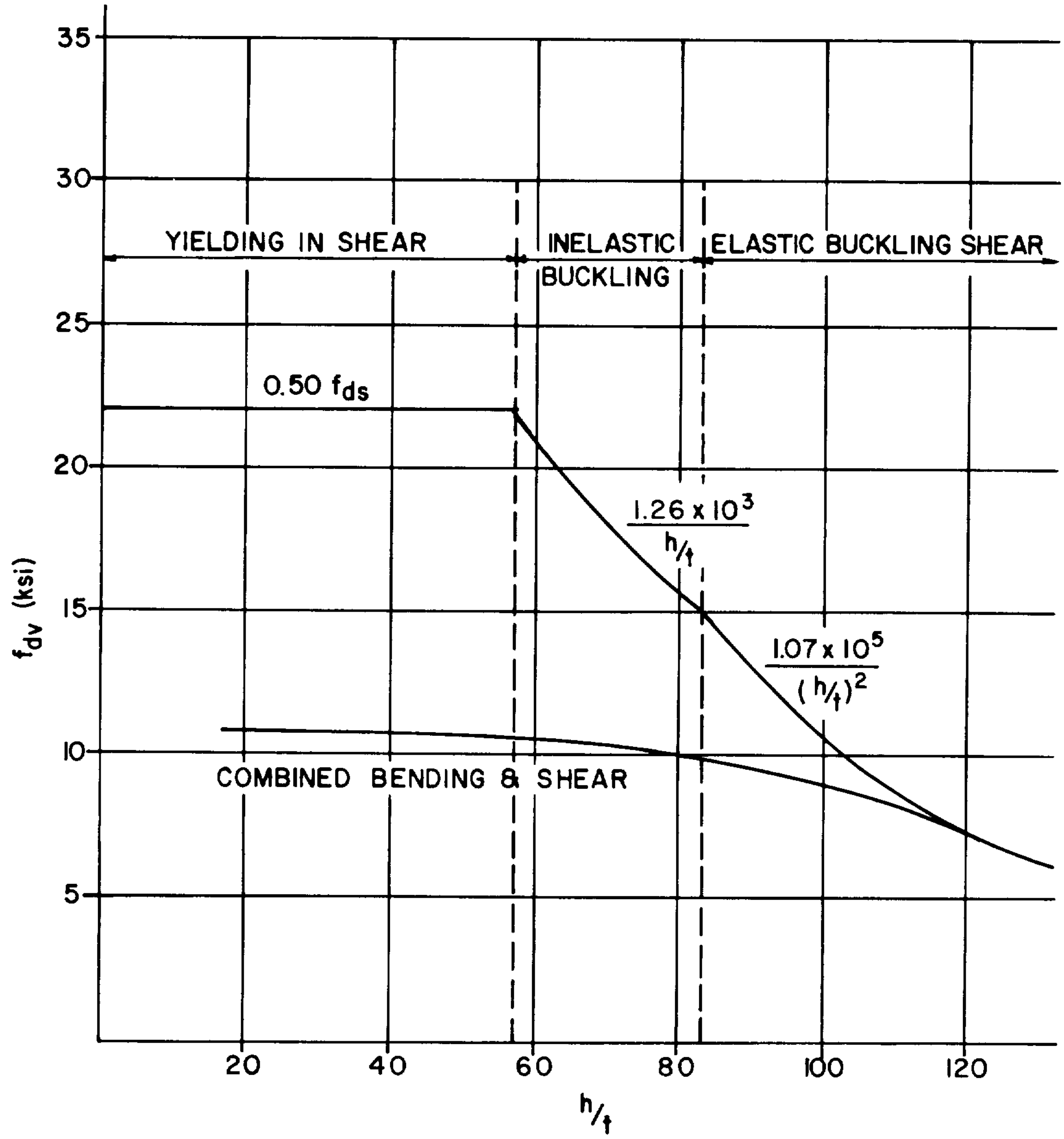


Figure 5-14 Allowable dynamic (design) shear stresses for webs of cold-formed members (f_{ds} = 44 ksi)

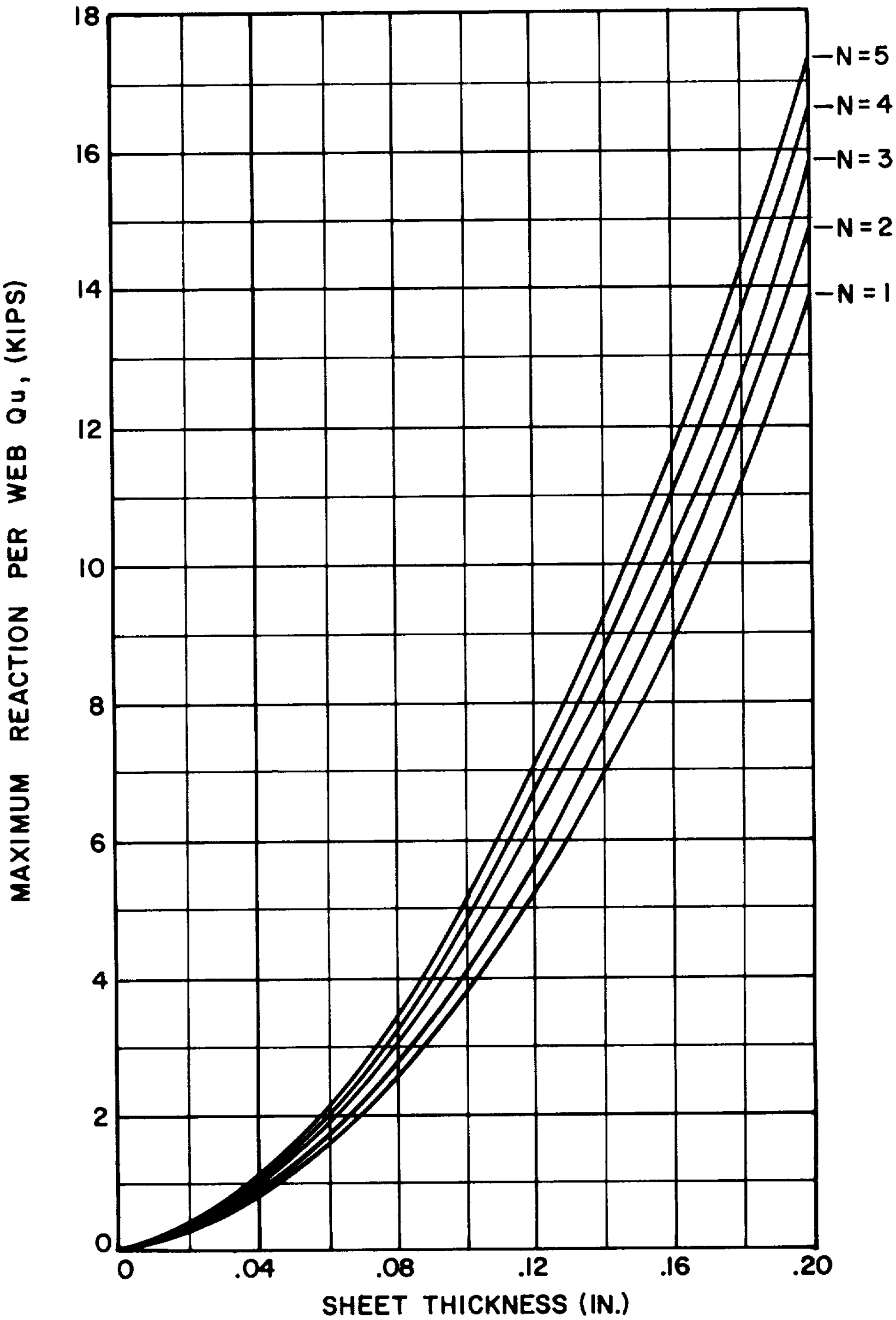


Figure 5-15 Maximum end support reaction for cold-formed steel sections (f_{ds} = 44 ksi)

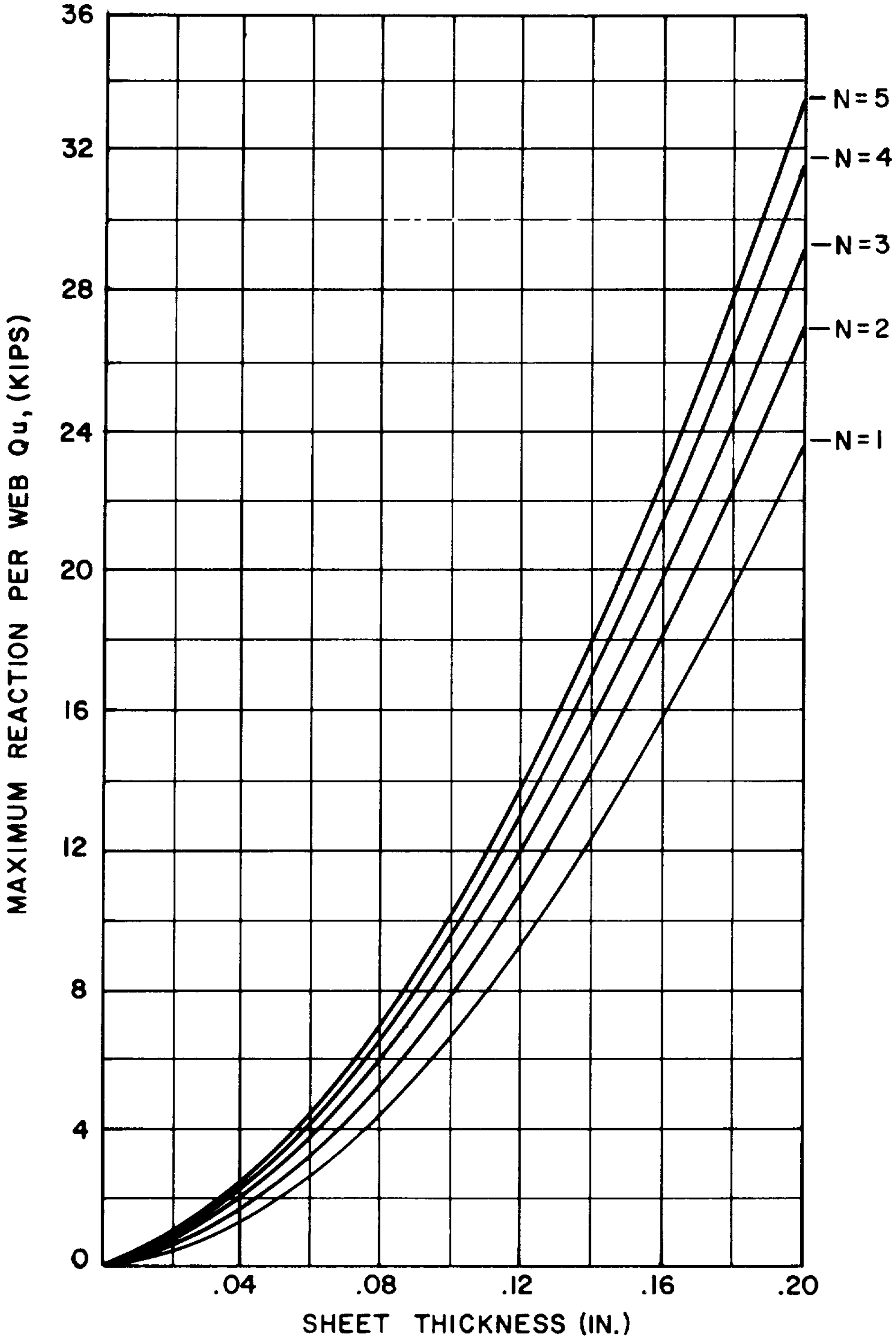


Figure 5-16 Maximum interior support reaction for cold-formed steel sections (f_{ds} = 44 ksi)

Table 5-5 Dynamic Design Shear Stress for Webs of Cold-formed Members (f_{ds} = 44 ksi)

(a) Simple Shear

$(h/t) \leq 57$	$f_{dv} = 0.50\ f_{ds} \leq 22.0$ ksi
$57 < (h/t) \leq 83$	$f_{dv} = 1.26 \times 10^3 / (h/t)$
$83 < (h/t) \leq 150$	$f_{dv} = 1.07 \times 10^5 / (h/t)$

(b) Combined Bending and Shear

(h/t)	f_{dv} (ksi)
20	10.94
30	10.84
40	10.72
50	10.57
60	10.42
70	10.22
80	9.94
90	9.62
100	9.00
110	8.25
120	7.43

Table 5-6 Dynamic Design Shear Stress for Webs of Cold-formed Members (f_{ds} = 66 ksi)

(a) Simple Shear

$(h/t) \leq 47$	$f_{dv} = 0.50\ f_{ds} \leq 33$ ksi
$47 < (h/t) \leq 67$	$f_{dv} = 1.54 \times 10^3/(h/t)$
$67 < (h/t) \leq 150$	$f_{dv} = 1.07 \times 10^5/(h/t)$

(b) Combined Bending and Shear

(h/t)	f_{dv} (ksi)
20	16.41
30	16.23
40	16.02
50	15.75
60	15.00
70	14.20
80	13.00
90	11.75
100	10.40
110	8.75
120	7.43

Table 5-7 **Dynamic Design Shear Stress for Webs of Cold-formed Members (f_{ds} = 88 ksi)**

(a) Simple Shear

$(h/t) \leq 41$	$f_{dv} = 0.50\ f_{ds} \leq 44$ ksi
$41 < (h/t) \leq 58$	$f_{dv} = 1.78 \times 10^3/(h/t)$
$58 < (h/t) \leq 150$	$f_{dv} = 1.07 \times 10^5/(h/t)$

(b) Combined Bending and Shear

(h/t)	f_{dv}(ksi)
20	21.60
30	21.00
40	20.00
50	18.80
60	17.50
70	16.00
80	14.30
90	12.50
100	10.75
110	8.84
120	7.43

Table 5-8 Summary of Deformation Criteria

Element	Highest level of Protection (Category No.) *	Additional Specifications	Deformation Type **	Maximum Deformation
Beams, purlins, spandrels or girts	1		θ μ	2° 10
	2		θ μ	12° 20
Frame structures	1		δ θ †	H/25 2°
Cold-formed steel floor and wall panels	1	Without tension-membrane action	θ μ	1.25° 1.75
		With tension-membrane action	θ μ	4° 6
Open-web joists	1		θ μ	2° 4
		Joists controlled by maximum end reaction	θ μ	1° 1
Plates	1		θ μ	2° 10
	2		θ μ	12° 20

θ = maximum member end rotation (degrees) measured from the chord joining the member ends.

δ = relative sidesway deflection between stories.

H = story height.

μ = ductility ratio (X_m/X_E)

* as defined in Chapter 1.
** whichever governs.
† individual frame member.

SPECIAL CONSIDERATIONS, BLAST DOORS

5-36. Blast Door Design

5-36.1. General

This section outlines procedures for the design of steel blast doors. Analytical procedures for the design of the individual elements of the blast door plate have been presented in earlier sections of this chapter. In addition to the door plate, door frames and anchorage, reversal bolts, gaskets and door operators are discussed. Blast doors are categorized by their functional requirements and method of opening.

5-36.2. Functions and Methods of Opening

5-36.2.1. Functional Requirements

Blast doors may be designed to contain an accidental explosion from within a structure so as to prevent pressure and fireball leakage and fragment propagation. Blast doors may also be designed to protect personnel and/or equipment from the effects of external blast loads. In this case, a limited amount of blast pressures may be permitted to leak into the protected area. In most cases, blast doors may be designed to protect the contents of a structure, thereby negating propagation when explosives are contained within the shelter.

5-36.2.2. Method of Opening

Blast doors may be grouped based on their method of opening, such as: (a) single leaf, (b) double leaf, (c) vertical lift, and (d) horizontal sliding.

5-36.3. Design Considerations

5-36.3.1. General

The design of a blast door is intrinsically related to its function during and/or after an explosion. Design considerations include whether or not the door should sustain permanent deflections, whether rebound mechanisms or fragment protection is required, and whether pressure leakage be tolerated. Finally, the design pressure range may dictate the type of door construction that is to be used, including solid steel plate or built-up doors.

5-36.3.2. Deflections

As stated in Section 5-16.7, plates can sustain a support rotation of 12 degrees without failing. This is applicable to blast doors providing that the resulting plate deflection does not collapse the door by pushing it through the opening. However, deflections may have to be limited if the mechanism used to open the door after an explosion is required to function. In addition, if a blast door is designed with a gasket so as to fully or nearly contain the pressure and fireball effects of an explosion, then deflections should be limited in order to ensure satisfactory performance of the gasket.

5-36.3.3. Rebound Mechanisms

Steel doors will be subjected to relatively large stress reversals caused by rebound. Blast doors may have to transfer these reversal loads by means of retracting pins or "reversal bolts." These heads can be mounted on any edge (sides, top, or bottom) of a doorplate. Reversal bolts can be designed as an integral part of the panic hardware assembly or, if tapered, they can be utilized in compressing the gasket around a periphery. The magnitude of the rebound force acting on the blast doors is discussed later.

5-36.3.4. Fragment Protection

A plate-type blast door, or the plate(s) of a built-up blast door may be sized to prevent fragment penetration. However, when the blast door is subjected to large blast loads and fragments, a supplementary fragment shield may be necessary since the combined effects of the fragments and pressures may cause premature door failure due to the notching effects produced by the fragments. Procedures for predicting the characteristics of primary fragments such as impact, velocity, and size of fragment are presented in Chapter 2. Methods for determining the depth of penetration of fragments into steel are given in Section 5-49.

5-36.3.5. Leakage Protection (gaskets)

Blast doors may be designed to partly or fully contain the pressure and fireball effects of an explosion in which case gaskets may have to be utilized around the edge if a door or its opening. A sample of a gasket is illustrated in Figure 5-17. This gasket will have to be compressed by means of a hydraulic operator which is capable of overcoming a force of 125 pounds per linear inch of the gasket. This gasket is made of neoprene conforming to the material callouts in Note 2 of Figure 5-17.

5-36.3.6. Type of Construction

Blast doors are formed from either solid steel plate or built-up steel construction.

Solid steel plate doors are usually used for the high pressure ranges (50 psi or greater) and where the door span is relatively short. Depending on plate thicknesses, these doors may be used when fragment impact is critical. These plates can range in thickness of 1-inch or greater. For thick plates, connections using high strength bolts or socket head cap screws are recommended in lieu of welding. However, the use of bolts or screws must preclude the passage of leakage pressures into or out of the structure depending on its use.

Built-up doors are used usually for the low pressure range and where long spans are encountered. A typical built-up blast door may consist of a peripheral frame made from steel channels with horizontal channels serving as intermediate supports for the interior and/or exterior steel cover plates. The pressure loads must be transferred to the channels via the plate. Concrete or some other material may be placed between the plates in order to add mass to the door or increase its fragment resistance capabilities.

5-36.4. Examples of Blast Door Designs

5-36.4.1. General

In order to illustrate the relationship between the function of a blast door and its design considerations, four examples are presented in the following section. Table 5-9 lists the design requirements of each of the above door examples.

5-36.4.2. Door Type A (Figure 5-18)

This blast door is designed to protect personnel and equipment from external blast pressures resulting from an accidental explosion. The door opening measures 8-feet high by 8-feet wide. It is a built-up double-leaf door consisting primarily of an exterior plate and a thinner interior plate both welded to a grid formed by steel tubes. Support rotations of each element (plate, channel, tube) have been limited to 2 degrees in order to assure successful operation of the panic hardware at the door interior. The direct blast load is transferred from the exterior plate to tubular members which form the door grid. The grid then transfers the loads to the door frame at the center of the opening through a set of pins attached to the top and bottom of the center mullions of the grid. At the exterior, the loads are transferred to the frame through the hinges which are attached to the exterior mullions and the frame. The reversal loads are also transferred by the pins and by the built-up door hinges. The center pins are also operated by the panic hardware assembly.

5-36.4.3. Door Type B (Figure 5-19)

This blast door is designed to prevent propagation from an accidental explosion into an explosives storage area. It is a built-up, sliding door protecting an opening 11-feet high by 16-feet wide, consisting of an exterior plate and a thinner interior plate. These plates are welded to vertical S-shapes which are spaced at 15-inch intervals. This door is designed to act as a one-way member, spanning vertically. Since flange buckling of the S-shapes is prohibited in the presence of the outer and inner plates acting as braces, the composite beam-plate arrangement is designed for a support rotation of 12 degrees. The yield capacity of the webs of the S-shapes in shear (Equation 5-16), as well as web crippling (Section 5-25), had to be considered in the design. This door has not been designed to resist reversal or rebound forces.

5-36.4.4. Door Type C (Figure 5-20)

This single-leaf blast door is designed as part of a containment cell which is used in the repeated testing of explosives. The door opening measures 4-feet 6-inches wide by 7-feet 6-inches high. It is the only door, in these samples, designed elastically since it is subjected to repeated blast loads. It consists primarily of a thick steel door plate protected from test fragments by a mild steel fragment shield. It is designed as a simply-supported (four sides) plate for direct internal loads and as a one-way element spanning the door width for rebound loads. It is equipped with a neoprene gasket around the periphery (Figure 5-17) as well as a series of six reversal bolts designed to transfer the rebound load into the door frame. The large thickness of the door plate warrants the use of high-strength, socket head cap screws in lieu of welding to connect the plate to the reversal bolt housing as well as to the fragment shield.

5-36.4.5. Door Type D (Figure 5-21)

This single-leaf blast door is designed as part of a containment structure which is used to protect nearby personnel and structures in the event of an accidental explosion. The door opening measures 4-feet wide by 7-feet high. It is designed as simply-supported on four sides for direct load and as a one-way element spanning the door width for rebound loads. It is equipped with a neoprene gasket around the periphery and a series of six reversal bolts which transfer the rebound load to the door frame. The reversal bolt housing and bearing blocks are welded to the door plate. Excessive deflections of the door plate under blast loading would hamper the sealing capacity of the gasket. Consequently, the door plate design rotation is limited to 2 degrees.

5-36.4.6. Other Types of Doors

Another type of blast door design is a steel arch or "bow" door. The tension arch door requires compression ties to develop the compression reactions from the arch. The compression arch door requires tension members to develop the tension reactions from the arch. These doors are illustrated in Figure 5-42.

5-36.5. Blast Door Rebound

Plate or element rebound can be determined for a single-degree-of-freedom system subjected to a triangular pulse (see Figure 5-13). However, when a system is subjected to a bilinear load, only a rigorous, step-by-step dynamic analysis can determine the percentage of elastic rebound. In lieu of a rigorous analysis, a method of determining the upper bound on the rebound force is presented here.

Three possible rebound scenarios are discussed. Figure 5-22 is helpful in describing each case.

(a) Case I - Gas load not present ($P_{gas} = 0$). In this case, the required rebound resistance is obtained from Figure 5-13.

(b) Case II - $t_m \le t_i$ In this case, the required rebound resistance is again obtained from Figure 5-10. This procedure, however, can overestimate the rebound load.

(c) Case III - $t_m > t_i$ Figure 5-22 illustrates the case whereby the time to reach the peak response, t_m, is greater than the point where the gas load begins to act (t_i). Assuming that the gas pressure can be considered constant over a period of time, it will act to lower the required rebound resistance since the resistance time curve will oscillate about the gas pressure time curve. In this case, the upper bound for the required rebound resistance is:

$$\bar{r} = r_u - P_{gas} \qquad 5\text{-}40$$

However, in all three cases, it is recommended that the required rebound resistance be at least equal to 50 percent of the peak positive door response.

5-36.6. Methods of Design

5-36.6.1. General

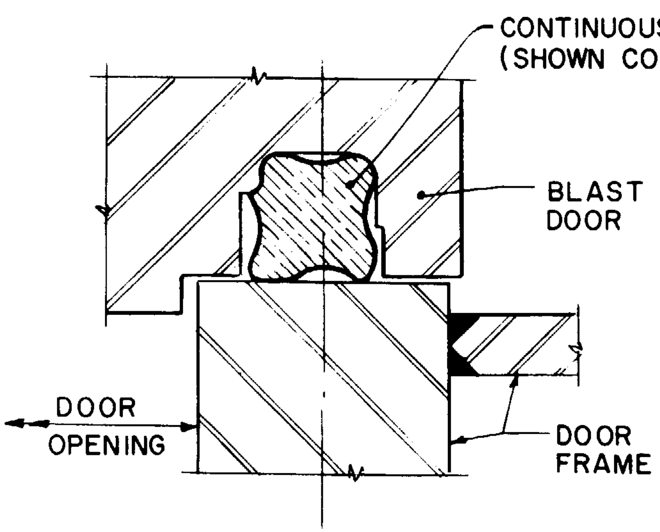
Techniques used for the design of two types of blast doors will be demonstrated. The first technique is used for the door illustrated in Figure 5-18 while the second is used for the door shown in Figure 5-21. Detailed procedures for the design of plate and beam elements, as well as the related design criteria, are presented in earlier section of this chapter and numerical examples are presented in Appendix A.

5-36.6.2. Built-up Door

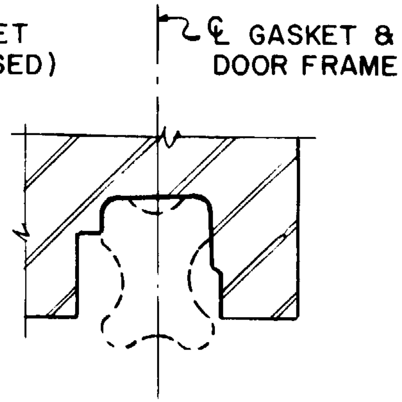
The built-up steel door shown in Figure 5-18 is constructed by welding the steel plates to the steel tubular grid (fillet welded to the exterior plate and plug-welded to the interior plate). The heavy exterior plate is designed as a continuous member supported by the tubes. The horizontal tubes, in turn, are designed as simply supported members, transferring load to the vertical tubes. The interior tubes are also designed as simply supported elements which transfer the direct and rebound loads to the pins while the side tubes transfer the direct load to the door frame proper and rebound loads to the hinges. The exterior tubes are also designed as simply supported elements with the supports located at the hinges.

5-36.6.3. Solid Steel Plate Door

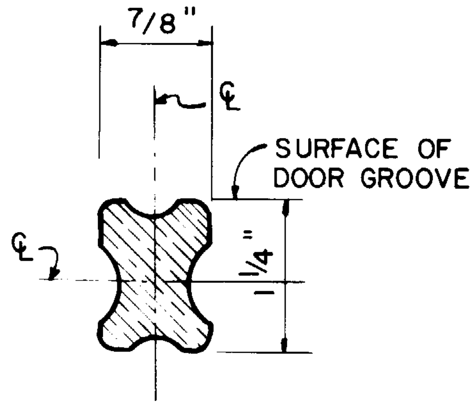
The steel plate of the blast door shown in Figure 5-21 is initially sized for blast pressures since no high speed fragments will be generated in the facility. The plate is sized for blast loading, considering the plate to be simply supported on four edges. The direct load is transferred to the four sides of the door frame. In rebound, the plate acts as a simple beam spanning the width of the door opening. The rebound force is transferred to the six reversal bolts and then into the door frame. The door frame, as illustrated in Figure 5-19, consists of two units; the first unit is imbedded into the concrete and the second unit is attached to the first one. This arrangement allows the first frame to be installed in the concrete wall prior to the fabrication of the door. After the door construction is completed, the subframe is attached to the embedded frame and, thus, the door installation is completed.



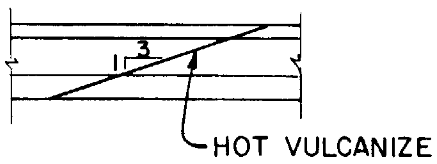
DETAIL
(TYPICAL AROUND PERIPHERY OF DOOR)



DOOR GROOVE



GASKET



GASKET SPLICE

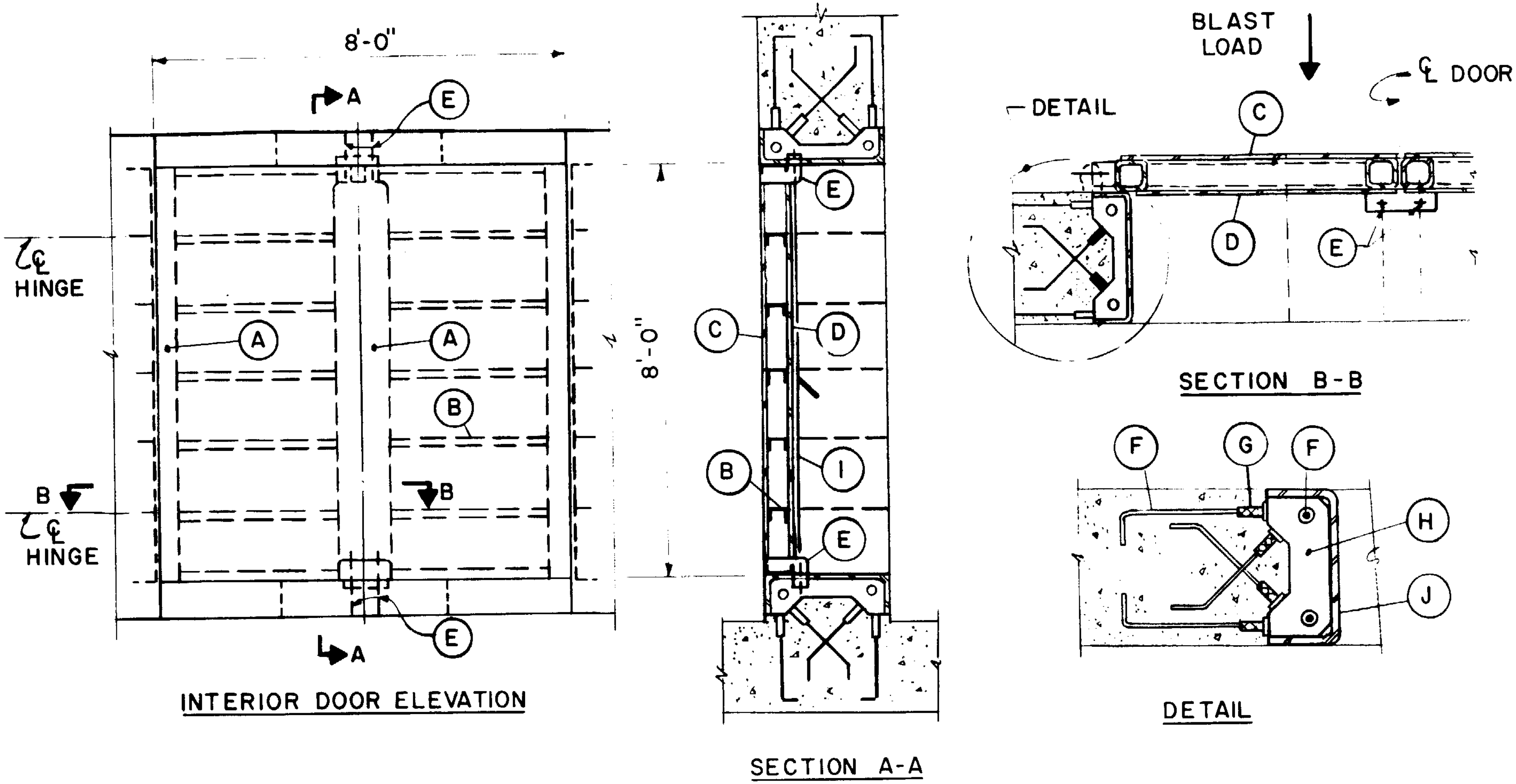
NOTES FOR GROOVE & GASKET

1. ALL SURFACES OF DOOR GROOVE SHALL BE MACHINED TO 125 MICROINCHES AND SHALL NOT BE PAINTED.

2. GASKETS FOR ALL DOORS SHALL MEET THE SPECIFICATION REQUIREMENTS AS PER ASTM D 2000-77a AND AS INDICATED IN THE FOLLOWING LINE CALL OUT; 2BC 520 A14 B14 C12 F17.

3. THE FORCE REQUIRED TO CLOSE DOOR IS APPROXIMATELY 125 POUNDS PER LINEAR INCH OF GASKET.

Figure 5-17 Gasket detail for blast door



LEGEND:

- (A) - STRUCTURAL TUBE
- (B) - CHANNEL
- (C) - EXTERIOR PLATE
- (D) - INTERIOR PLATE
- (E) - PINS FOR LOAD TRANSFER AND PANIC HARDWARE
- (F) - REINFORCING BAR
- (G) - MECHANICAL ANCHOR
- (H) - STIFFENER PLATE
- (J) - BENT DOOR FRAME PLATE
- (I) - PANIC HARDWARE

Figure 5-18 Built-up double-leaf blast door with frame built into concrete

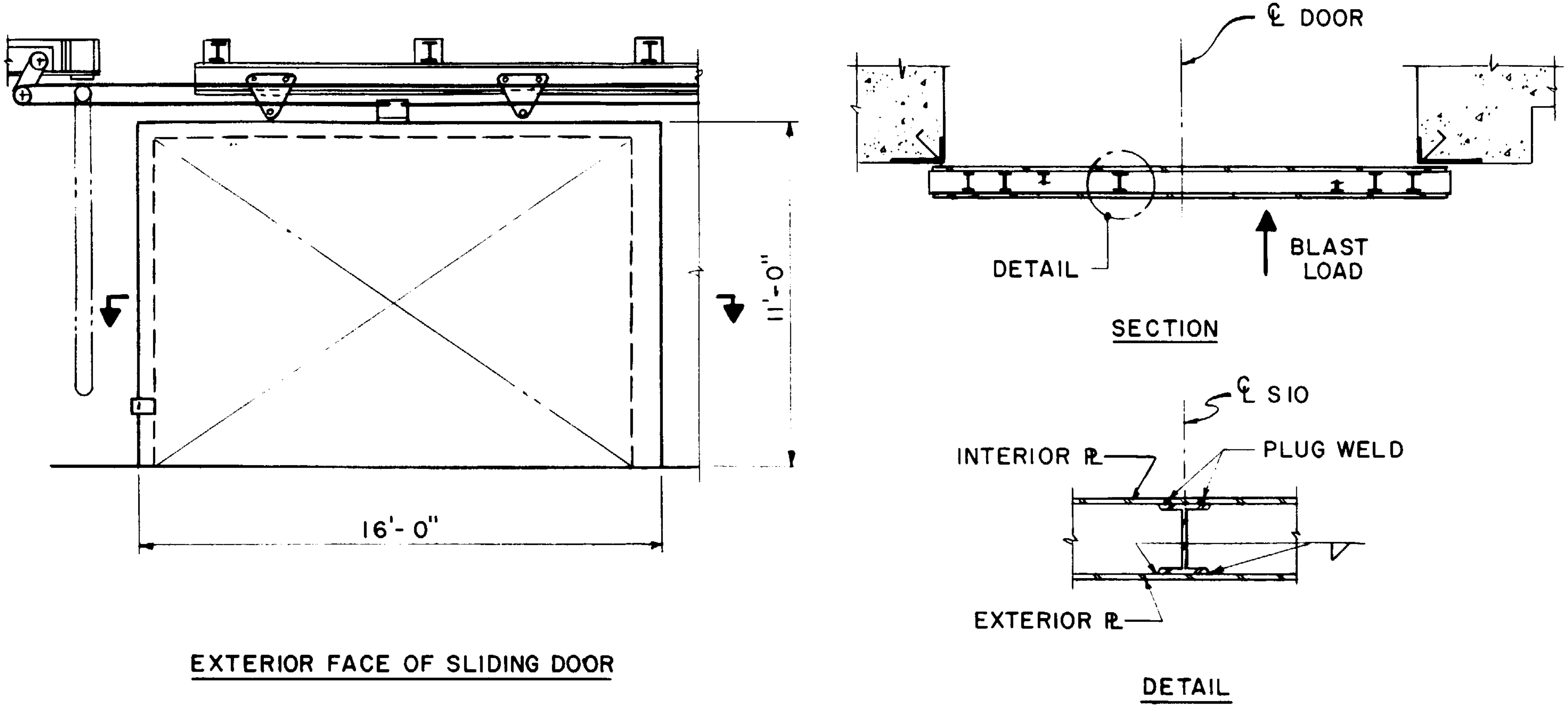


Figure 5-19 Horizontal sliding blast door

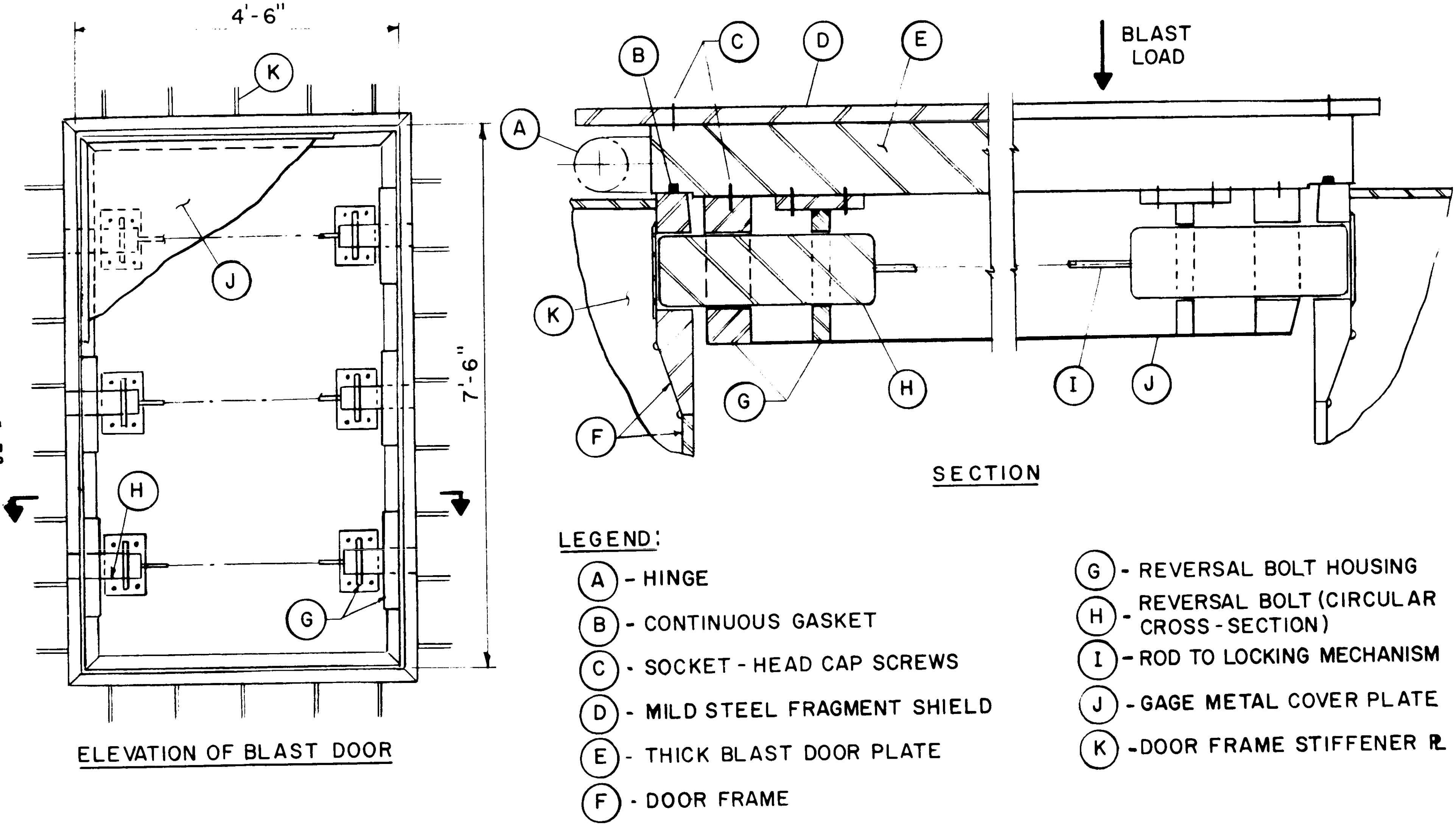
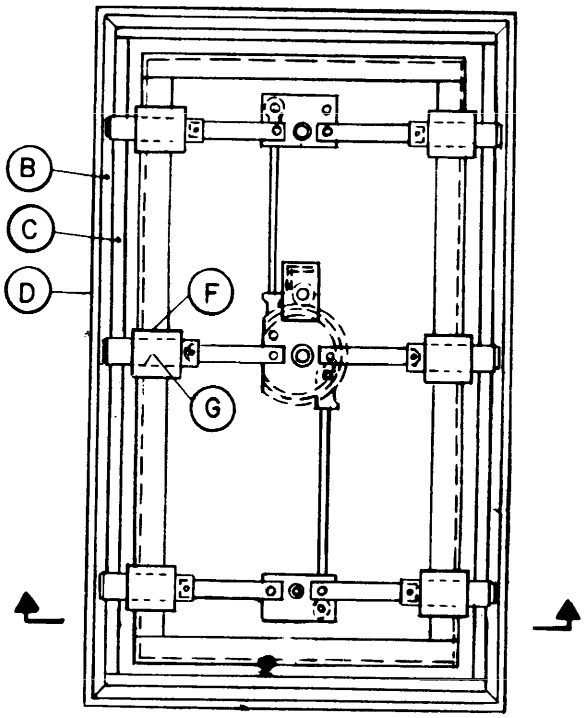
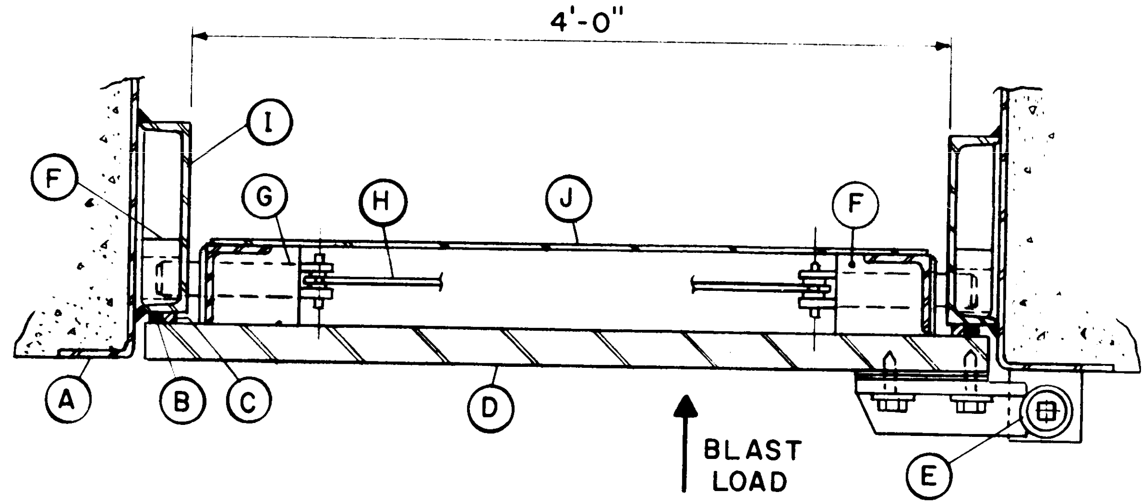


Figure 5-20 Single-leaf blast door with fragment shield (very high pressure)



ELEVATION OF BLAST DOOR

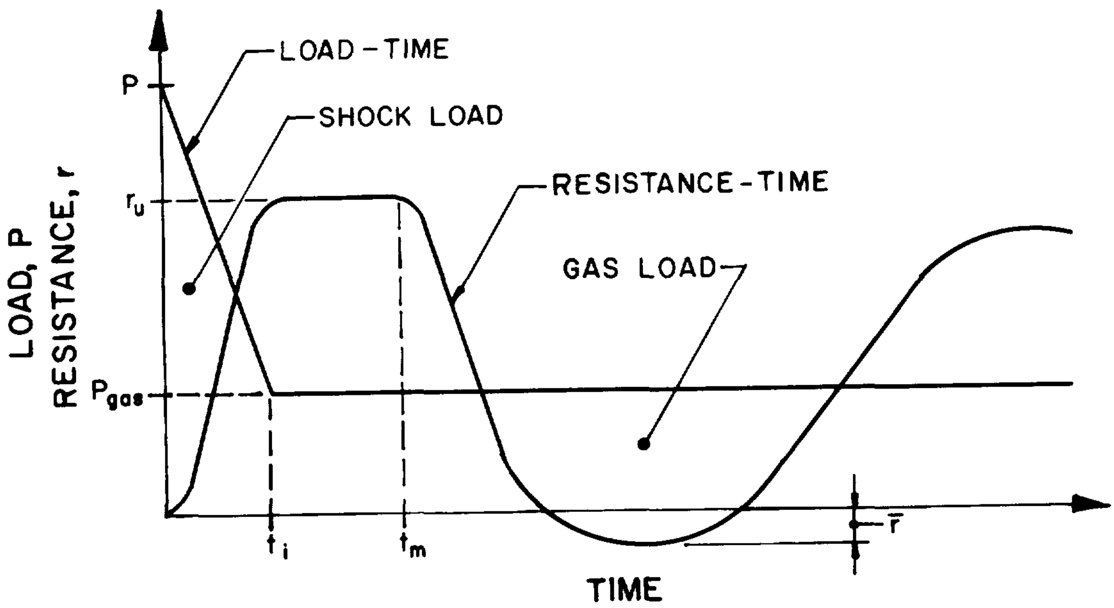


SECTION

LEGEND:

- (A) - STEEL FRAME EMBEDDED IN CONCRETE.
- (B) - CONTINUOUS GASKET
- (C) - BEARING BLOCK
- (D) - BLAST DOOR PLATE
- (E) - DOOR HINGE
- (F) - REVERSAL BOLT HOUSING
- (G) - REVERSAL BOLT
- (H) - BAR CONNECTED TO CLOSURE MECHANISM
- (I) - STEEL FRAME EQUIPPED WITH BLAST DOOR
- (J) - LIGHT GAGE COVER PLATE

Figure 5-21 Single-leaf blast door (high pressure)



P	=	PEAK LOAD
P_{gas}	=	GAS PRESSURE
r_u	=	PEAK POSITIVE RESPONSE
$\bar{r}$	=	REQUIRED REBOUND RESISTANCE
t_i	=	TIME AT WHICH SHOCK AND GAS LOADS INTERSECT
t_m	=	TIME TO REACH MAXIMUM RESPONSE

Figure 5-22 Bilinear blast load and single-degree-of-freedom response for determining rebound resistance

Table 5-9 Design Requirements for Sample Blast Doors

Door Description			Design Requirements								
			Permanent Deflections			Rebound Mechanisms		Fragment Protection		Level of Leakage Protection Required	
Door	Figure	Method of Opening	None	Limited	Large	Yes	No	Yes	No	Low	High
A	5-18	Double-Leaf		X		X			X	X	
B	5-19	Sliding			X		X		X	X	
C	5-20	Single-Leaf	X			X		X			X
D	5-21	Single-Leaf		X		X		X			X

COLUMNS AND BEAM COLUMNS

5-37. Plastic Design Criteria

5-37.1. General

The design criteria for columns and beam columns must account for their behavior not only as individual members but also as members of the overall frame structure. Depending on the nature of the loading, several design cases may be encountered. Listed below are the necessary equations for the dynamic design of steel columns and beam columns.

5-37.2. In-plane Loads

In the plane of bending of compression members which would develop a plastic hinge at ultimate loading, the slenderness ratio l/r shall not exceed the constant (C_c) defined as:

$$C_c = (2\pi^2 E/f_{ds})^{1/2} \qquad 5\text{-}41$$

where, E = modulus of elasticity of steel (psi)

f_{ds} = dynamic design stress (see Section 5-13)

The ultimate strength of an axially loaded compression member shall be taken as:

$$P_u = 1.7AF_a \qquad 5\text{-}42$$

where A = gross area of member,

$$F_a = \frac{(1 - (Kl/r)^2 / 2C_c^2)\, f_{ds}}{5/3 + 3(Kl/r)/8C_c - (Kl/r)^3 / 8C_c^3}\text{, and} \qquad 5\text{-}43$$

Kl/r = largest effective slenderness ratio listed in Table 5-10 or 5-11

5-37.3. Combined Axial Loads and Biaxial Bending

Members subject to combined axial load and biaxial bending moment should be proportioned so as to satisfy the following set of interaction formulas:

$$P/P_u + C_{mx}M_x /(1 - P/P_{ex})M_{mx} + C_{my}M_y /(1 - P/P_{ey})M_y \le 1.0 \qquad 5\text{-}44$$

$$P/P_p + M_x /1.18M_{px} + M_y /1.18M_{py} \le 1.0 \text{ for } P/P_p \ge 0.15 \qquad 5\text{-}45$$

or

$$M_x /M_{px} + M_y /M_{py} \le 1.0 \text{ for } P/P_p < 0.15 \qquad 5\text{-}46$$

where M_x, M_y = maximum applied moments about the x- and y-axes

P = applied axial load

$P_{ex} = 23/12AF'_{ex}$

P_{ey} = $23/12Af'_{ey}$

F'_{ex} = $12\pi^2E/[23(Kl_b/r_x)^2]$

F'_{ey} = $12\pi^2E/[23(Kl_b/r_y)^2]$

l_b = actual unbraced length in the plane of bending

r_x, r_y = corresponding radii of gyration

P_p = Af_{ds}

C_{mx}, C_{my} = coefficients applied to bending term in interaction formula and dependent upon column curvature caused by applied moments (AISC Specification, Section 1.6.1)

M_{px}, M_{py} = plastic bending capacities about x and y axes

$(M_{px} = Z_xf_{ds},\ M_{py} = Z_yf_{ds})$

M_{mx}, M_{my} = moments that can be resisted by the member in the absence of axial load.

For columns braced in the weak direction, $M_{mx} = M_{px}$ and $M_{my} = M_{py}$.

When columns are unbraced in the weak direction:

$$M_{mx} = [1.07 - (l/r_y)\ (f_{ds})^{1/2}\ /3160]\ M_{px} \le M_{px} \qquad 5\text{-}47$$

$$M_{my} = [1.07 - (l/r_x)\ (f_{ds})^{1/2}\ /3160]\ M_{py} \le M_{py} \qquad 5\text{-}48$$

Subscripts x and y indicate the axis of bending about which a particular design property applies. Also, columns may be considered braced in the weak direction when the provisions of Section 5-26 are satisfied. In addition, beam columns should also satisfy the requirements of Section 5-23.

5-38. Effective Length Ratios for Beam-columns

The basis for determining the effective lengths of beam columns for use in the calculation of P_u, P_{ex}, M_{mx}, and M_{my} in plastic design is outlined below.

For plastically designed braced and unbraced planar frames which are supported against displacement normal to their planes, the effective length ratios in Tables 5-10 and 5-11 shall apply.

Table 5-10 corresponds to bending about the strong axis of a member, while Table 5-11 corresponds to bending about the weak axis. In each case, l is the distance between points of lateral support corresponding to r_x or r_y, as applicable. The effective length factor, K, in the plane of bending shall be governed by the provisions of Section 5-40.

For columns subjected to biaxial bending, the effective lengths given in Tables 5-10 and 5-11 apply for bending about the respective axes, except that P_u for unbraced frames shall be based on the larger of the ratios Kl/r_x or

Kl/r_y. In addition, the larger of the slenderness ratios, l/r_x or l/r_y, shall not exceed C_c.

5-39. Effective Length Factor, K

In plastic design, it is usually sufficiently accurate to use the K factors from Table C1.8.1 of the AISC Manual (reproduced here as Table 5-12) for the condition closest to that in question rather than to refer to the alignment chart (Figure C.1.8.2 of AISC Manual). It is permissible to interpolate between different conditions in Table 5-12 using engineering judgment. In general, a design value of K equal to 1.5 is conservative for the columns of unbraced frames when the base of the column is assumed pinned, since conventional column base details will usually provide partial rotational restraint at the column base. For girders of unbraced frames, a design K value of 0.75 is recommended.

Table 5-10 Effective Length Ratios for Beam Columns (Webs of members in the plane of the frame; i.e., bending about the strong axis)

Braced Planar Frames*	One-and Two-Story Unbraced Planar Frames*
P_u Use larger ratio, l/r_y or l/r_x	Use larger ratio, l/r_y or Kl/r_x
P_{ex} Use l/r_x	Use Kl/r_x
M_{mx} Use l/r_y	Use l/r_y

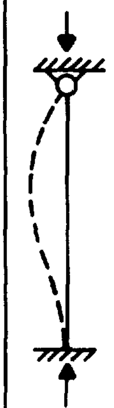
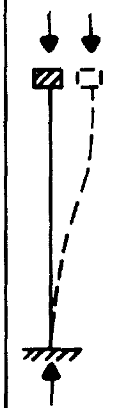
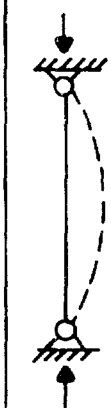
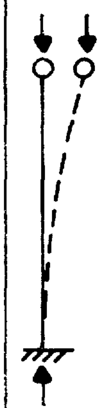
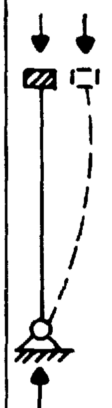
* l/r_x shall not exceed C_c.

Table 5-11 Effective Length Ratios for Beam Columns (Flanges of members in the plane of the frame; i.e., bending about the weak axis)

Braced Planar Frames*	One- and Two-Story Unbraced Planar Frames*
P_u Use larger ratio, l/r_y or l/r_x	Use larger ratio, l/r_x or Kl/r_y
P_{ey} Use l/r_y	Use Kl/r_y
M_{my} Use l/r_x	Use l/r_x

* l/r_y shall not exceed C_c.

Table 5-12 Effective Length Factors for Columns and Beam-columns

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line.						
Theoretical K value.	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated.	0.65	0.80	1.2	1.0	2.10	2.0
End condition code.		Rotation fixed and translation fixed.				
		Rotation free and translation fixed.				
		Rotation fixed and translation free.				
		Rotation free and translation free.				

FRAME DESIGN

5-40. General

The dynamic plastic design of frames for blast resistant structures is oriented toward industrial building applications common to ammunition manufacturing and storage facilities, i.e., relatively low, single-story, multi-bay structures. This treatment applies principally to acceptor structures subjected to relatively low blast overpressures.

The design of blast resistant frames is characterized by (a) simultaneous application of vertical and horizontal pressure-time loadings with peak pressures considerably in excess of conventional loads, (b) design criteria permitting inelastic local and overall dynamic structural deformations (deflections and rotations), and (c) design requirements dictated by the operational needs of the facility and the need for reusability with minor repair work after an incident must be considered.

Rigid frame construction is recommended in the design of blast resistant structures since this system provides open interior space combined with substantial resistance to lateral forces. In addition, this type of construction possesses inherent energy absorption capability due to the successive development of plastic hinges up to the ultimate capacity of the structure. However, where the interior since and wall opening requirements permit, it may be as effective to provide bracing.

The particular objective in this section is to provide rational procedures for efficiently performing the preliminary design of blast resistant frames. Rigid frames as well as frames with supplementary bracing and with rigid or nonrigid connections are considered. In both cases, preliminary dynamic load factors are provided for establishing equivalent static loads for both the local and overall frame mechanism. Based upon the mechanism method, as employed in static plastic design, estimates are made for the required plastic bending capacities as well as approximate values for the axial loads and shears in the frame members. The dynamic deflections and rotations in the sidesway and local beam mechanism modes are estimated based upon single degree-of-freedom analyses. The design criteria and the procedures established for the design of individual members previously discussed apply for this preliminary design procedure.

In order to confirm that a trial design meets the recommended deformation criteria of Table 5-8 and to verify the adequacy of the member sizes established on the basis of estimated dynamic forces and moments, a rigorous frame analysis should be performed. This analysis should consider the moments produced by the axial load deflection P-delta effects) in determining the sizes of individual elements. Several computer programs are available through the repositories listed in Section 5-4. These programs have the capability of performing a multi-degree-of-freedom, nonlinear, dynamic analysis of braced and unbraced, rigid and nonrigid frames of one or more story structures.

5-41. Trial Design of Single-Story Rigid Frames

5-41.1. Collapse Mechanisms

General expressions for the possible collapse mechanism of single-story rigid frames are presented in Table 5-13 for pinned and fixed base frames subjected to combined vertical and horizontal blast loads.

The objective of this trial design is to proportion the frame members such that the governing mechanism represents an economical solution. For a particular frame, the ratio of horizontal to vertical peak loading denoted by α is influenced by the horizontal frame plan of the structure and is determined as follows:

$$\alpha = q_h / q_v \qquad \text{5-49}$$

where

$q_v = p_v b_v$ = peak vertical load on rigid frame

$q_h = p_h b_h$ = peak horizontal load on rigid frame

p_v = blast overpressure on roof

p_h = reflected blast pressure on front wall

b_v = tributary width for vertical loading

b_h = tributary width for horizontal loading

The orientation of the roof purlins with respect to the blast load directions are shown in Figure 5-23. The value of α will usually lie in the range from about 1.8 to 2.5 when the direction of the blast load is perpendicular to the roof purlins. In this case, the roof purlins are supported by the frame and the tributary width is the same for the horizontal and vertical load. The value of α is much higher when the direction of the blast load is parallel to the roof purlins. In this case, the roof purlins are not supported by the girder of the frame and the tributary width of the vertical loading (b_v = purlin spacing) is much smaller than the tributary width of the horizontal loading (b_h = frame spacing).

It is assumed in this procedure that the plastic bending capacity of the roof girder, M_p, is constant for all bays. The capacity of the exterior and interior columns are taken as CM_p and C_1M_p, respectively. Since the exterior column is generally subjected to reflected pressures, it is recommended that a value of C greater than 1.0 be selected. In analyzing a given frame with certain member properties, the controlling mechanism is the one with the lowest resistance. In design, however, the load is fixed and the required design plastic moment is the largest M_p value obtained from all possible mechanisms. For that purpose, C and C_1 should be selected so as to minimize the value of the maximum required M_p from among all possible mechanisms. After a few trials, it will become obvious which choice of C and C_1 tends to minimize the largest value of M_p.

5-41.2. Dynamic Deflections and Rotations

It will normally be more economical to proportion the members so that the controlling failure mechanism is a combined mechanism rather than a beam mechanism. The mechanism having the least resistance constitutes an acceptable mode of failure provided that the magnitudes of the maximum deflections and rotations do not exceed the maximum values recommended in Table 5-8.

5-41.3. Dynamic Load Factors

To obtain initial estimates of the required mechanism resistance, the dynamic load factors listed in this section may be used to obtain equivalent static loads for the indicated mechanisms. These load factors are necessarily approximate and make no distinction for different end conditions. However, they are expected to result in reasonable estimates of the required resistance for a trial design. Once the trial member sizes are established, then the natural period and deflection of the frame can be calculated.

It is recommended that the DLF for a beam collapse mechanism be equal to 1.25 while that for a panel or combined collapse mechanism be equal to 0.625. The DLF for a frame is lower than that for a beam mechanism, since the natural period of vibration in the sidesway mode will normally be much greater than the natural periods of vibration of the individual elements.

5-41.4. Loads in Frame Members

Estimates of the peak axial forces in the girders and the peak shears in the columns are obtained from Figure 5-24. In applying the values of Figure 5-24, the equivalent horizontal static load shall be computed using the dynamic load factor for a panel or combined sidesway mechanism.

Preliminary values of the peak axial loads in the columns and the peak shears in the girders may be computed by multiplying the equivalent vertical static load by the roof tributary area. Since the axial loads in the columns are due to the reaction from the roof girders, the equivalent static vertical load should be computed using the dynamic load factor for the beam mechanism.

5-41.5. Sizing of Frame Members

Each member in a frame under the action of horizontal and vertical blast loads is subjected to combined bending moments and axial loads. However, the phasing between critical values of the axial force and bending moment cannot be established using a simplified analysis. Therefore, it is recommended that the peak axial loads and moments obtained from Figure 5-24 be assumed to act concurrently for the purpose of trial beam-column design. The overall resistance of the frame depends upon the ultimate strength of the members acting as beam-columns.

When an exterior frame of a building is positioned such that the shock front is parallel to frame, the loadings on each end of the building are equal and sideways action will only occur in the direction of the shock wave propagation. Frame action will also be in one direction, in the direction of the sidesway. If the blast wave impinges on a building from a quartering direction, then the columns and girders in the exterior frames are subjected to biaxial bending due to the simultaneous loads acting on the various faces of the structure. This action will also cause sidesway in both directions of the structure. The interior girders will usually be subjected to bending in one direction only. However, interior columns may be subjected to either uniaxial or biaxial bending, depending upon the column connections to the girder system. In such cases, the moments and forces can be calculated by analyzing the response of the frame in each direction and superimposing the respective moments and forces acting on the individual elements. This approach is generally conservative since it assumes that the peak values of the forces in one direction occur simultaneously throughout the three-dimensional structure.

Having estimated the maximum values of the forces and moments throughout the frame, the preliminary sizing of the members can be performed using the criteria previously presented for beams and columns.

5-41.6. Stiffness and Deflection

The stiffness factor K for single-story rectangular frames subjected to uniform horizontal loading is defined in Table 5-14. Considering an equivalent single degree-of-freedom system, the sidesway natural period of this frame is

$$T_N = 2\pi(m_e/KK_L)^{1/2} \qquad \text{5-50}$$

where K_L is a load factor that modifies K, the frame stiffness, due to a uniform load, so that the product KK_L is the equivalent stiffness due to a unit load applied at the equivalent lumped mass m_e.

The load factor is given by

$$K_L = 0.55\ (1 - 0.25\beta) \qquad \text{5-51}$$

where β is the base fixity factor and is equal to zero and one for pinned base and fixed base frames, respectively.

The equivalent mass m_e to be used in calculating the period of a sidesway mode consists of the total roof mass plus one-third of the column and wall masses. Since all of these masses are considered to be concentrated at the roof level, the mass factor, K_M, is equal to one.

The limiting resistance R_u is given by

$$R_u = \alpha wH \qquad \text{5-52}$$

where w is equal to the equivalent static uniform load based on the dynamic load factor for a panel or combined sidesway mechanism.

The equivalent elastic deflection X_E corresponding to R_u is

$$X_E = R_u\ /\ K_E \qquad \text{5-53}$$

Knowing the sidesway resistance R_u and the sidesway natural period of vibration T_n, the ductility ratio (μ) for the sidesway deflection of the frame can be computed using the dynamic response charts (Chapter 3). The maximum deflection X_m is then calculated from

$$X_m = \mu X_E \tag{5-54}$$

where

μ = ductility ratio in sidesway

5-42. Trial Design of Single-Story Frames with Supplementary Bracing

5-42.1. General

Frames with supplementary bracing can consist of (a) rigid frames in one direction and bracing in the other direction, (b) braced frames in two directions with rigid connections, and (c) braced frames in two directions with pinned connections. Most braced frames utilize pinned connections.

5-42.2. Collapse Mechanisms

The possible collapse mechanisms of single-story frames with diagonal tension bracing (X-bracing) are presented in Tables 5-15 and 5-16 for pinned-base frames with rigid and nonrigid girder-to-column connections, respectively. In these tables, the cross sectional area of the tension brace is denoted by A_b, the dynamic design stress for the bracing member is f_{ds}, and the number of braced bays is denoted by the parameter m. In each case, the ultimate capacity of the frame is expressed in terms of the equivalent static load and the member ultimate strength (either M_p or $A_b f_{ds}$). In developing these expressions in the tables, the same assumptions were made as for rigid frames, i.e., M_p for the roof girder is constant for all bays, the bay width L is constant, and the column moment capacity coefficient C is greater than 1.0.

For rigid frames with tension bracing, it is necessary to vary C, C_1, and A_b in order to achieve an economical design. When nonrigid girder to column connections are used, C and C_1 drop out of the resistance function for the sidesway mechanism and the area of the bracing can be calculated directly.

5-42.3. Bracing Ductility Ratio

Tension brace members are not expected to remain elastic under the blast loading. Therefore, it is necessary to determine if this yielding will be excessive when the system is permitted to deflect to the limits of the design criteria previously given.

The ductility ratio associated with tension yielding of the bracing is defined as the maximum strain in the brace divided by its yield strain. Assuming small deflections and neglecting axial deformations in the girders and columns, the ductility ratio is given by

$$\mu = \delta (\cos^2\gamma) E/Lf_{ds} \tag{5-55}$$

where

μ = ductility ratio

δ = sidesway deflection, inches

γ = vertical angle between the bracing and a horizontal plane are maximum slenderness requirements for the frame members. In general, values of A_b of about 2 square inches will result in a substantial increase in the overall resistance for frames with rigid connections. Hence, an assumed brace area in this range is recommended as a starting point. The design of the beams and columns of the frames follow the procedures previously presented.

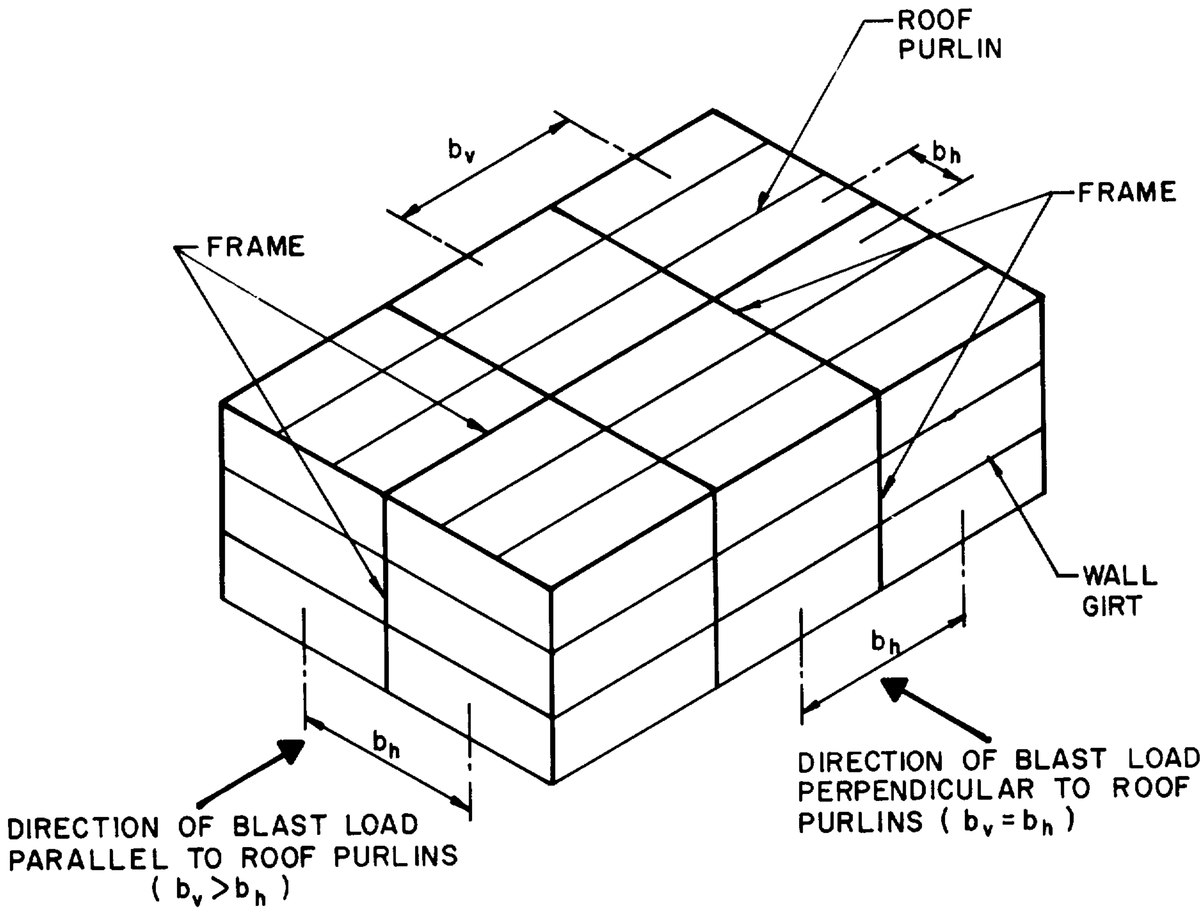
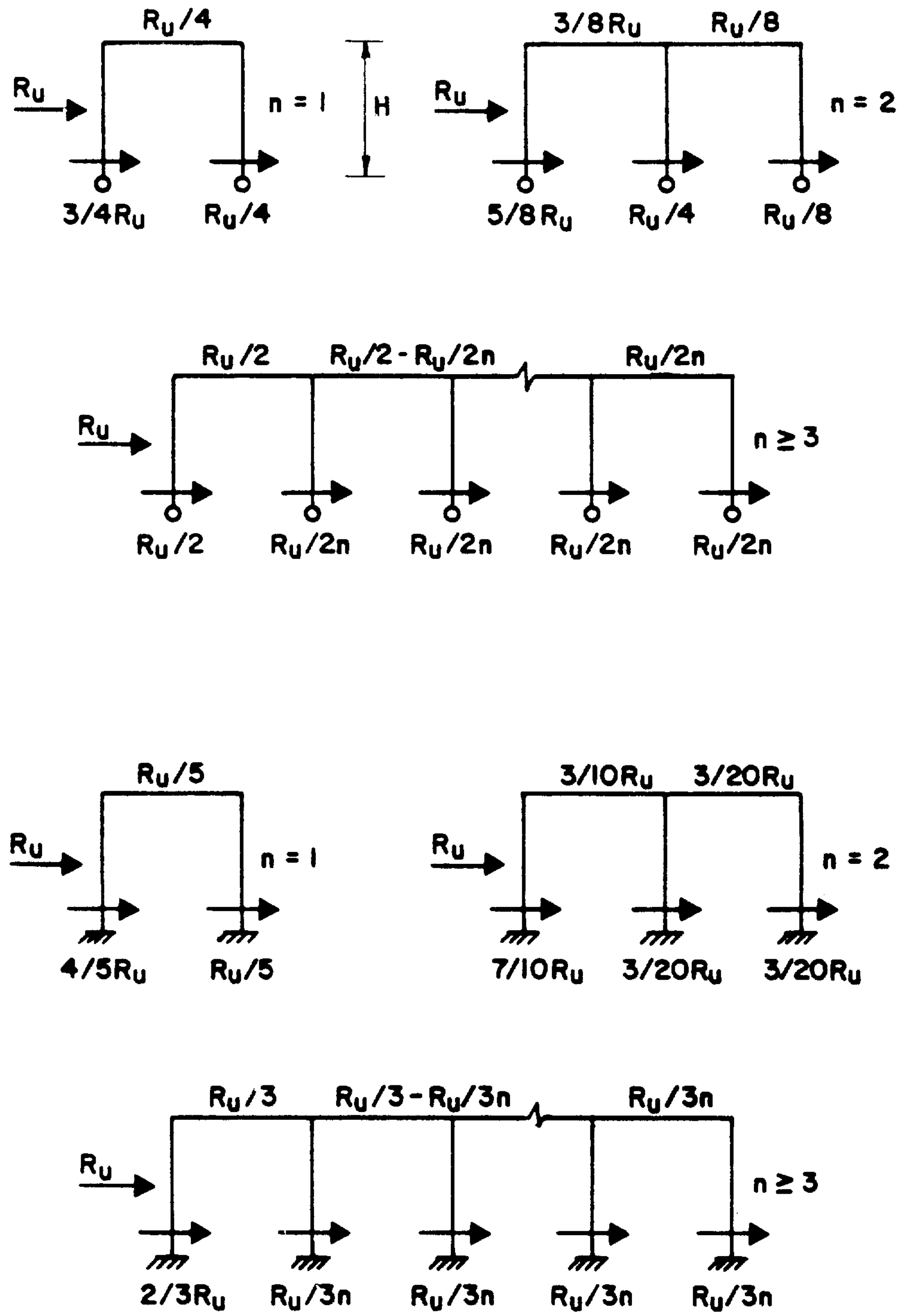


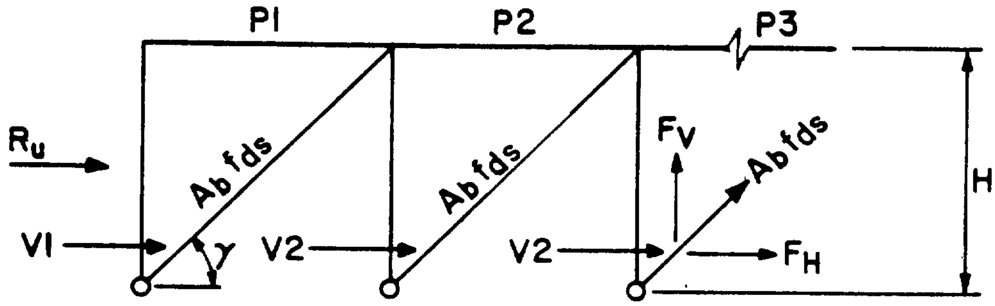
Figure 5-23 Orientation of roof purlins with respect to blast load direction for frame blast loading



n = NUMBER OF BAYS

$R_u = awH$ = EQUIVALENT HORIZONTAL STATIC LOAD

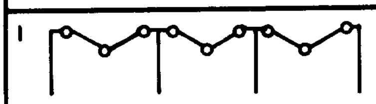
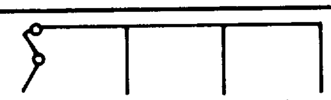
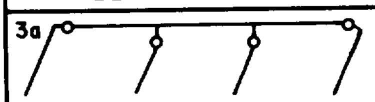
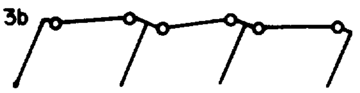
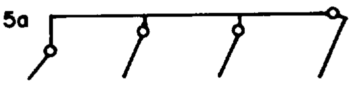
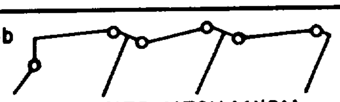
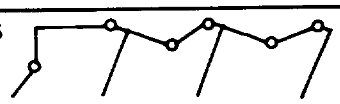
Figure 5-24 Estimates of peak shears and axial loads in rigid frames due to horizontal loads

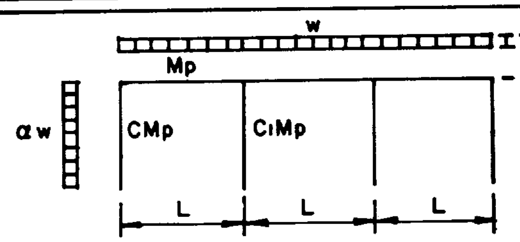


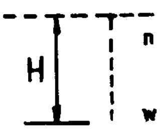
$$R_u = \alpha w H$$
$$V1 = R_u/2 + M_p/H$$
$$P1 = R_u/2 - M_p/H$$
$$P2 = P1 - V2 - F_H$$
$$F_H = A_b f_{ds} \cos \gamma$$
$$F_V = A_b f_{ds} \sin \gamma$$
$$V2 = R_u/2n - M_p/nH$$
$$P3 = P2 - V2 - F_H$$
$$P_n = P(n-1) - V2 - F_H$$

Figure 5-25 Estimates of peak shears and axial loads in braced frames due to horizontal loads

Table 5-13 Collapse Mechanisms for Rigid Frames with Fixed and Pinned Bases

COLLAPSE MECHANISM	PLASTIC MOMENT M_p	
	PINNED BASES	FIXED BASES
1  BEAM MECHANISM	$\frac{wL^2}{16}$	$\frac{wL^2}{16}$
2  BEAM MECHANISM	$\frac{\alpha w H^2}{4(2C+1)}$	$\frac{\alpha w H^2}{4(3C+1)}$
3a  PANEL MECHANISM	$\frac{\alpha w H^2}{2} \cdot \frac{1}{2+(n-1)C_1}$ $(C_1 \leq 2)^*$	$\frac{\alpha w H^2}{4} \cdot \frac{1}{1+(n-1)C_1+C}$ $(C_1 \leq 2)^*$
3b  PANEL MECHANISM	$\frac{\alpha w H^2}{4n}$ $(C_1 \geq 2)^*$	$\frac{\alpha w H^2}{2} \cdot \frac{1}{2(n+C)+(n-1)C_1}$ $(C_1 \geq 2)^*$
4  COMBINED MECHANISM	$\frac{w}{8n}(\alpha H^2+\frac{n}{2}L^2)$	$\frac{w}{2} \cdot \frac{\alpha H^2+\frac{n}{2}L^2}{2(2n+C)+(n-1)C_1}$
5a  COMBINED MECHANISM	$\frac{\frac{3}{8}\alpha w H^2}{C+\frac{1}{2}+\frac{C_1}{2}(n-1)}$ $(C_1 \leq 2)^*$	$\frac{\frac{3}{8}\alpha w H^2}{\frac{5}{2}C+(n-1)C_1+\frac{1}{2}}$ $(C_1 \leq 2)^*$
5b  COMBINED MECHANISM	$\frac{\frac{3}{8}\alpha w H^2}{C+(n-\frac{1}{2})}$ $(C_1 \geq 2)^*$	$\frac{\frac{3}{8}\alpha w H^2}{\frac{5}{2}C+(n-1)\frac{C_1}{2}+(n-\frac{1}{2})}$ $(C_1 \geq 2)^*$
6  COMBINED MECHANISM	$\frac{\frac{w}{8}[3\alpha H^2+(n-1)L^2]}{C+(2n-\frac{3}{2})}$	$\frac{\frac{w}{8}[3\alpha H^2+(n-1)L^2]}{\frac{5}{2}C+(n-1)\frac{C_1}{2}+(2n-\frac{3}{2})}$





n = NUMBER OF BAYS = 1, 2, 3 ......

w = UNIFORM EQUIVELENT STATIC LOAD

* FOR $C_1 = 2$ HINGES FORM IN THE GIRDERS AND COLUMNS AT INTERIOR JOINTS.

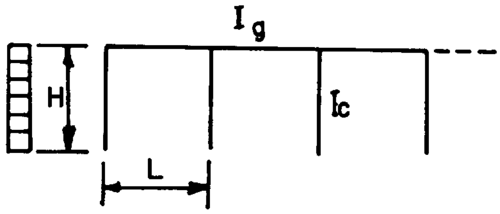
Table 5-14 Stiffness Factors for Single Story, Multi-bay Rigid Frames Subjected to Uniform Horizontal Loading

STIFFNESS FACTOR $K = \frac{E I_{ca}}{H^3} \cdot C_2 \cdot \left[1 + (0.7 - 0.1\beta)(n-1)\right]$

n = NUMBER OF BAYS

β = BASE FIXITY FACTOR†

$$D = \frac{I_g / L}{I_{ca}(0.75 + 0.25\beta)/H}$$



I_{ca} = AVERAGE COLUMN MOMENT OF INERTIA = $\sum I_c / (n+1)$

D	C_2		
	β = 1.0	β = 0.5*	β = 0
0.25	26.7	14.9	3.06
0.50	32.0	17.8	4.65
1.00	37.3	20.6	6.04

* VALUES OF C_2 ARE APPROXIMATE FOR THIS β

† β = 1.0 FOR FIXED BASE
= 0.0 FOR HINGED BASE

WHERE:

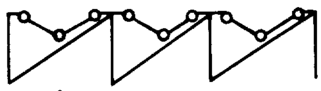
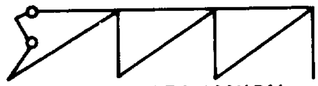
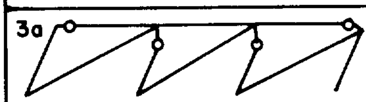
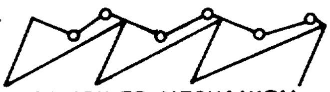
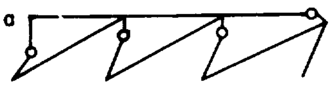
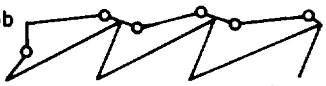
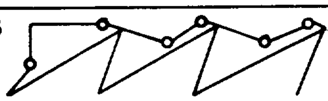
E = MODULUS OF ELASTICITY (PSI)

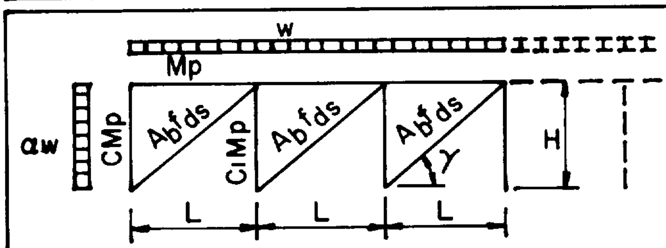
I_{ca}, I_g, I_c = MOMENT OF INERTIA ($IN.^4$)

H = HEIGHT (FEET)

L = BAY LENGTH (FEET)

Table 5-15 Collapse Mechanisms for Rigid Frames with Supplementary Bracing and Pinned Bases

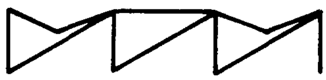
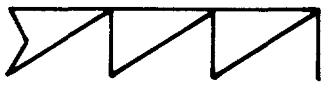
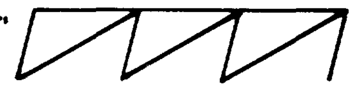
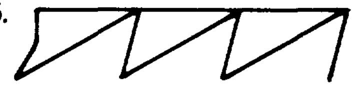
COLLAPSE MECHANISM	PLASTIC MOMENT M_p
1  BEAM MECHANISM	$\frac{wL^2}{16}$
2  BEAM MECHANISM	$\frac{\alpha w H^2}{4(2C+1)}$
3a  PANEL MECHANISM	$\left(\frac{\alpha wH^2}{2} - mA_b f_{ds} H\cos\gamma\right)\frac{1}{2+(n-1)C_1}$ $(C_1 \leq 2)$*
3b  PANEL MECHANISM	$\frac{\alpha wH^2}{4n} - \frac{mA_b f_{ds} H\cos\gamma}{2n}$ $(C_1 \geq 2)$*
4  COMBINED MECHANISM	$\frac{w}{8n}(\alpha H^2 + \frac{n}{2}L^2) - \frac{mA_b f_{ds} H\cos\gamma}{4n}$
5a  COMBINED MECHANISM	$\frac{\frac{3}{8}\alpha wH^2 - \frac{m}{2}A_b f_{ds} H\cos\gamma}{C+\frac{1}{2}+\frac{C_1}{2}(n-1)}$ $(C_1 \leq 2)$*
5b  COMBINED MECHANISM	$\frac{\frac{3}{8}\alpha wH^2 - \frac{m}{2}A_b f_{ds} H\cos\gamma}{C+(n-\frac{1}{2})}$ $(C_1 \geq 2)$*
6  COMBINED MECHANISM	$\frac{\frac{w}{8}[3\alpha H+(n-1)L] - \frac{m}{2}A_b f_{ds} H\cos\gamma}{C+(2n-\frac{3}{2})}$



m = NUMBER OF BRACED BAYS
n = NUMBER OF BAYS = 1,2,3. . .
w = UNIFORM EQUIVALENT STATIC LOAD

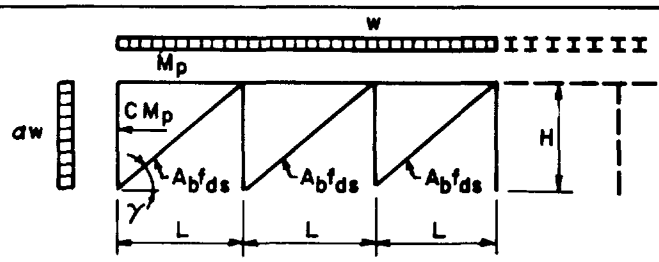
* FOR $C_1 = 2$ HINGES FORM IN THE GIRDERS AND COLUMNS AT INTERIOR JOINTS.

Table 5-16 Collapse Mechanisms for Frames with Supplementary Bracing, Non-rigid Girder-to-Column Connections and Pinned Bases

COLLAPSE MECHANISM	ULTIMATE CAPACITY	FRAMING TYPE
1.  BEAM MECHANISM EXTERIOR GIRDER	$M_p = wL^2/8$ $M_p = wL^2/12$ $M_p = wL^2/16$	1 2 3
2.  BEAM MECHANISM INTERIOR GIRDER	$M_p = wL^2/8$ $M_p = wL^2/16$	1 2 & 3
3.  BEAM MECHANISM BLASTWARD COLUMN	$M_p = \alpha w H^2/8$ $M_p = \dfrac{\alpha w H^2}{4(2C+1)}$	1 & 2 3
4.  PANEL MECHANISM	$A_b f_{ds} = \alpha w H / 2m \cos\gamma$ $A_b f_{ds} = \dfrac{\alpha w H}{2m\cos\gamma} - \dfrac{2M_p}{mH\cos\gamma}$	1 & 2 3
5.  COMBINED MECHANISM	$A_b f_{ds} = \dfrac{3\alpha w H}{4m\cos\gamma} - \dfrac{(2C+1)M_p}{mH\cos\gamma}$	3

GIRDER FRAMING TYPE:

1. GIRDER SIMPLY SUPPORTED BETWEEN COLUMNS
2. GIRDER CONTINUOUS OVER COLUMNS
3. GIRDER CONTINUOUS OVER COLUMNS AND RIGIDLY CONNECTED TO EXTERIOR COLUMNS ONLY



m = NUMBER OF BRACED BAYS

w = UNIFORM EQUIVALENT STATIC LOAD

CONNECTIONS

5-43. General

The connections in a steel structure designed in accordance with plastic design concepts must fulfill their function up to the ultimate load capacity of the structure. In order to allow the members to reach their full plastic moments, the connections must be capable of transferring moments, shears and axial loads with sufficient strength, proper stiffness and adequate rotation capacity.

Connections must be designed with consideration of economical fabrication and ease of erection. Connecting devices may be rivets, bolts, welds, screws or various combinations thereof.

5-44. Types of Connections

The various connection types generally encountered in steel structures can be classified as primary member connections, secondary member connections and panel attachments. Primary member connections are corner frame, beam-to-column, beam-to-girder and column base connections as well as splices. Secondary member connections are purlin-to-frame, girt-to-frame and bracing connections. Panel attachments are roof-to-floor panel and wall siding connections.

Primary member connections refer to those used in design and construction of the framing of primary members. They generally involve the attachment of hot-rolled sections to one another, either to create specific support conditions or to achieve continuity of a member or the structure. In that respect, connections used in framing may be classified into three groups, namely, rigid, flexible (nonrigid) and semirigid, depending upon their degree of restraint which is the ratio of the actual end moment that may be developed to the end moment in a fully fixed-ended beam. Approximately, the degree of restraint is generally considered as over 90 percent for rigid connections, between 20 to 90 percent for semirigid connections and below 20 percent for flexible connections.

It should be mentioned that the strength and rotation characteristics of semirigid connections are dependent upon the properties of the intermediate connection elements (angles, plates, tees) and thus, are subject to much variation. Since semirigid structural analyses are seldom undertaken due to their great complexity, no further details on semirigid connections will be given here.

Secondary member connections are used to fasten members such as purlins, girts or bracing members to the primary members of a frame, either directly or by means of auxiliary sections such as angles and tees.

Basic requirements for primary and secondary member connections, as well as general guidelines for proper design, are presented in Sections 5-45 and 5-46. In addition, dynamic design stresses to be used in the selection and sizing of fastening devices are given in Section 5-47.

Panel attachments are used to attach elements of the skin or outer shell of an installation as well as floor and wall panels to the supporting skeleton.

Connections of this type are distinguished by the fact that they fasten relatively thin sheet material to one another or to heavier rolled sections. Roof decks and wall siding have to withstand during their lifetime (apart from accidental blast loads) exposure to weather, uplift forces, buffeting and vibration due to winds, etc. For this reason, and because of their widespread use, special care should be taken in design to ensure their adequate behavior. Some basic requirements for panel connections are presented in Section 5-48.

5-45. Requirements for Main Framing Connections

The design requirements for frame connections may be illustrated by considering the behavior of a typical corner connection as shown in Figure 5-26. Two members are joined together without stiffening of the corner web. Assuming that the web thickness is insufficient, the behavior of the connection is represented by Curve 1 which shows that yielding due to shear force starts at a relatively low load. Even though the connection rotates past the required hinge rotation, the plastic moment M_p is not reached. In addition, the elastic deformations are also larger than those assumed by the theoretical design curve.

A second and different connection may behave as indicated by Curve 2. Although the elastic stiffness is satisfactory and the maximum capacity exceeds M_p, the connection failed before reaching the required hinge rotation and thus, is unsatisfactory.

These considerations indicate that connections must be designed for strength, stiffness and rotation capacity. They must transmit the required moment, shear and axial load, and develop the plastic moment M_p of the members.

Normally, an examination of a connection to see if it meets the requirements of stiffness will not be necessary. Compared to the total length of the member, the length of the connection is small, and, if the connection is slightly more flexible than the member which it joins, the general effect on the structural behavior is not great. Approximately, the average unit rotation of the connecting zone should not exceed that of an equivalent length of the members being joined.

Of equal importance with the strength of the connection is an adequate reserve of ductility after the plastic moment has been attained. Rotation capacity at plastic hinge locations is essential to the development of the full ultimate load capacity of the structure.

5-46. Design of Connections

It is not the intent of this section to present procedures and equations for the design of the various types of connections likely to be encountered in the blast-resistant design of a steel structure. Instead, the considerations necessary for a proper design will be outlined.

After completion of the dynamic analysis of the structure, the members are sized for the given loadings. The moments, shears, and axial loads at the connections are known. Full recognition must be given to the consideration of rebound or stress reversal in designing the connections. Additionally, in continuous structures, the maximum values of P, M, and V may not occur simultaneously and thus, several combinations may have to be considered.

With rigid connections such as a continuous column-girder intersection, the web area within the boundaries of the connection should meet the shear stress requirements of Section 5-23. If the web area is unsatisfactory, diagonal stiffeners or web doubler plates should be provided.

Stiffeners will normally be required to prevent web crippling and preserve flange continuity wherever flange-to-flange connections occur at columns in a continuous frame. Web crippling must also be checked at points of load application such as beam-girder intersections. In these cases, the requirements of Section 5-25 of this chapter and Sections 1.10.5 and 1.10.10 of the AISC Specification must be considered.

Since bolted joints will develop yield stresses only after slippage of the members has occurred, the use of friction-type bolted connections is not recommended.

5-47. Dynamic Design Stresses for Connections

In accordance with Section 2.8 of the AISC Specification, bolts, rivets and welds shall be proportioned to resist the maximum forces using stresses equal to 1.7 times those given in Part 1 of the Specification. Additionally, these stresses are increased by the dynamic increase factor specified in Section 5-12.2; hence,

$$f_d = 1.7cf_s \qquad 5\text{-}63$$

where f_d = the maximum dynamic design stress for connections

c = the dynamic increase factor Figure 5-2 or Table 5-2)

f_s = the allowable equivalent static design stress of the bolt, rivet, or weld

Rather than compiling new tables for maximum dynamic loads for the various types of connections, the designer will find it advantageous to divide the forces being considered by the factor 1.7c and then to refer to the allowable load tables in Part 1 of the AISC Specification.

5-48. Requirements for Floor and Wall Panel Connections

Panel connections, in general, can be considered either panel-to-panel connections, or panel-to-supporting-frame connections. The former type involves the attachment of relatively light-gage materials to each other such that they act together as an integral unit. The latter type is generally used to attach sheet panels to heavier cross sections.

The most common type of fastener for decking and steel wall panels is the self-tapping screw with or without washer. Even for conventional design and regular wind loading, the screw fasteners have often been the source of local failure by tearing the sheeting material. It is evident that under blast loading and particularly on rebound, screw connectors will be even more vulnerable to this type of failure. Special care should be taken in design to reduce the probability of failure by using oversized washers and/or increased material thickness at the connection itself.

Due to the magnitude of forces involved, special types of connectors, as shown in Figure 5-27, will usually be necessary. These may consist of self-piercing, self-tapping screws of larger diameters with oversized washers, puddle welds or washer plug welds, threaded connectors fired into the elements to be attached, or threaded studs, welded to the supporting members, which fasten the panel by means of a special arrangement of bushing and nut.

Apart from fulfilling their function of cladding and load-resisting surfaces, by carrying loads perpendicular to their surface, floor, roof and wall, steel panels can, when adequately connected, develop substantial resistance to in-plane forces, acting as diaphragms contributing a great deal to the overall stiffness and stability of the structure. As a result, decking connections are, in many cases, subjected to a combination of shearing forces and pull-out forces. It is to be remembered also that after the panel has deflected under blast loading, the catenary action sustained by the flat sheet of the decking represents an important reserve capacity against total collapse. To allow for such catenary action to take place, connectors and especially end connectors should be made strong enough to withstand the membrane forces that develop.

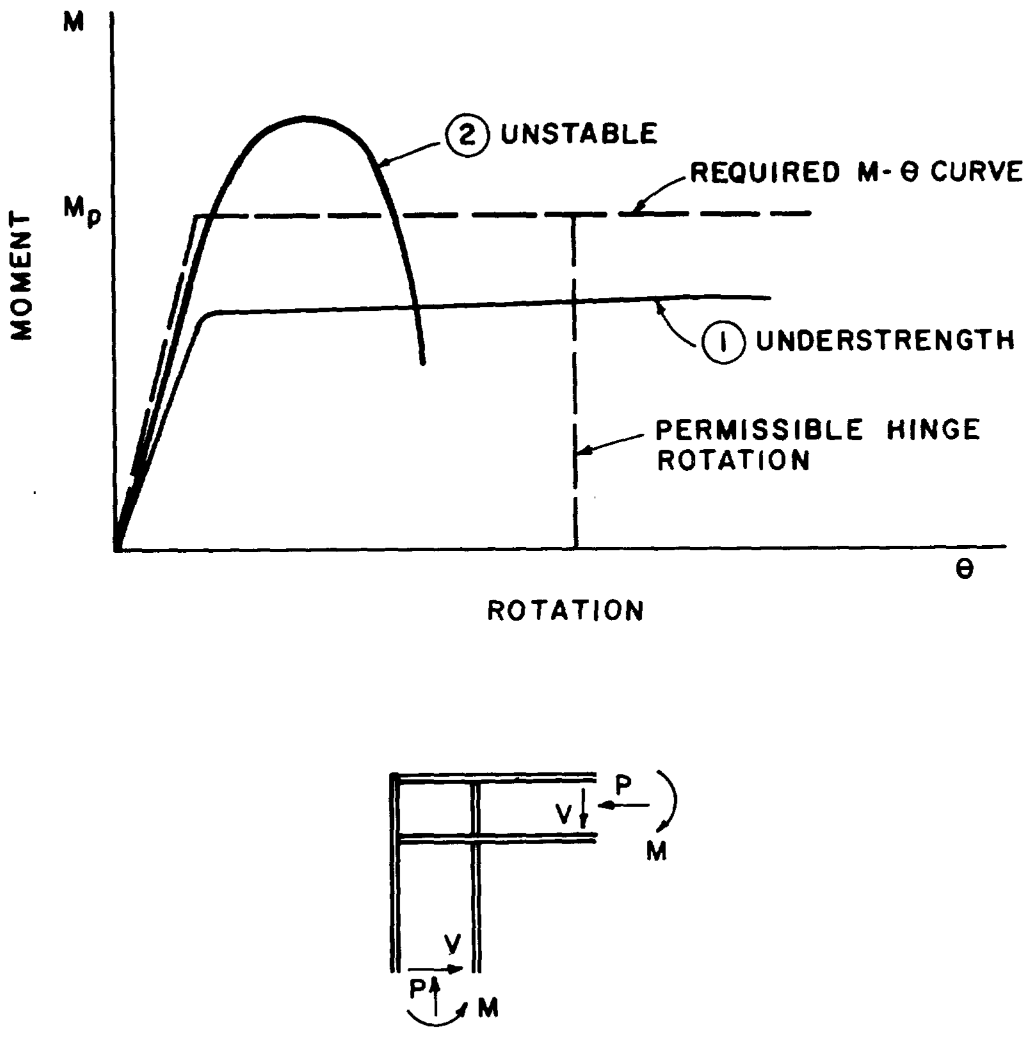


Figure 5-26 Corner connection behavior

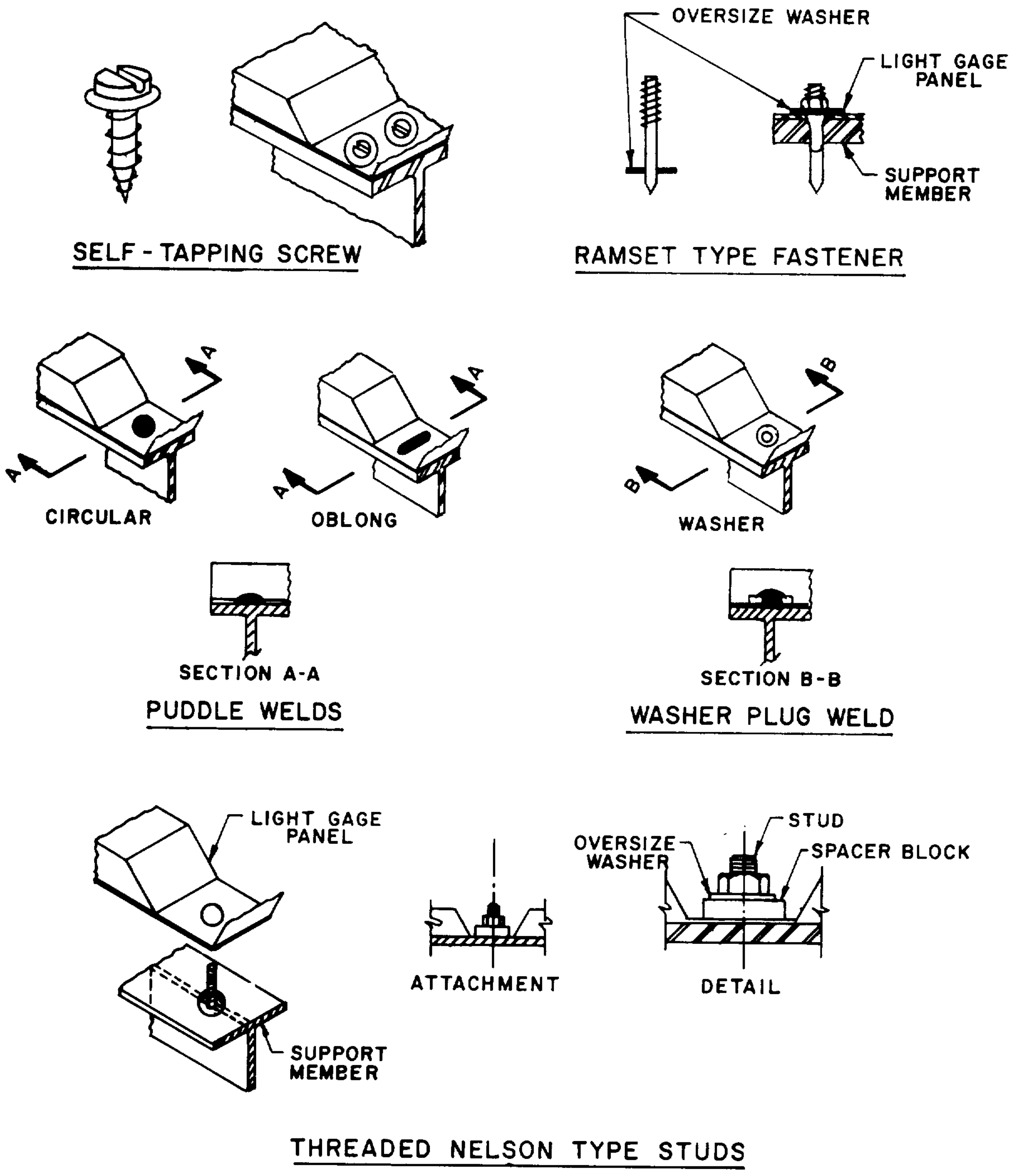


Figure 5-27 Typical connections for cold-formed steel panels

FRAGMENT PENETRATION

5-49. Penetration of Fragments into Steel

5-49.1. Failure Mechanisms

In deriving a prediction equation for the penetration and perforation of steel plates, it is important to recognize the failure mechanisms. The failure mode of primary concern in mild to medium hard homogeneous steel plates subjected to normal impact is ductile failure. In this mode, as the missile penetrates the plate, plastically deformed material is pushed aside and petals or lips are formed on both the front and back faces with no material being ejected from the plate. For plates with Brinell hardness values above 300, failure by "plugging" is a strong possibility. In this brittle mode of failure, a plug of material is formed ahead of the penetrating missile and is ejected from the back side of the plate. A third mode of failure is disking or flaking, in which circular disks or irregular flakes are thrown from the back face. This type of failure is mainly a concern with plates of inferior quality steel and should not, therefore, be a common problem in the design of protective structures.

5-49.2. Primary Fragment Penetration Equations

In protective design involving primary fragments, a penetration equation is required which yields reliable estimates corresponding to the standard primary fragment illustrated in figure 4-77 of Chapter 4. These design equations consider only normal penetration which is critical for the design of protective structures. These equations apply to penetration into mild steel and are conservative for plates with a Brinell hardness value above 150. Steel penetration equations in design for primary fragment impact are expressed in the following forms:

For AP steel fragments penetrating mild steel plates,

$$x = 0.30\ W_f^{0.33}\ V_s^{1.22} \qquad 5\text{-}64$$

and for mild steel fragments penetrating mild steel plates,

$$x = 0.21\ W_f^{0.33}\ V_s^{1.22} \qquad 5\text{-}65$$

where

x = depth of penetration (in.)

W_f = fragment weight (oz.)

V_s = striking velocity of fragment (kfps)

Charts for steel penetration by primary fragments according to these equations are presented in figures 5-28 and 5-29.

To estimate the penetration of metal fragments other than armor piercing, the procedures outlined in Section 4-60.3 of Chapter 4 are entirely applicable to steel plates.

5-49.3. Residual Velocity After Perforation of Steel Plate

The penetration equations presented in Section 5-49.2 may be used for predicting the occurrence of perforation of metallic barriers and for calculating the residual fragment velocity after perforation.

For normal impact of a steel fragment, with the shape illustrated in figure 4-77 of Chapter 4, the equation for residual velocity is

$$V_r / V_s = [1 - (V_x / V_s)^2]^{1/2} / (1 + t/d) \qquad 5\text{-}66$$

where V_r = residual velocity

V_s = striking velocity

V_x = critical perforation velocity for the fragment of impacting the plate of thickness t (see explanation below)

d = diameter of cylindrical portion of fragment (in.), as illustrated in figure 4-77 of Chapter 4

The value of V_x is determined from Figure 5-28 or 5-29 by substituting the plate thickness t for the penetration depth x and reading the corresponding value of striking velocity, V_s. This striking velocity becomes the critical perforation velocity, V_x. A plot of the residual velocity equation for a range of t/d ratios is presented in Figure 5-30.

Multiple plate penetration problems may be analyzed by the successive application of Equations 5-64 or 5-65 for predicting the depth of penetration and Equation 5-66 for calculating the residual velocity upon perforation of the plate. In addition, composite construction, consisting of concrete walls with attached spall plates, can be analyzed for fragment impact by tracing the motion of the fragment through each successive layer. The striking velocity of the fragment upon each intermediate layer is the residual fragment velocity after perforation of the previous layer. The conservative assumptions are made that the fragment remains intact during the penetration and that it does not deviate from a straight line path as it crosses the interface between the different media.

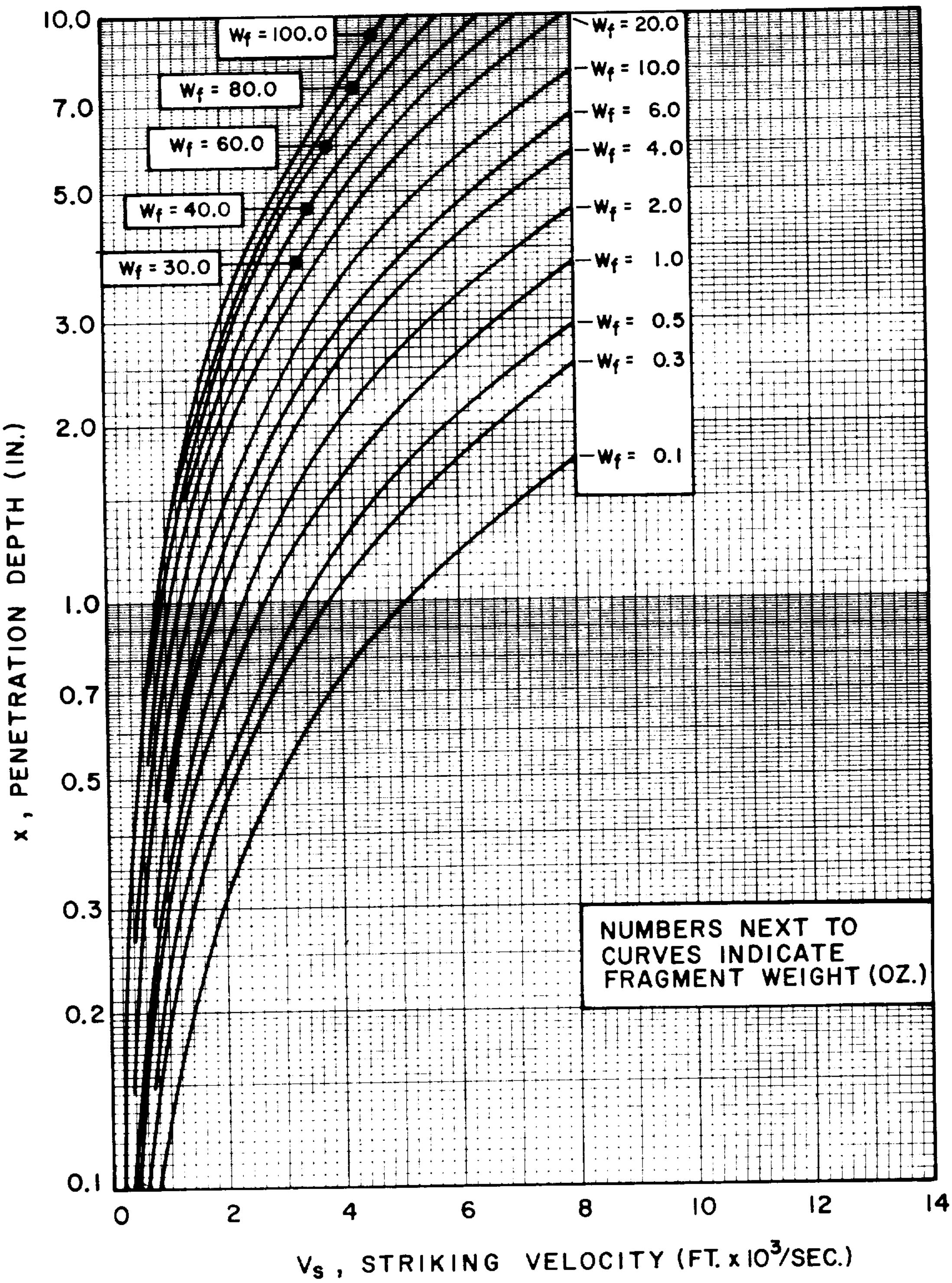


Figure 5-28 Steel penetration design chart - AP steel fragments penetrating mild steel plates

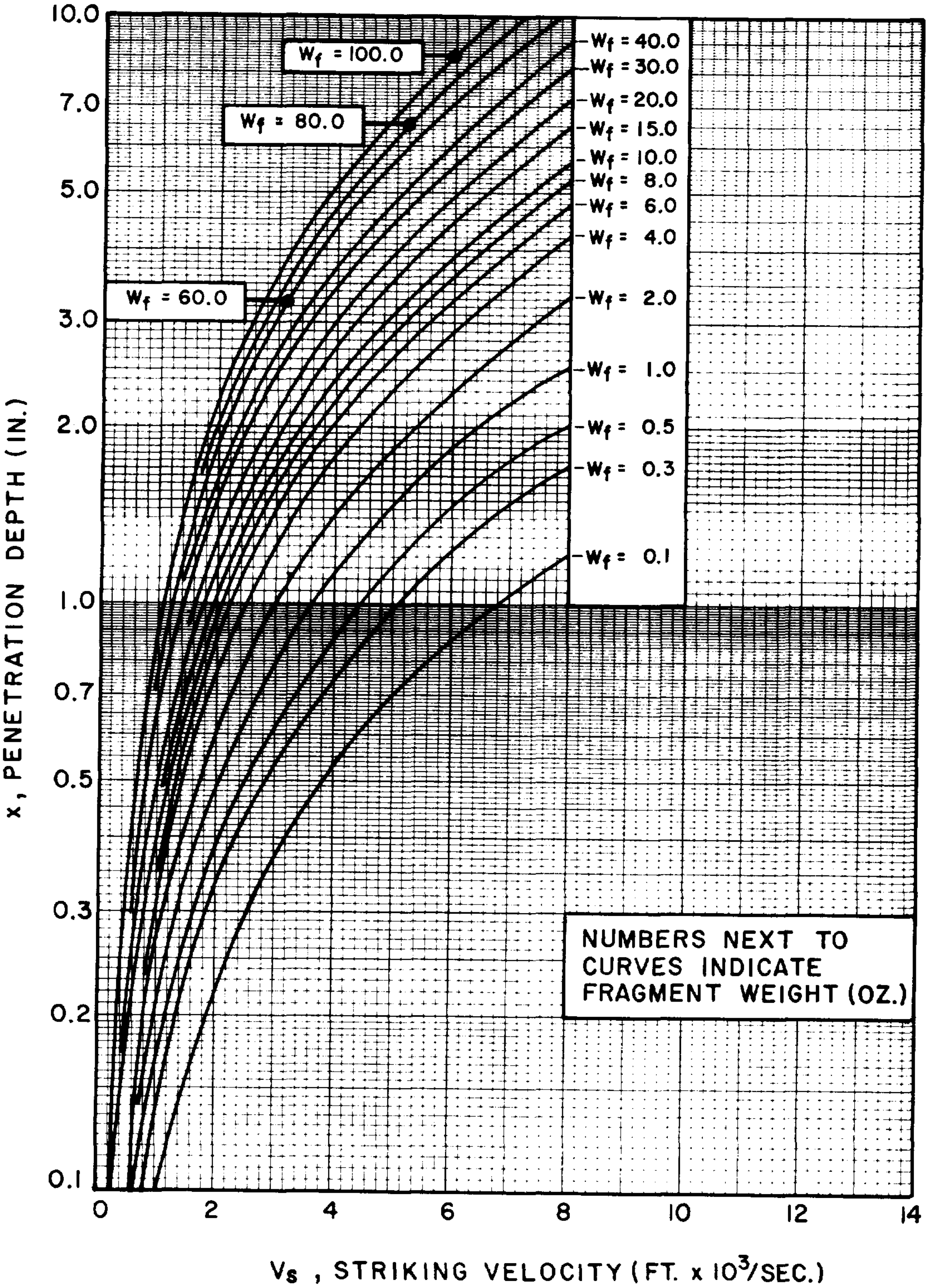


Figure 5-29 Steel penetration design chart - mild steel fragments penetrating mild steel plates

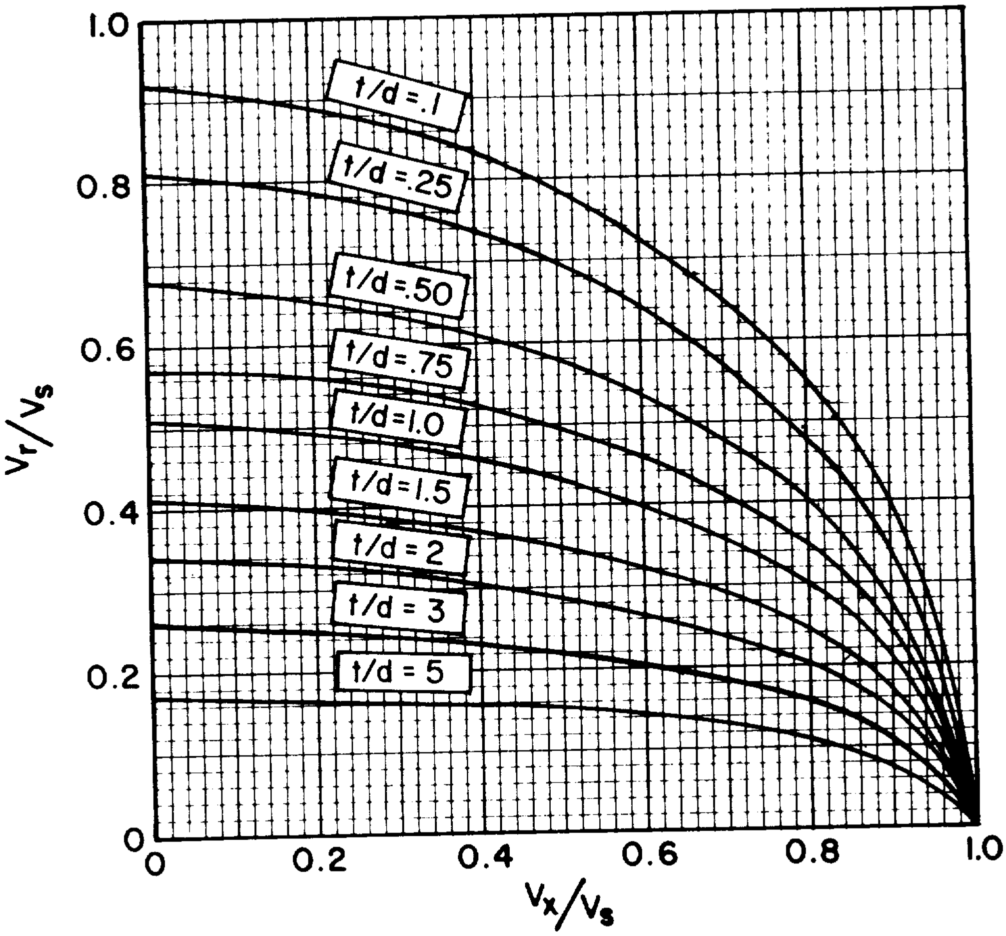


Figure 5-30 Residual fragment velocity upon perforation of steel barriers

TYPICAL DETAILS FOR BLAST-RESISTANT STEEL STRUCTURES

5-50. General

This section presents several examples of typical framing connections, structural details and blast doors used in industrial installations designed to resist accidental blast loadings. This section is intended to augment those details presented in prior sections of this chapter.

5-51. Steel Framed Buildings

Such buildings are often rectangular in plan, two or three bays wide and four or more bays long. Figure 5-31 shows an example of a typical framing plan for a single-story building designed to resist a pressure-time blast loading impinging on the structure at an angle with respect to its main axes. The structural system consists of an orthogonal network of rigid frames. The girders of the frames running parallel to the building length serve also as purlins and are placed, for ease of erection, on top of the frames spanning across the structure's width.

Figures 5-32 to 5-35 present typical framing details related to the general layout of Figure 5-31. As a rule, the columns are fabricated without splices, the plate covers and connection plates are shop welded to the columns, and all girder to column connections are field bolted. A channel is welded on top of the frame girders to cover the bolted connections and prevent (avoid) interference with the roof decking. All of the framing connections are designed to minimize stress concentrations and to avoid triaxial strains. They combine ductility with ease of fabrication.

5-52. Cold-formed, Light Gage Steel Panels

Figure 5-36 shows typical cross sections of cold-formed, light gage steel panels commonly used in industrial installations. The closed sections, which are composed of a corrugated hat section and a flat sheet, are used to resist blast pressures in the low pressure range, whereas the open hat section is recommended only for very low pressure situations as siding or roofing material. A typical vertical section illustrates the attachment of the steel paneling to the supporting members. Of particular interest is the detail at the corner between the exterior wall and the roof, which is designed to prevent peeling of the decking that may be caused by negative pressures at the roof edge.

Figure 5-37 gives some typical arrangements of welded connections for attaching cold-formed steel panels to their supporting elements. Type A refers to an intermediate support whereas Type B refers to an end support. It is recommended that the diameter of puddle welds be 3/4 of an inch minimum and should not exceed 1-1/2 inches because of space limitations in the panel valleys. For deeper panels, it is often necessary to provide two rows of puddle welds at the intermediate supports in order to resist the uplift forces in rebound. It should be noted that welds close to the hooked edge of the panel are recommended to prevent lifting of adjacent panels.

Figure 5-38 shows an arrangement of bolted connections for the attachment of cold-formed steel panels to the structural framing. The bolted conne tion consists of: a threaded stud resistance welded to the supporting memb r, a square steel block with a concentric hold used as a spacer, and a washer and nut for fastening. Figure 5-39 presents a cross section of that connection with all the relevant details along with information pertaining to puddle welds.

5-53. Blast Doors

Figures 5-40 and 5-41 show details of single-leaf and double-leaf blast doors, respectively. Figure 5-40 presents a single-leaf door installed in a steel structure. The design is typical of doors intended to resist relatively low pressure levels. It is interesting to note that the door is furnished with its tubing frame to ensure proper fabrication and to provide adequate stiffness during erection. In Figure 5-41, the double-leaf door with its frame is installed in place and attached to the concrete structure. In both figures details of hinges, latches, anchors, and panic hardware are illustrated. It should be noted that the pins at the panic latch ends are made of aluminum in order to eliminate the danger of sparking, a hazard in ammunition facilities which might arise from steel-on-steel striking.

Figure 5-42 shows details of compression arch and tension arch doors. The tension arch door requires compression ties to develop the compression reactions for the arch and to prevent the door from being blown through the opening. The compression arch door requires a tension tie plate to develop the reactions and to prevent large distortions in the door that may bind it in place.

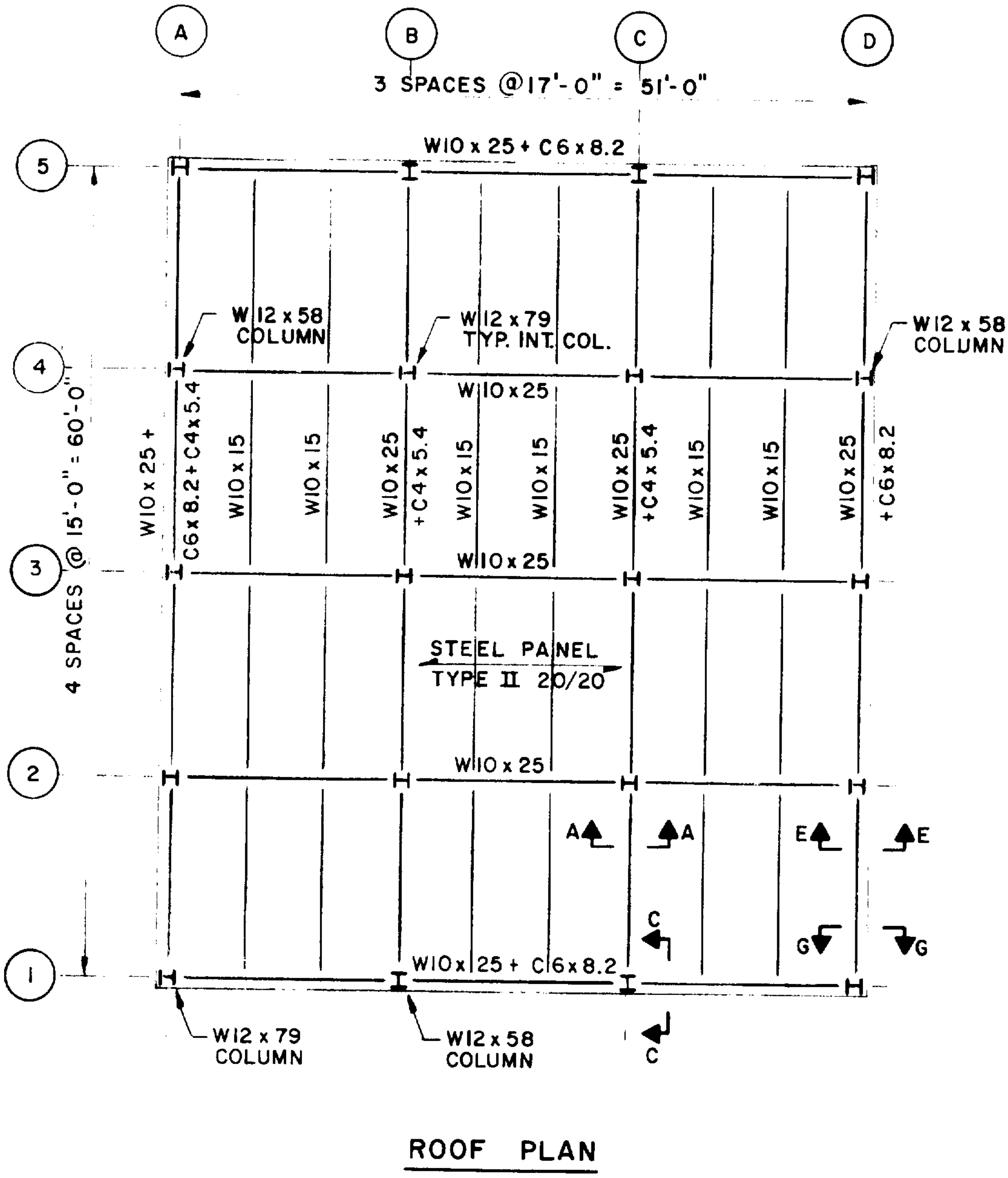
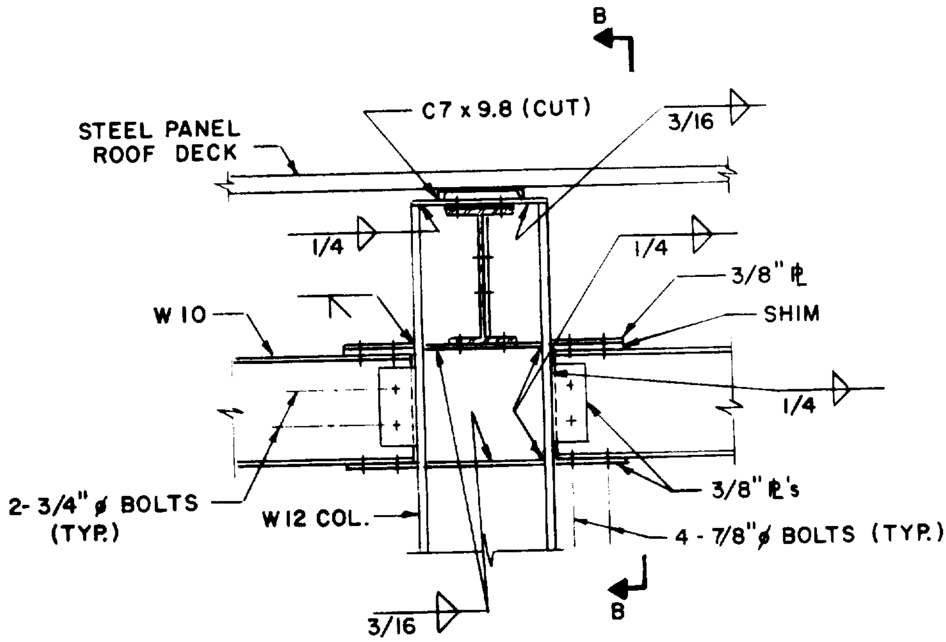
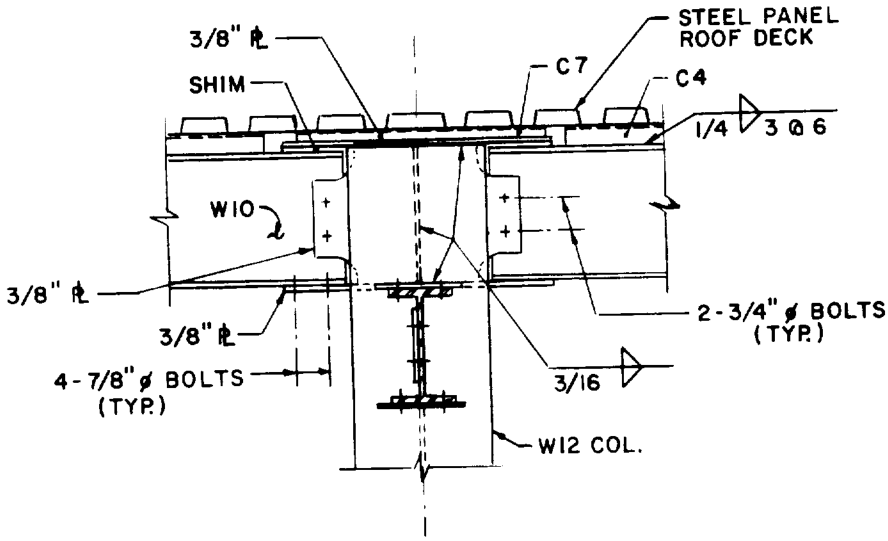


Figure 5-31 Typical framing plan for a single-story blast-resistant steel structure

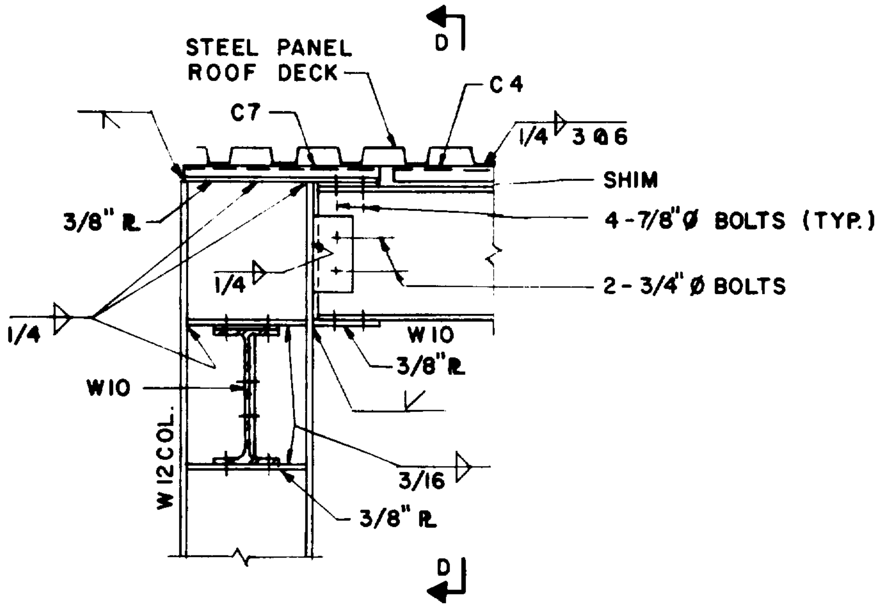


SECTION A-A

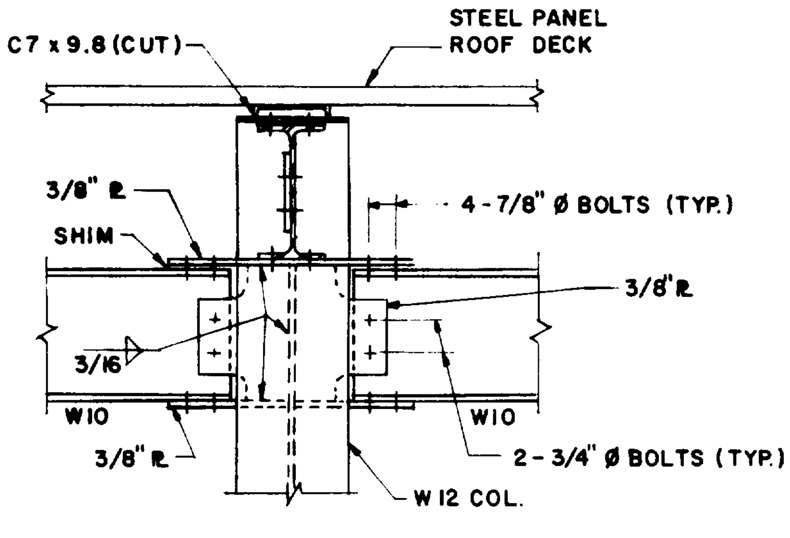


SECTION B-B

Figure 5-32 Typical framing detail at interior Column 2-C

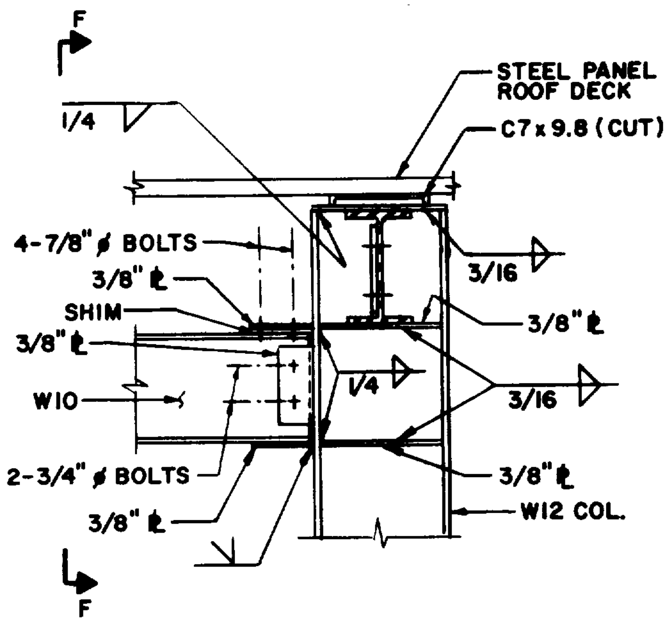


SECTION C-C

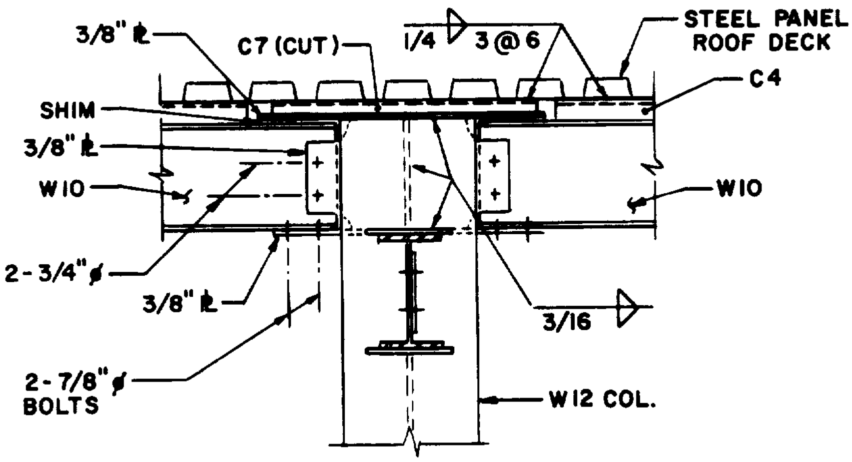


SECTION D-D

Figure 5-33 Typical framing detail at end Column 1-C



SECTION E-E



SECTION F-F

Figure 5-34 Typical framing detail at side column 2-D

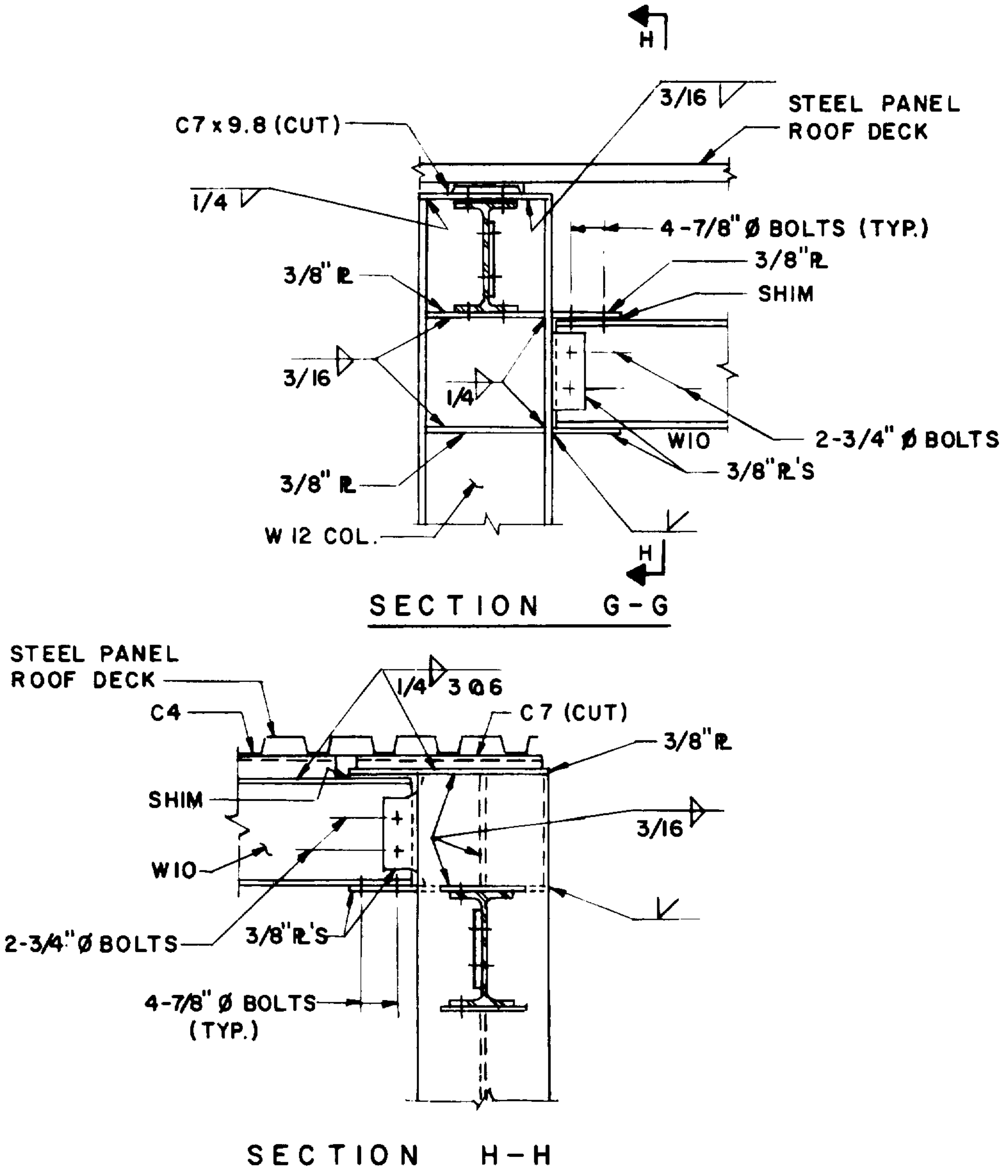


Figure 5-35 Typical framing detail at corner Column 1-D

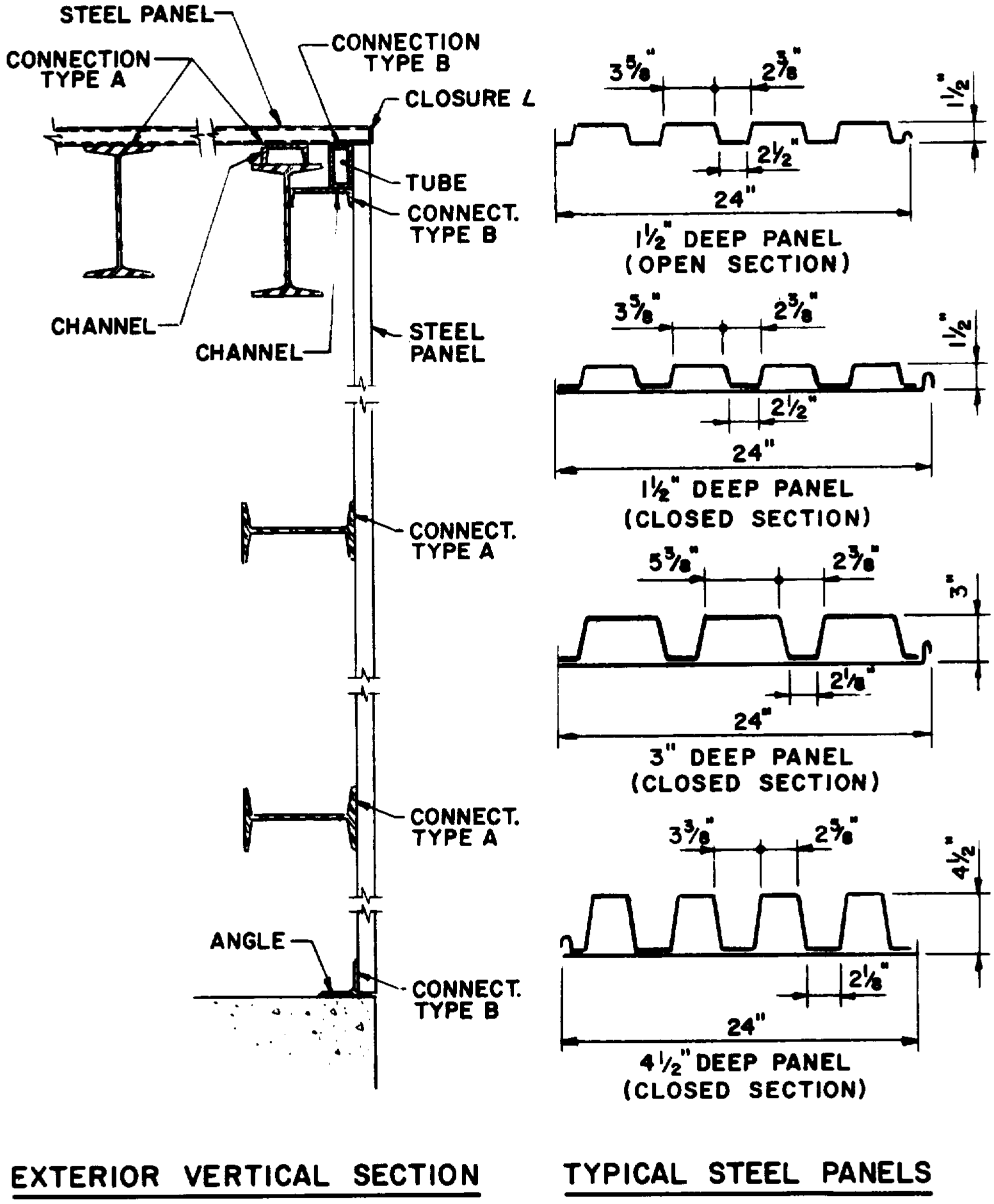


Figure 5-36 Typical details for cold-formed, light gage steel paneling

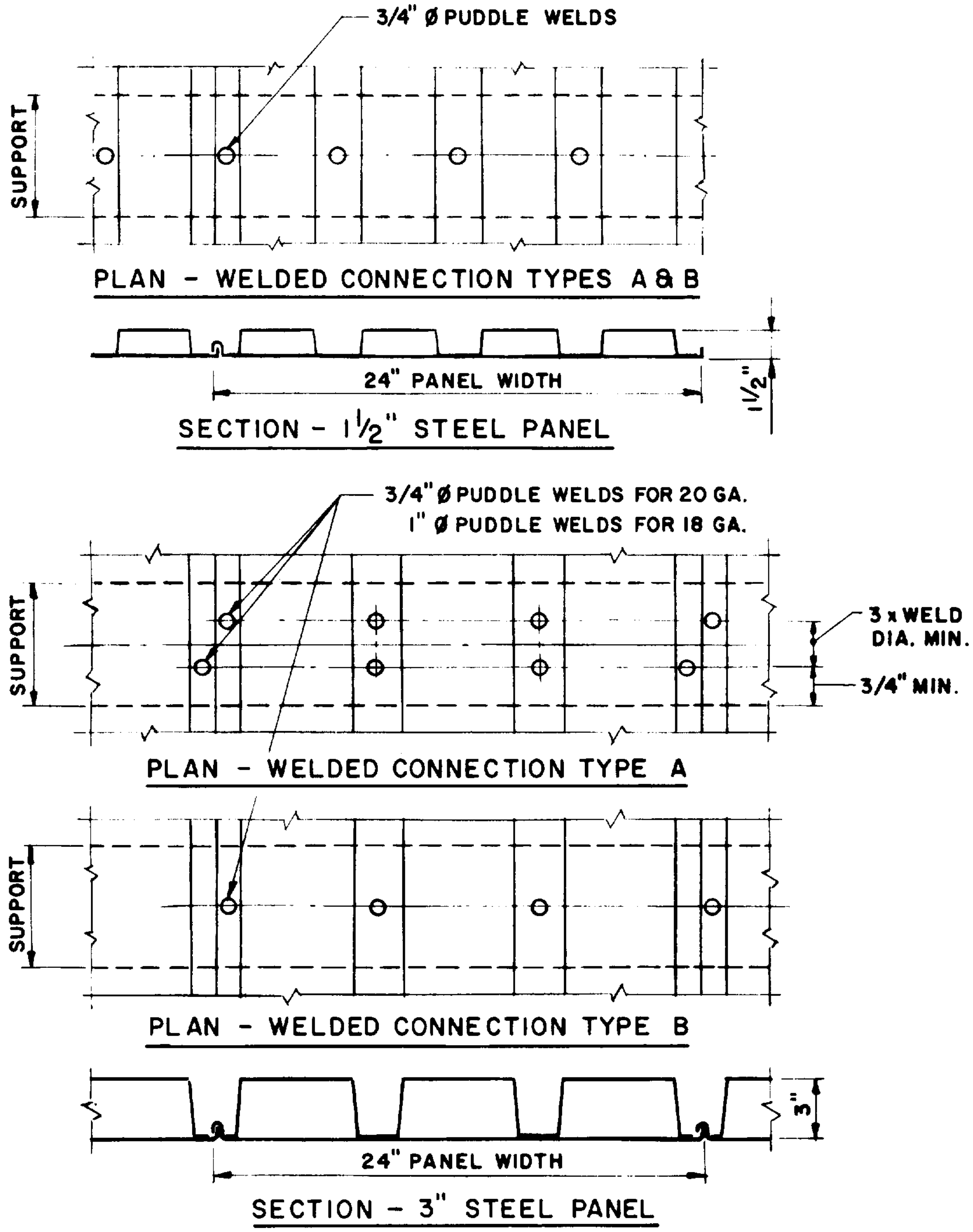


Figure 5-37 Typical welded connections for attaching cold-formed steel panels to supporting members

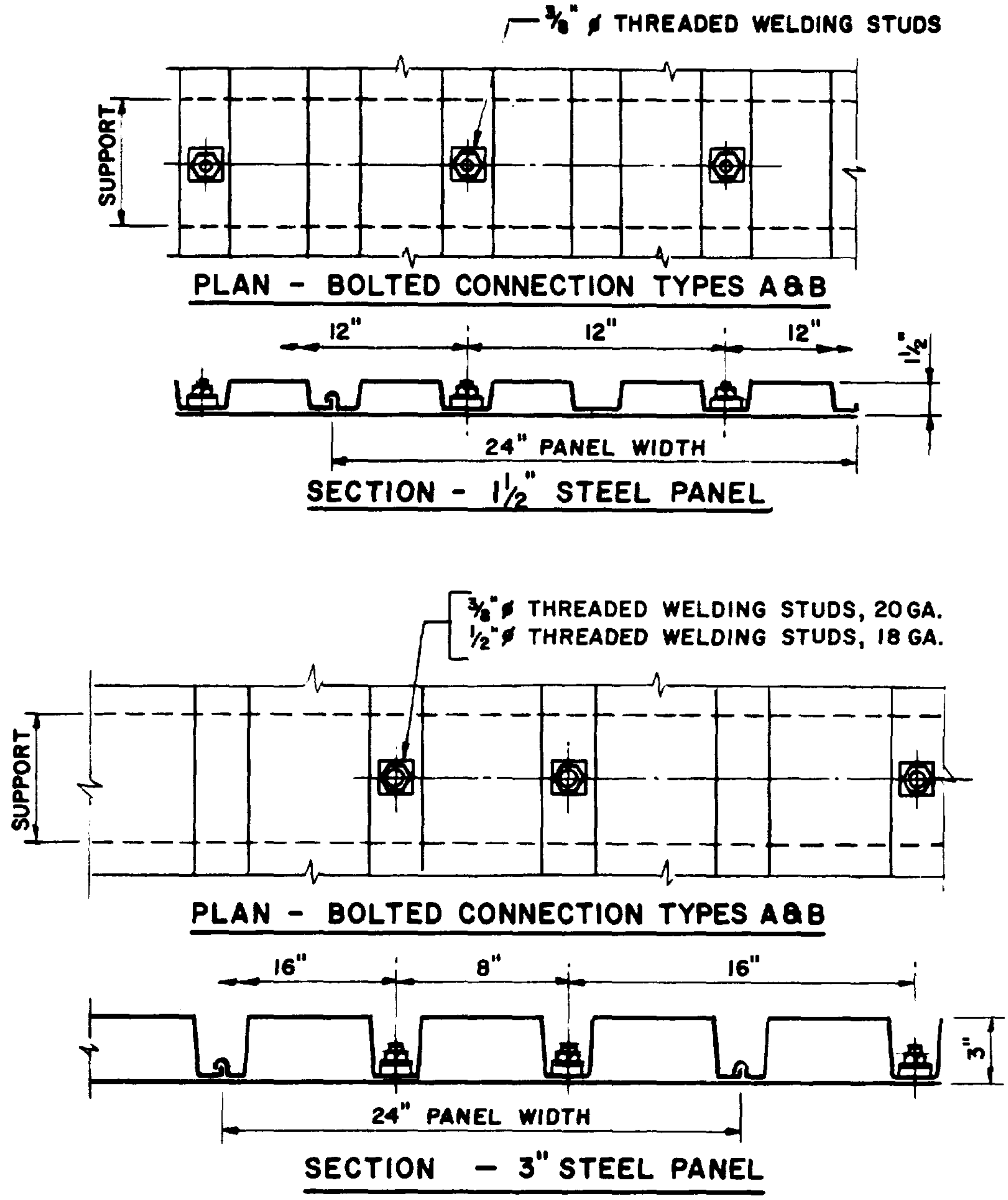


Figure 5-38 Typical bolted connections for attaching cold-formed steel panels to supporting members

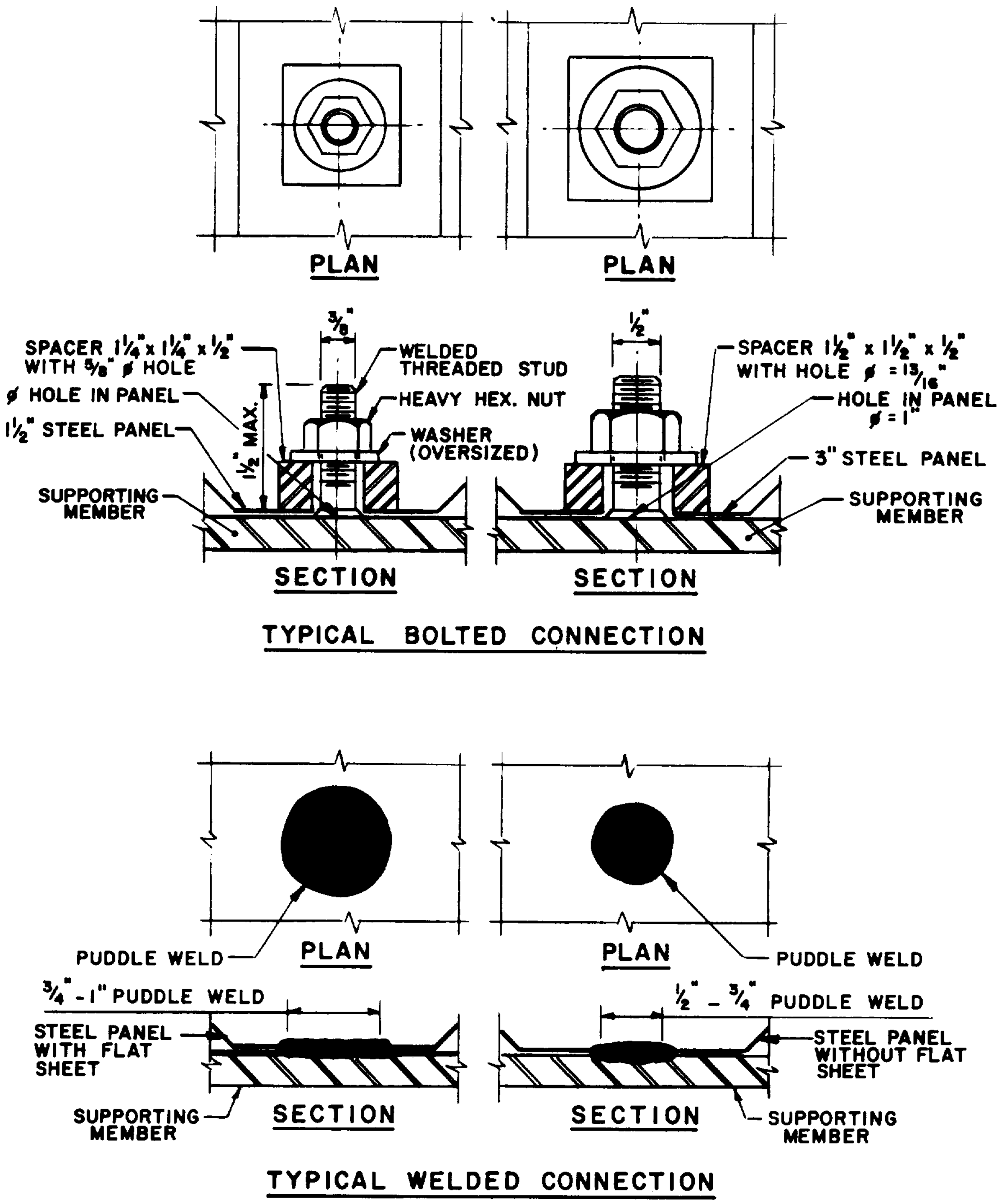


Figure 5-39 Details of typical fasteners for cold-formed steel panels

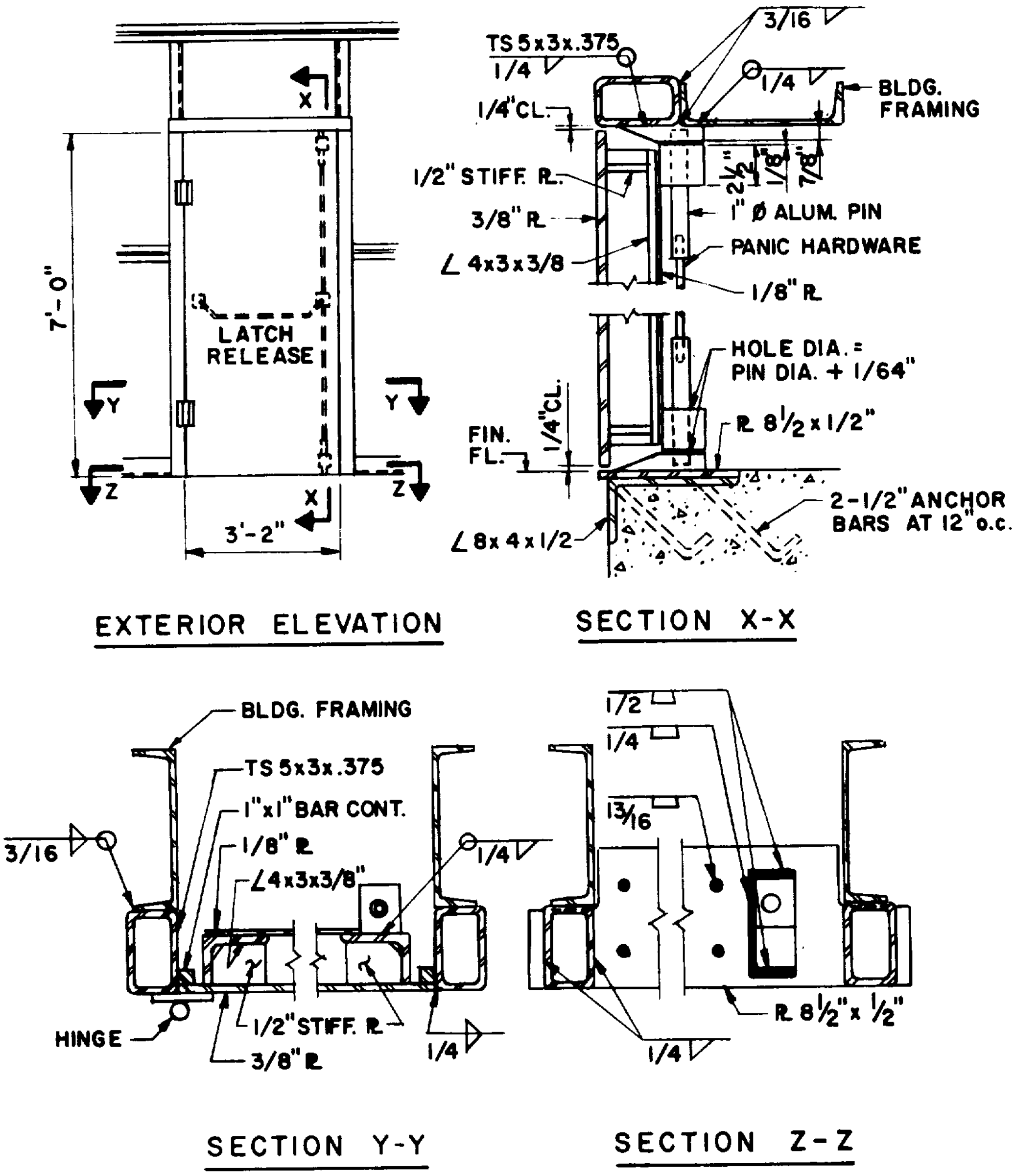


Figure 5-40 Single-leaf blast door installed in a steel structure

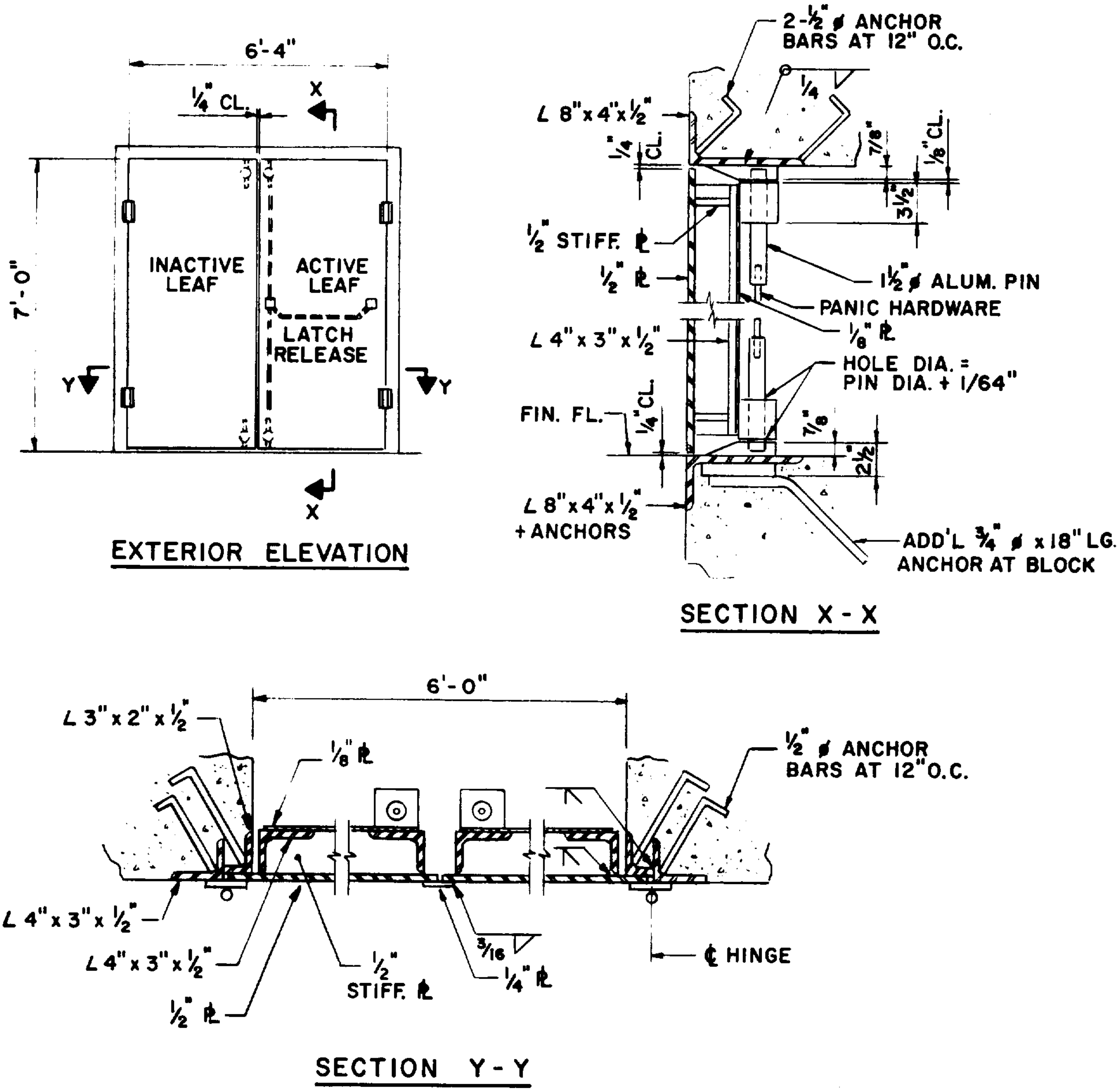
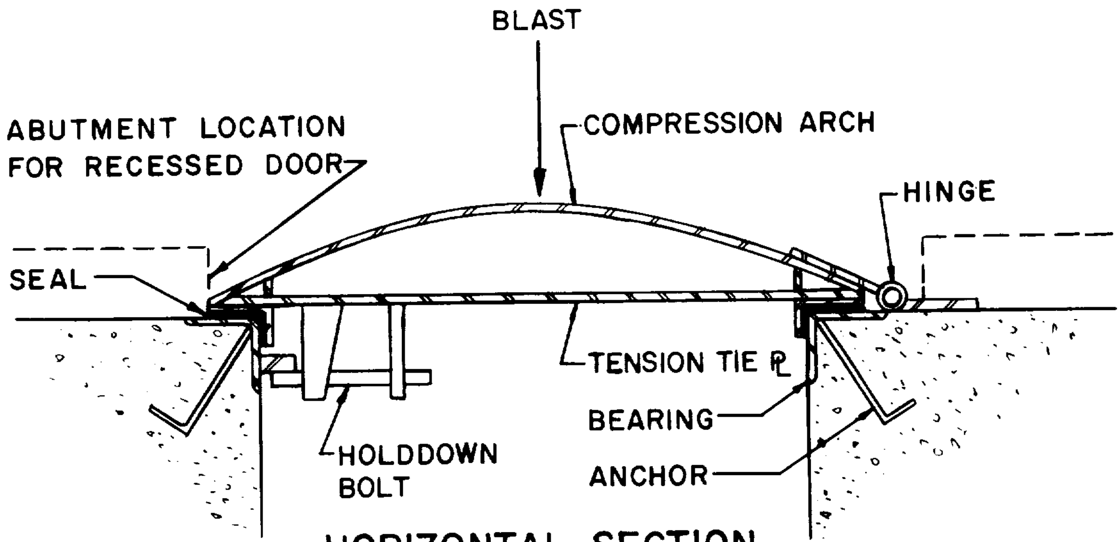
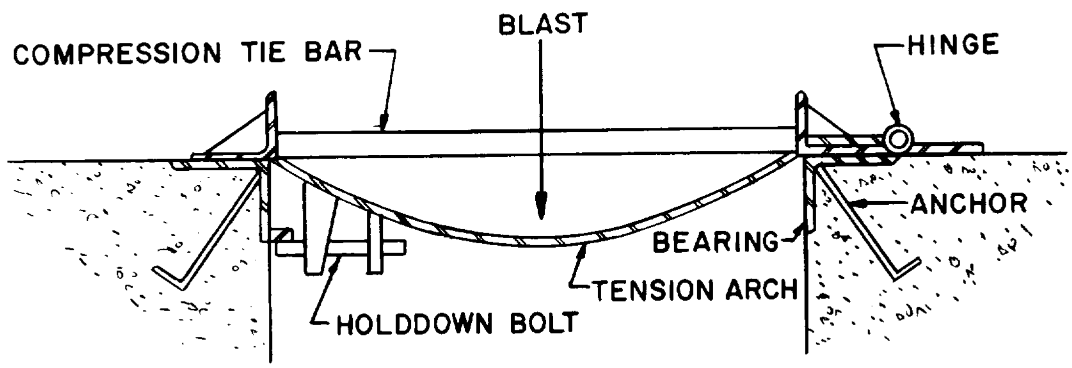


Figure 5-41 Double-leaf blast door installed in a concrete structure



HORIZONTAL SECTION

COMPRESSION ARCH DOOR



HORIZONTAL SECTION

TENSION ARCH DOOR

Figure 5-42 Compression-arch and tension-arch blast doors

APPENDIX 5A

ILLUSTRATIVE EXAMPLES

This appendix presents detailed design procedures and numerical examples on the following topics:

1. Flexural elements subjected to pressure-time loading
2. Lateral bracing requirements
3. Cold-formed steel panels
4. Columns and beam-columns
5. Open-web joists
6. Single-story rigid frames
7. Blast doors
8. Unsymmetrical bending

References are made to the appropriate sections of this chapter and to charts, tables, and equations from Chapter 3 "Principles of Dynamic Analysis".

Problem 5A-1 Design of Beams for Pressure-Time Loading

Problem: Design of a purlin or girt as a flexural member which responds to a pressure-time loading.

Procedure:

Step 1. Establish the design parameters:

a. Pressure-time load

b. Design criteria: Maximum support rotation, θ, depending on protection category.

c. Span length, L, beam spacing, b, and support conditions.

d. Properties and type of steel used, i.e., f_y and E.

Step 2. Determine the equivalent static load, w, using the following preliminary dynamic load factors as discussed in Section 5-22.3.

$$DLF = \begin{cases} 1.0 \text{ for } \theta = 2^\circ \\ 0.5 \text{ for } \theta = 12^\circ \end{cases}$$

Step 3. Using the appropriate resistance formula from Table 3-1 and the equivalent static load derived in Step 2, determine M_p.

Step 4. Select a member size using equation 5-7 or 5-8. Check the local buckling criteria of Section 5-24 for the member chosen.

Step 5. Determine the mass, m, including the weight of the decking over a distance center-to-center of purlins or girts, and the weight of the members.

Step 6. Calculate the equivalent mass M_e using Table 3-12 (Chapter 3).

Step 7. Determine the equivalent elastic stiffness K_E from Table 3.1.

Step 8. Calculate the natural period of vibration, T_N, using equation 5-15.

Step 9. Determine the total resistance, R_u, and peak pressure load, P. Enter appropriate chart in Section 3-19.3 with the ratios T/T_N and P/R_u and the values of C_1, and C_2 in order to establish the ductility ratio μ.

Step 10. Check the assumed DIF used in Step 4. Enter the response charts with the ratio T/T_N and μ and to determine t_E. Using equation 5-1, determine the strain rate. Using figure 5-2, determine the DIF and C. If there is a significant difference from that assumed, repeat Steps 4 through 9.

Step 11. Calculate the equivalent elastic deflection X_E as given by the equation

$$X_E = R_u/K_E$$

and establish the maximum deflection X_m given by

$$X_m = \mu X_E$$

Compute the corresponding member end rotation. Compare Θ with the criteria summarized in Section 5-35.

$$\tan \Theta = X_m/(L/2)$$

Step 12. Check for shear using equation 5-16 and Table 3-9.

Step 13. If a different member size is required, repeat Steps 2 through 12 by selecting a new dynamic load factor.

Example 5A-1 Design of a Beam for Pressure-Time Loading

Required: Design a simply supported beam for shear and flexure in a low pressure range where personnel protection is required.

Step 1. Given:

a. Pressure-time loading (Figure 5A-1)

b. Criteria: Personnel protection required. Support rotation limited to 2°.

c. Structural configuration (Figure 5A-1).

d. $f_y = 36$ ksi, $E = 30 \times 10^3$ ksi, A36 steel

e. Compression flanged braced.

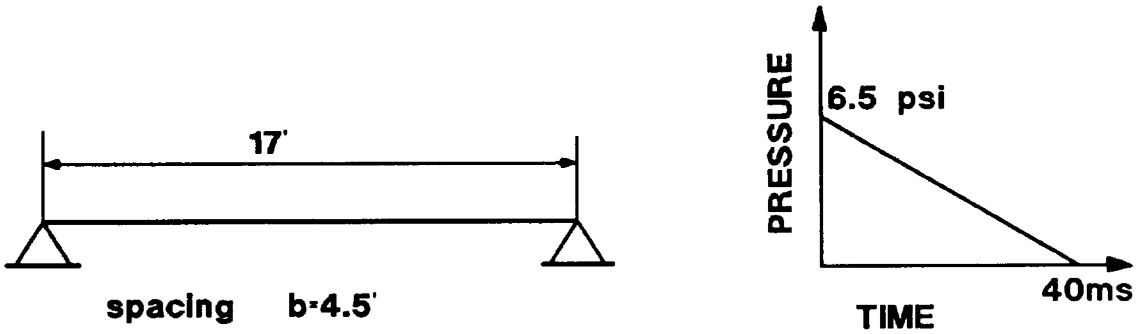


Figure 5A-1 Beam configuration and loading, Example 5A-1

Step 2. Determine the equivalent static load (i.e., required resistance). For this pressure range, the equivalent static load is assumed equal to the peak pressure (Section 5-22.3). The running load becomes:

$$w = 1.0 \times 6.5 \times 4.5 \times 144/1000 = 4.21 \text{ k/ft}$$

Step 3. Determine required M_p.

$$M_p = \frac{wL^2}{8} = \frac{4.21 \times 172}{8} = 152.1 \text{ k-ft} \qquad \text{(Table 3-1)}$$

Step 4. Select a member.

$$(S + Z) = \frac{2M_p}{f_{ds}} = \frac{2 \times 152.1 \times 12}{51.1} = 71.4 \text{ in}^3 \qquad \text{(Equation 5-7)}$$

where

$$f_{ds} = a \times c \times f_y = 1.1 \times 1.29 \times 36 = 51.1 \text{ ksi} \qquad \text{(Equation 5-2)}$$

where

$a = 1.1$ from Section 5-13.2

$c = 1.29$ corresponding to a DIF in the low pressure range (see Table 5-2)

Select W12 x 26, $S = 33.4 \text{ in}^3$ $\quad I = 204 \text{ in}^4$

$Z = 37.2 \text{ in}^3$

$S + Z = 70.6 \text{ in}^3$

$M_p = (70.6 \times 51.1)/(2 \times 12) = 150.3$ k-ft

Check local buckling criteria.

$d/t_w = 53.1 < 412/(f_y)^{1/2} = 68.7$ O.K. (Equation 5-17)

$b_f/2t_f = 8.5$ O.K. (Section 5-24)

Step 5. Calculate M.

$$M = \frac{wL}{g} = \frac{[(4.5 \times 4.8) + 26]\ (17 \times 10^6)}{32.2 \times 1000} = 25{,}130\ (\text{k-ms}^2)/\text{ft}$$

Step 6. Calculate the effective mass, M_e, for a response in the elasto-plastic range.

$K_{LM} = (0.78 + 0.66)/2 = 0.72$ (Table 3-12)

$M_e = 0.72 \times 25{,}130 = 18{,}100 \text{ k-ms}^2/\text{ft}$

Step 7. Determine K_E.

$$K^E = \frac{384\ EI}{5L^3} = \frac{384 \times 30 \times 10^3 \times 204}{5 \times 17^3 \times 144} = 664 \text{ k/ft} \qquad \text{(Table 3-8)}$$

Step 8. Calculate T_N.

$T_N = 2\pi(M_e/K_E)^{1/2} = 2\pi(18{,}100/664)^{1/2}$ (Equation 5-15)

Step 9. Establish the ductility ratio μ and compare with the criteria.

$T/T_N = 40/32.8 = 1.22$

$$P = p \times L \times b = \frac{6.5 \times 17 \times 4.5 \times 144}{1000} = 71.6 \text{ kips}$$

$R_u = 8M_p/L = (8 \times 150.3)/17 = 70.7$ kips

$P/R_u = 71.6/70.7 = 1.01$

From figure 3-64a,

$\mu = X_m / X_E = 2.1$

At this point, the designer would check lateral bracing requirements. Sample problem 5A-2 outlines this procedure.

Step 10. Check the assumed DIF.

From Table 3-64a, for $P/R_u = 1.01$ and $T/T_N = 1.22$.

$t_E/T = 0.24$

$t_E = 0.24 \times 40 = 9.6$ ms

Find $\dot{\epsilon}$:

$\dot{\epsilon} = f_{ds}/E_s t_E = 51.1/30 \times 10^3 \times .0096 = 0.177$ in/in/sec (Equation 5-1)

From figure 5-2

DIF = 1.31 = 1.29 O.K.

Step 11. Determine X_E:

$X_E = R_u/K_E = (70.7 \times 12)/664 = 1.28$ inch

Find X_m:

$X_m = \mu X_E = 2.1 \times 1.28 = 2.69$ inches

Find end rotation, θ.

$\tan\theta = X_m/(L/2) = 2.69/(8.5 \times 12) = 0.0264$ (Table 3-5)

$\theta = 1.52° < 2°$ O.K.

Step 12. Check shear.

Dynamic yield stress in shear

$f_{dv} = 0.55\ f_{ds} = 0.55 \times 51.1 = 28.1$ ksi (Equation 5-4)

Ultimate shear capacity

$V_p = f_{dv} \times A_w = 28.1 \times 0.23 \times 12 = 77.6$ kips (Equation 5-16)

Maximum support shear

$V_s = r_u \times L/2 = R_u/2 = 70.7/2 = 35.4$ kips (Table 3-9)

$V_p > V_s$ O.K.

Problem 5A-2 Spacing of Lateral Bracing

Problem: Investigate the adequacy of the lateral bracing specified for a flexural member.

The design procedure for determining the maximum permissible spacing of lateral bracing is essentially a trial and error procedure if the unbraced length is determined by the consideration of lateral torsional buckling only. However, in practical design, the unbraced length is usually fixed by the spacing of purlins and girts and then must be investigated for lateral torsional buckling.

Procedure:

Step 1. Establish design parameters.

a. Bending moment diagram obtained from a design analysis.

b. Unbraced length, l, and radius of gyration of the member, r_y, about its weak axis.

c. Dynamic design strength, f_{ds}. (Section 5-13)

d. Design ductility ratio, μ, from a design analysis.

Step 2. From the moment diagram, find the end moment ratio, M/M_p, for each segment of the beam between points of bracing. (Note that the end moment ratio is positive when the segment is bent in reverse curvature and negative when bent in single curvature).

Step 3. Compute the maximum permissible unbraced length, l_{cr}, using equation 5-20 or 5-21, as applicable. Since the spacing of purlins and girts is usually uniform, the particular unbraced length that must be investigated in a design will be the one with the largest moment ratio. The spacing of bracing in nonyielded segments of a member should be checked against the requirements of Section 1.5.1.4.5a of the AISC Specification (see Section 5-26.3).

Step 4. The actual length of a segment being investigated should be less than or equal to l_{cr}.

Example 5A-2 Spacing of Lateral Bracing

Required: Investigate the unbraced lengths shown for the W10 x 39 beam in figure 5A-2.

Step 1. Given:

a. Bending moment diagram shown in figure 5A-2.

b. Unbraced length (each segment) = 36 inches
r_y = 1.98 inches

c. Dynamic design stress = 51.1 ksi

d. Design ductility ration, μ = 5.

Step 2. The moment ratio is -0.5 for segments BC and CD (single curvature) and 0.5 for segments AB and DE (double curvature).

Step 3. Determine the maximum permissible unbraced length. By inspection, equation 5-21 results in the lower value of l_{cr}.

$$\frac{\beta l_{cr}}{r_y} = \frac{1375}{f_{ds}}$$

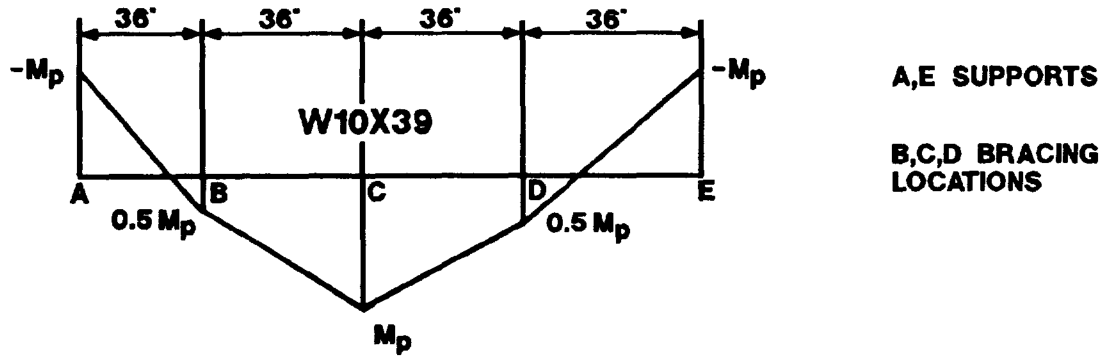


Figure 5A-2 Bending moment diagram, Example 5A-2

From figure 5-9 for $x_m/x_E = 5$, $\beta = 1.36$

$$\therefore l_{cr} = \frac{1375 \times 1.98}{1.36 \times 51.1} = 39.2"$$

Step 4. Since the actual unbraced length is less than 39.2 inches, the spacing of the bracing is adequate.

Problem 5A-3 Design a Roof Deck as a Flexural Member which Responds to Pressure-Time Loading

Problem: Design of cold-formed, light gauge steel panels subjected to pressure-time loading.

Step 1. Establish the design parameters:

a. Pressure-time loading

b. Design criteria: Specify values of μ and θ depending upon whether tension-membrane action is present or not.

c. Span length and support conditions

d. Mechanical properties of steel

Step 2. Determine an equivalent uniformly distributed static load for a 1-ft width of panel, using the following preliminary dynamic load factors.

	Tension-membrane action not present	Tension-membrane action present
DLF	1.33	1.00

These load factors are based on an average value of $T/T_N = 10.0$ the recommended design ductility ratios. They are derived using figure 3-64 of Chapter 3.

Equivalent static load $\qquad w = DLF \times p \times b$
$\qquad b = 1$ ft

Step 3. Using the equivalent load derived in step 2, determine the ultimate moment capacity using equation 5-29 or 5-30 (assume positive and negative are the same).

Step 4. Determine required section moduli using equation 5-27 or 5-28.

Select a panel.

Step 5. Determine actual section properties of the panel:

S^+, S^-, I_{20}, w.

Step 6. Compute r_u, the maximum unit resistance per 1-ft width of panel using equation 5-29 or 5-30.

Step 7. Determine the equivalent elastic stiffness, $K_E = r_uL/X_E$, using equation 5-31.

Step 8. Compute the natural period of vibration.

$$T_N = 2\pi (0.74\ mL/K_E)^{1/2} \qquad \text{(Equation 5-32)}$$

Step 9. Calculate P/r_u and T/T_N. Enter figure 3-64 with the ratios P/r_u and T/T_N to establish the actual ductility ratio μ.

Compare μ with the criteria of step 1. If μ is larger than the criteria value, repeat steps 4 to 9.

Step 10. Compute the equivalent elastic deflection X_E using $X_E = r_uL/K_E$. Evaluate the maximum deflection, $X_m = \mu X_E$.

Determine the maximum panel end rotation.

$$\Theta = \tan^{-1}[X_m/(L/2)]$$

Compare Θ with the criteria of step 1. If Θ is larger than specified in the criteria, select another panel and repeat steps 5 to 10.

Step 11. Check resistance in rebound using figure 5-13.

Step 12. Check panel for maximum resistance in shear by applying the criteria relative to:

a. Simple shear, Table 5-5a, 5-6a or 5-7a.

b. Combined bending and shear, Table 5-5b, 5-6b or 5-7b.

c. Web crippling, figures 5-15 or 5-16.

If the panel is inadequate in shear, select a new member and repeat steps 4 to 12.

Example 5A-3 Design a Roof Deck as a Flexural Member which Responds to Pressure-Time Loading

Required: Design a continuous cold-formed steel panel in a low pressure range.

Step 1. Given:

a. Pressure-time loading (figure 5A-3).

b. Criteria: (Tension-membrane action present)

maximum ductility ratio $\mu_{max} = 6$

maximum rotation $\theta_{max} = 4°$

c. Structural configuration figure 5A-3.

d. Steel A446, grade a

$E = 30 \times 10^6$ psi

$f_{ds} = a \times c \times f_y = 1.21 \times 1.1 \times 33{,}000 = 44{,}000$ psi

(Equation 5-26)

Step 2. Determine the equivalent static load.
Say DLF = 1.0

$w = DLF \times p \times b = 1.0 \times 5.0 \times 12 \times 12 = 720$ lb/ft

Step 3. Determine required ultimate moment capacities. For preliminary selection, assume

$M_{up} = M_{un} = wL^2/10.8 = 720 \times (4.5)^2/10.8 = 1{,}350$ lb-ft

(Equation 5-30)

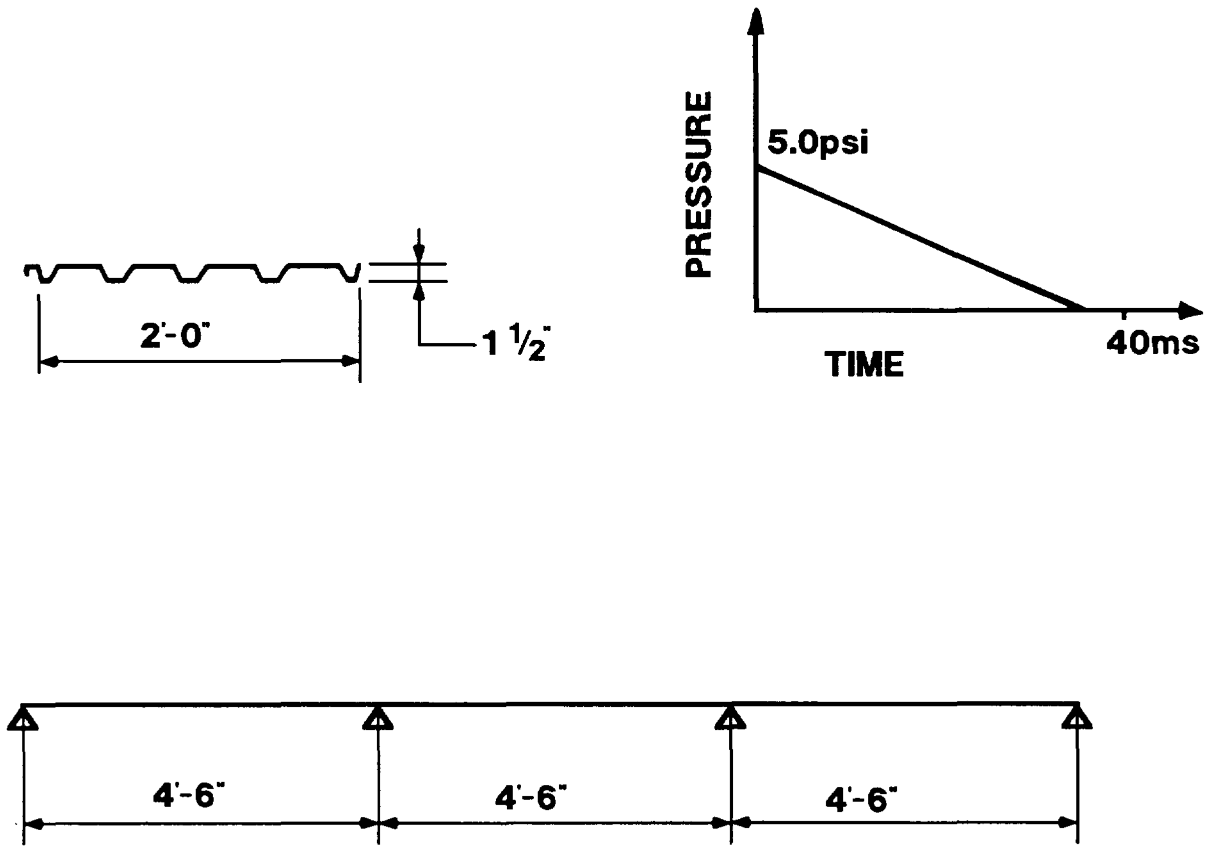


Figure 5A-3 Roof decking configuration and loading, Example 5A-3

Step 4. Determine required section moduli.

$S^+ = S^- = (1350 \times 12)/44{,}000 = 0.368 \text{ in}^3$

(Select Sec. 3-18, 1-1/2 inches deep)

Step 5. Determine actual section properties.

For manufacturer's guide:

$S^+ = 0.398 \text{ in}^3$

$S^- = 0.380 \text{ in}^3$

$I_{20} = 0.337 \text{ in}^4$

$w = 2.9$ psf

Step 6. Compute maximum unit resistance r_u.

$M_{un} = (44{,}000 \times 0.398)/12 = 1{,}459$ lb-ft (Equation 5-27)

$M \quad = (44{,}000 \times 0.380)/12 = 1{,}393$ lb-ft (Equation 5-28)

$$r_u = 3.6\ (M_{un} + 2M_{up})/L^2 = \frac{3.6}{4.5^2}\ (1{,}393 + 2 \times 1{,}459) = 766 \text{ lb/ft}$$

(Equation 5-30)

Step 7. Determine equivalent static stiffness.

$$K_E = \frac{r_u L}{X_E} = \frac{EI_{20} \times r_u \times L}{0.0062 \times r_u \times L^4} = \frac{EI_{20}}{0.0062\ L^3}$$ (Equation 5-31)

$$\frac{30^6 \times 10 \quad \times 0.337}{0.0062\ (4.5^3) \times 144} = 124{,}260 \text{ lb/ft}$$

Step 8. Compute the natural period of vibration for the 1-ft width of panel.

$mL = w/g = (2.9 \times 10^6 \times 4.5)/32.2 = 4.05 \times 10^5 \text{ lb-ms}^2/\text{ft}$

$T_N = 2x[(0.74 \times 4.05 \times 105)/124,260]^{1/2} = 9.75$ msec

Step 9. Calculate P/r_u and T/T_N

$P = p \times b = 5.0 \times 12 \times 12 = 720$ lb/ft

$P/r_u = 720/766 = 0.94$

$T/T_N = 40/9.75 = 4.10$

Entering figure 3-64a with these values.

$X_m/X_E = 3.5 < 6$ O.K.

Step 10. Check maximum deflection and rotation.

$X_E = r_u L/K_E = 766 \times 4.5/124,260 = 0.028$ ft

$X_m = 3.5\ X_E = 0.098$ ft

$\theta = \tan^{-1}[X_m/(L/2)] = \tan^{-1}[0.098/2.25] = 2.5 < 4"$ O.K.

Step 11. Check resistance in rebound.

From figure 5-13, $\bar{r}/r = 0.33$; O.K. since available maximum elastic resistance in rebound is approximately equal to that under direct loading.

Step 12. Check resistance in shear.

a. Interior support (combined shear and bending). Determine dynamic shear capacity of a 1-ft width of panel:

$h = (1.500 - 2t)$ inches, $t = 0.048$ inch

$= 1.500 - 0.096 = 1.404$ inches

$h/t = 1.404/0.048 = 29.25 = 30$

$f_{dv} = 10.84$ ksi (Table 5-5)

Total web area for 1-ft width of panel:

$(8 \times h \times t)/2 = 4 \times 1.404 \times 0.048 = 0.270 \text{ in}^2$

$V_u = 0.270 \times 10.84 = 2.92 \text{ k} = 2{,}922 \text{ lb}$

Determine maximum dynamic shear force:

The maximum shear at an interior support of a continuous panel using limit design is:

$V_{max} = 0.55\, r_u L = 0.55 \times 766 \times 4.5 = 1{,}896 \text{ lb}$

$= 1{,}896 \text{ lb} < 2{,}922 \text{ lb}$ O.K.

b. End support (simple shear)

Determine dynamic shear capacity of a 1-ft width of panel:

For $h/t \leq 57$, $f_{dv} = 0.50\, f_{ds} = 0.5 \times 44.0 = 22.0$ ksi (Table 5-5a)

$V_u = 0.270 \times 22{,}000 = 5{,}940$

Determine maximum dynamic shear force:

The maximum shear at an end support of a continuous panel using limit design is

$V_{max} = 0.45 \times r_u \times L = 0.45 \times 766 \times 4.5$

$= 1{,}551 \text{ lb} < 5{,}940 \text{ lb}$ O.K.

c. Web crippling (4 webs per foot)

End support ($N = 2\text{-}\frac{1}{2}$ inches)

$Q_u = 1{,}200 \times 4 = 4{,}800 \text{ lb} > 1{,}551$ O.K. (figure 5-15)

Interior support (N = 5 inches)

Q_u = (2,400 x 4)/2 = 4,800 lb > 1,896 O.K. (figure 5-16)

Problem 5A-4 Design of Columns and Beam-Columns

Problem: Design a column or beam-column for axial load combined with bending about the strong axis.

Procedure:

Step 1. Establish design parameters.

a. Bending moment M, axial load P, and shear V are obtained from either a preliminary design analysis or a computer analysis.

b. Span length l and unbraced lengths l_x and l_y.

c. Properties of structural steel:

Minimum yield strength f_y

Dynamic increase factor c (Table 5-2)

Dynamic design strength f_{ds} (Equation 5-2)

Step 2. Select a preliminary member size with a section modulus S such that $S \geq M/f_{ds}$ and $b_f/2t_f$ complies with the structural steel being used (Section 5-24).

Step 3. Calculate P_y (Section 5-24) and the ratio P/P_y. Using either equation 5-17 or 5-18, determine the maximum allowable d/t_w ratio and compare it to that of the section chosen. If the allowable d/t_w ratio is less than that of the trial section, choose a new trial section.

Step 4. Check the shear capacity of the web. Determine the web area A_w (Section 5-23) and the allowable dynamic shear stress f_{dv} (equation 5-4). Calculate the web shear capacity V_p (equation 5-16) and compare to the design shear V. If inadequate, choose a new trial section and return to Step 3.

Step 5. Determine the radii of gyration, r_x and r_y, and plastic section modulus, Z, of the trial section from the AISC Handbook.

Step 6. Calculate the following quantities using the various design parameters:

a. Equivalent plastic resisting moment

$$M_p = f_{ds}Z \qquad \text{(Equation 5-8)}$$

b. Effective slenderness ratios Kl_x/r_x and Kl_y/r_y. For the effective length factor K, see Section 1.8 of the Commentary on the AISC Specification and Section 5-38.

c. Allowable axial stress F_a corresponding to the larger value of Kl/r.

d. Allowable moment M_m from equation 5-47 or 5-48.

e. F'_e and "Euler" buckling load P_e (Section 5-37.3).

f. Plastic axial load (Section 5-37.3) and ultimate axial load P_u (equation 5-42).

g. Coefficient C_m (Section 1.6.1 AISC Specification).

Step 7. Using the quantities obtained in Step 6 and the applied moment M and axial load P, check the interaction formulas (equations 5-44 and 5-45). Both formulas must be satisfied for the trial section to be adequate.

Example 5A-4 (a) Design of a Roof Girder as a Beam-Column

Required: Design a fixed-ended roof girder in a framed structure for combined bending and axial load in a low pressure range.

Step 1. Given:

a. Preliminary computer analysis gives the following values for design:

$M_x = 115$ ft-kips

$M_y = 0$

$P = 53.5$ kips

$V = 15.1$ kips

b. Span length $l_x = 17'\text{-}0"$

Unbraced lengths $l_x = 17'\text{-}0"$ and $l_y = 17'\text{-}0"$

c. A36 structural steel

$f_y = 36$ ksi

$c = 1.29$ (Table 5-2)

$a = 1.1$ (Section 5-12.1)

$\therefore f_{ds} = c \times a \times f_y = 1.29 \times 1.1 \times 36 = 51.1$ ksi (Equation 5-2)

Step 2.

$S = M_x/f_{ds} = 115\ (12)/51.1 = 27.0\ \text{in}^3$

Try W 12 x 30 ($S = 38.6\ \text{in}^3$)

$A = 8.79\ \text{in}^2$ $\quad d/t_w = 47.5$

$b_f/2t_f = 7.4 < 8.5$ O.K. (Section 5-24)

Step 3.

$P_y = Af_y = 8.79 \times 36 = 316$ kips (Section 5-24)

$P/P_y = 53.5/316 = 0.169 < 0.27$

$d/t_w = \left[412/(f_y)^{1/2}\right]\left[1 - 1.4\ (P/P_y)\right]$ (Equation 5-17)

$= (412/(36)^{1/2})\ [1 - 1.4\ (0.169)] = 52.4 > 47.5$ O.K.

Step 4.

$V_p = f_{dv}\ A_w$ (Equation 5-16)

$f_{dv} = 0.55\ f_{ds} = 0.55\ (51.1) = 28.1$ ksi (Equation 5-4)

$A_w = t_w(d - 2t_f) = 0.260\ [12.34 - 2\ (0.440)]$

$= 2.98\ \text{in}^2$ (Section 5-23)

$V_p = 28.1(2.98) = 83.7$ kips > 15.1 kips O.K.

Step 5. $r_x = 5.21$ in.

$r_y = 1.52$ in. (AISC Manual)

$Z = 43.1$ in^3

Step 6.

a. $M_{px} = f_{ds} \times Z_x = 51.1 \times 43.1 \times 1/12 = 183.5$ ft-kips (Equation 5-8)

b. $K = 0.75$ (Section 5-39)

$Kl_x/r_x = [0.75(17)12] / 5.21 = 29$

$Kl_y/r_y = [0.75(17)12] / 1.52 = 101$

c. $F_a = 12.85$ ksi for $Kl_y/r_y = 101$ and $f_y = 36$ ksi (Appendix A, AISC Specification)

$1.42(12.85) = 18.25$ ksi for $f_{ds} = 51.1$ ksi

d. $$M_{mx} = \left[1.07 - \frac{(1/r_y)(f_{ds})^{1/2}}{3{,}160}\right] M_{px} \leq M_{px} \qquad \text{(Equation 5-47)}$$

$$= \left[1.07 - \frac{(204/1.52)\ (51.1)^{1/2}}{3{,}160}\right] 183.5 = 140.6 < 183.5 \text{ ft-kips}$$

e. $$F'_{ex} = \frac{12\pi^2 E}{23(Kl_b/r_x)^2} = \frac{12\pi^2(29{,}000)}{23(29)^2} = 177.6 \text{ ksi}$$

(Section 5-37.3)

$$P_{ex} = \frac{23AF'_{ex}}{12} = \frac{23(8.79)177.6}{12} = 2{,}992 \text{ kips}$$

(Section 5-37.3)

f. $P_p = f_{ds}A = 51.1(8.79) = 449$ kips (Section 5-37.3)

$$P_u = 1.7AF_a = 1.7(8.79)18.25 = 273 \text{ kips} \qquad \text{(Equation 5-42)}$$

g. $C = 0.85$ (Section 1.6.1, AISC Specification)

Step 7.

$$\frac{P}{P_u} + \frac{C_{mx} M_x}{(1 - P/P_{ex})M_{mx}} + \frac{C_{my} M_y}{(1 - P/P_{ey})M_{my}} \le 1 \qquad \text{(Equation 5-44)}$$

$$= \frac{53.5}{273} + \frac{0.85(115)}{(1 - 53.5/2992)140.6} = 0.196 + 0.708 = 0.904 < 1 \quad \text{O.K.}$$

$$\frac{P}{P_p} + \frac{M_x}{1.18M_{px}} + \frac{M_y}{1.18M_{py}} \le 1 \qquad \text{(Equation 5-45)}$$

$$= \frac{53.5}{449} + \frac{115}{1.18(183.5)} = 0.119 + 0.531 = 0.650 < 1 \quad \text{O.K.}$$

Trial section meets the requirements of Section 5-37.3.

Example 5A-4 (b) Design of Column

Required: Design of an exterior fixed-pinned column in a framed structure for biaxial bending plus axial loads in a low pressure range.

Step 1. Given:

a. Preliminary design analysis of a particular column gives the following values at a critical section:

$M_x = 311$ ft-kips

$M_y = 34$ ft-kips

$P = 76$ kips

$V = 54$ kips

b. Span length $l = 17'\text{-}3"$

Unbraced lengths $l_x = 17'\text{-}3"$ and $l_y = 4'\text{-}0"$ (laterally supported by wall girts).

c. A36 structural steel

$f_y = 36$ ksi

$c = 1.29$ (Table 5-2)

$a = 1.1$ (Section 5-12.1)

$f_{ds} = a \times c \times f_y = 1.1 \times 1.29 \times 36 = 51.1$ ksi (Equation 5-2)

Step 2.

$S = M_x/f_{ds} = 311(12)/51.1 = 73.0$ in^3

Try W 14 x 68 ($S = 103$ in^3)

$A = 20.0$ in^2 $d/t_w = 33.8$

$b_f/2t_f = 7.0 < 8.5$ O.K. (Section 5-24)

Step 3.

$P_y = Af_y = 20.0(36) = 720$ kips (Section 5-24)

$P/P_y = 76/720 = 0.106 < 0.27$

$d/t_w = [412/(f_y)^{1/2}]\ [1 - 1.4(P/P_y)]$ (Equation 5-17)

$= [412/(36)^{1/2}]\ [1 - 1.4(0.106)] = 58.5 > 32.9$ O.K.

Step 4.

$V_p = f_{dv} A_w$ (Equation 5-16)

$f_{dv} = 0.55 f_{ds} = 0.55(51.1) = 28.1$ ksi (Equation 5-4)

$A_w = t_w(d - 2t_f) = 0.415\ [14.04 - 2(0.720)] = 5.23$ in^2
(Section 5-23)

$V_p = 28.1(5.23) = 147$ kips > 54 kips O.K.

Step 5. $r_x = 6.01$ inches

$r_y = 2.46$ inches

$Z_x = 115\ in^3$ (AISC Manual)

$Z_y = 36.9\ in^3$

Step 6.

a. $M_p = f_{ds}Z$ (Equation 5-8)

$M_{px} = 51.1 \times 115 \times 1/12 = 490$ ft-kips

$M_{py} = 51.1 \times 36.9 \times 1/12 = 157$ ft-kips

b. Use $K = 1.5$ (Section 5-39)

$$\frac{Kl_x}{r_x} = \frac{1.5(17.25)12}{6.01} = 52$$

$$\frac{Kl_y}{r_y} = \frac{1.5(4.00)12}{2.46} = 29$$

c. $F_a = 18.17$ ksi for $Kl_x/r_x = 52$ and $f_y = 36$ ksi

$1.42(18.17) = 25.79$ ksi for $f_{ds} = 51.1$ ksi

$M_{mx} = M_{px} = 490$ ft-kips

d. $M_{my} = M_{py} = 157$ ft-kips (Section 5-37.3)

$$F'_{ex} = \frac{12\pi^2 E}{23(Kl_b/r_x)^2} = \frac{12\pi^2(29,000)}{23(52)^2} = 55.2 \text{ ksi}$$ (Section 5-37.3)

$$F'_{ex} = \frac{12\pi^2 E}{23(Kl_b/r_y)^2} = \frac{12\pi^2(29,000)}{23(29)^2} = 178 \text{ ksi}$$ (Section 537.3)

$$P_{ex} = \frac{23AF'_{ex}}{12} = \frac{23(20.0)55.2}{12} = 2{,}116 \text{ kips}$$

$$P_{ey} = \frac{23AF'_{ey}}{12} = \frac{23(20.0)178}{12} = 6{,}823 \text{ kips}$$ (Section 5-37.3)

$$P_p = f_{ds}A = 51.1(20) = 1{,}022 \text{ kips}$$

$$P_u = 1.7AF_a = 1.7(20)25.79 = 877 \text{ kips}$$

$$C_{mx} = C_{my} = 0.85$$ (Section 1.6.1, AISC Specification)

Step 7.

$$\frac{P}{P_u} + \frac{C_{mx}M_x}{(1 - P/P_{ex})M_{mx}} + \frac{C_{my}M_y}{(1 - P/P_{ey})M_{my}} \leq 1$$ (Equation 5-44)

$$\frac{76}{877} + \frac{0.85(311)}{(1 - 76/2116)490} + \frac{0.85(34)}{(1 - 76/6{,}823)157} =$$

$$0.087 + 0.560 + 0.186 = 0.833 < 1 \quad \text{O.K.}$$

$$P/P_p + M_x/(1.18M_{px}) + M_y/(1.18M_{py}) \leq 1$$ (Equation 5-45)

$$76/1022 + 311/[1.18(490)] + 34/[1.18(157)] =$$

$$0.074 + 0.538 + 0.183 = 0.795 < 1 \quad \text{O.K.}$$

Trial section meets the requirements of Section 5-37.3

Problem 5A-5 Design of Open-Web Steel Joists

Problem: Analysis or design of an open-web joist subjected to a pressure-time loading.

Procedure:

Step 1. Establish design parameters.

a. Pressure-time curve

b. Clear span length and joist spacing

c. Minimum yield stress f_y for chord and web members

Dynamic increase factor, c. (Table 5-2)

d. Design ductility ratio μ and maximum end rotation, θ.

e. Determine whether joist design is controlled by maximum end reaction.

Step 2. Select a preliminary joist size as follows:

a. Assume a dynamic load factor (Section 5-22.3)

b. Compute equivalent static load on joist due to blast overpressure

$w_1 = DLF \times p \times b$
(Dead load of joist and decking not included)

c. Equivalent service live load on joist
$w_2 = w_1/1.7 \times a \times c$ (Section 5-33)

d. From "Standard Load Tables" adopted by the Steel Joist Institute, select a joist for the given span and the structural steel being used, with a safe service load (dead load of joist and decking excluded) equal to or greater than w_2. Check whether ultimate capacity of joist is controlled by flexure or by shear.

Step 3. Find the resistance of the joist by multiplying the safe service load by 1.7 x a x c (Section 5-33).

Step 4. Calculate the stiffness of the joist, K_E, using Table 3-8. Determine the equivalent elastic deflection X_E given by

$X_E = r_u L/K_E$

Step 5. Determine the effective mass using the weight of the joist with its tributary area of decking, and the corresponding load-mass factor given in Table 3-12 of Chapter 3.

Calculate the natural period of vibration, T_N.

Step 6. Follow procedure outlined in step 6a or 6b depending on whether the joist capacity is controlled by flexure or by shear.

Step 6a. Joist design controlled by flexure.

a. Find ductility ratio $\mu = X_m/X_E$ from the response charts in Chapter 3, using the values of T/T_N and P/r_u.

b. Check if the ductility ratio and maximum end rotation meet the criteria requirements outlined in Section 5-35.

If the above requirements are not satisfied, select another dynamic load factor and repeat Steps 2 to 5.

c. Check the selection of the dynamic increase factor used in Step 2c. Using the response charts, find t_E to determine the strain rate, ϵ in equation 5-1. Using figure 5-2, determine DIF. (If elastic response, use T/T_N and appropriate response charts to check DIF).

d. Check if the top chord meets the requirements for a beam-column (Section 5-37.3).

Step 6b. Joist design controlled by shear.

a. Find ductility ratio $\mu = X_m/X_E$ from the response charts in Chapter 3, using the values of T/T_N and P/r_u.

b. If $\mu \leq 1.0$, design is O.K.

If $\mu > 1.0$, assume a higher dynamic load factor and repeat Steps 2 to 5. Continue until $\mu \leq 1.0$. Check end rotation, θ, against design criteria.

c. Check the selection of the dynamic increase factor used in Step 2c, using the value of T/T_N and the appropriate elastic response chart in Section 3-19.3.

d. Since the capacity is controlled by maximum end reaction, it will generally not be necessary to check the top chord as a beam-column. However, when such a check is warranted, the procedure in Step 6a can be followed.

Step 7. Check the bottom chord for rebound.

a. Determine the required resistance, $\bar{r}$, for elastic behavior in rebound.

b. Compute the bending moment, M, and find the axial forces in top and bottom chords using $P = M/d$ where d is taken as the distance between the centroids of the top and bottom chord sections.

c. Determine the ultimate axial load capacity of the bottom chord considering the actual slenderness ratio of its elements.

$$P_u = 1.7AF_a$$

where F_a is defined in Section 5-37.3.

The value of F_a can be obtained by using either equation 5-43 or the tables in the AISC Specification which give allowable stresses for compression members. When using these tables, the yield stress should be taken equal to f_{ds}.

d. Check if $P_u > P$.

Determine bracing requirements.

Example 5A-5 (a) Design of an Open-Web Steel Joist

Required: Design a simply-supported open-web steel joist whose capacity is controlled by flexure.

Solution:
Step 1. Given:

a. Pressure-time loading [figure 5A-5 (a)]

b. Clear span = 50'-0"
Spacing of joists = 7'-0"
Weight of decking = 4 psf

c. Structural steel properties

Chords $f_y = 50{,}000$ psi

Web $f_y = 36{,}000$ psi

Dynamic increase factor (chords only).

$c = 1.19$ (Table 5-2, for A588)

Dynamic design stress, $f_{ds} = c \times a \times f_y$ (Equation 5-2)

Chords $f_{ds} = 1.19 \times 1.1 \times 50{,}000 = 65{,}450$ psi

d. Design criteria (Section 5-35)

Maximum ductility ratio: $\mu_{max} = 4.0$

Maximum end rotation: $\theta_{max} = 2°$

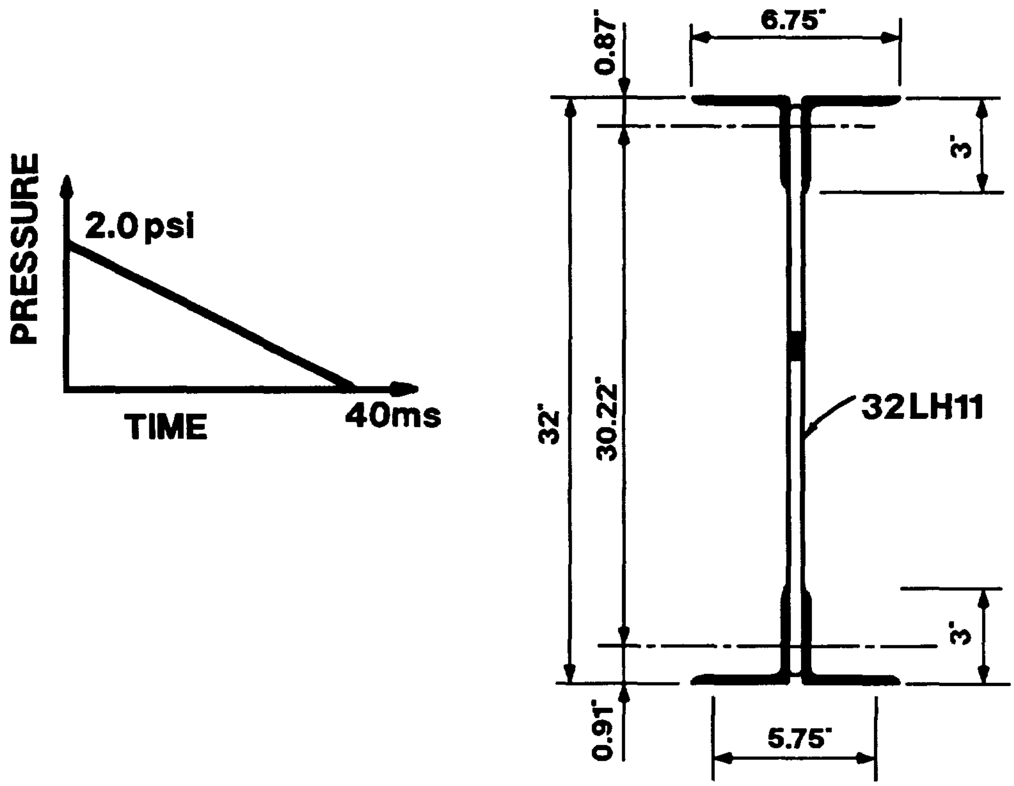


Figure 5A-5(a) Joist cross-section and loading, Example 5A-5(a)

Step 2. Selection of joist size

a. Assume a dynamic load factor. For preliminary design, a DLF = 1.0 is generally recommended. However, since the span is quite long in this case, a DLF of 0.62 is selected.

b. Equivalent static load on joist:

$$w_1 = 0.62 \times 2.0 \times 144 \times 7.0 = 1,250 \text{ lb/ft}$$

c. Service live load on joist:

$$w_2 = w_1/1.7 \times 1.19 \times 1.1 = 1,250/2.23 = 561 \text{ lb/ft}$$

d. Using the "Standard Specifications, Load Tables and Weight Tables" of the Steel Joist Institute, for a span of 50'-0", try 32LH11. Joist tables show that capacity is controlled by flexure.

Total load-carrying capacity (including dead load = 602 lb/ft).

Approximate weight of joist and decking

= 28 + (4 x 7) = 56 lb/ft

Total load-carrying capacity (excluding dead load = 602 - 56 = 546 lb/ft)

The following section properties refer to the selected joist 32LH11 [Figure 5A-5 (a)]:

Top Chord:
Two 3 x 3 x 5/16 angles

$A = 3.56 \text{ in}^2$

$r_x = 0.92$ in.

$r_y = 1.54$ in.

$I_x = 3.02 \text{ in}_4$

Bottom Chord:
Two 3 x 2-1/2 x 1/4 angles

$A = 2.62 \text{ in}^2$

$r_x = 0.945$ in.

$r_y = 1.28$ in.

$I_x = 2.35 \text{ in}^4$

I_{xx} for joist = $1{,}383.0 \text{ in}^4$

Panel length = 51 inches

Step 3. Resistance per unit length

$r_u = 1.7 \times 1.19 \times 1.1 \times 546 = 1215$ lb/ft (Section 5-33)

Step 4.

$$K_E = 384\ EI/5L^3 = \frac{384 \times 29 \times 10^6 \times 1{,}383}{5(12 \times 50)^3} = 14{,}260 \text{ lb/in}$$ (Table 3-8)

$$X_E = r_uL/K_E = \frac{1{,}215 \times 50}{14{,}260} = 4.26 \text{ inches}$$

Step 5. Total mass of joist plus decking

$$M = \frac{56 \times 50 \times 10^6}{386} = 7.25 \times 10^6 \text{ lb-ms}^2/\text{in}$$

Total effective mass $M_e = K_{LM}M$

$K_{LM} = 0.5(0.78 + 0.66) = 0.72$ (Table 3-12)

$M_e = 0.72\ (7.25 \times 10^6) = 5.22 \times 10^6$ lb-ms^2/in

Natural period $T_N = 2\pi(M_e/K_E)^{1/2} = 2\pi(5{,}220{,}000/14{,}260)^{1/2} = 120.2$ ms

Behavior controlled by flexure. Use Step 6a.

Step 6a. a.

$T/T_N = 40/120.2 = 0.332$

$$P/r_u = \frac{2.0 \times 144 \times 7}{1{,}105} = 1.82$$

From figure 3-64a,

$\mu = X_m/X_E = 2.3 < 4$ O.K.

b.

$X_m = 2.3 \times 3.87 = 8.9$ inches

$\tan \Theta = X_m/(L/2) = 8.9/(25 \times 12) = 0.0297$

$\Theta = 1.7° < 2°$ O.K.

c. Check selection of DIF.

From Table 3-64a, for $\mu = 2.3$ and $T/T_N = 0.33$

$t_E/T = 0.55$, $t_E = 0.55 \times 40 = 22$ ms

Find $\dot{\epsilon}$

$\dot{\epsilon} = f_{ds}/E_s t_E = 65.45/30 \times 10^3 \times .022 = 0.099$ in/in/sec

(Equation 5-1)

From figure 5-2 (average of A36 and A514)

DIF = 1.18 = 1.19 assumed O.K.

d. Check top chord as a beam column.

Maximum moment at mid-span

$$M = r_u L^2/8 = \frac{1{,}105 \times (50)^2 \times 12}{8 \times 1{,}000} = 4{,}144 \text{ in-kips}$$

Maximum axial load in chords

$P = M/d$

d = distance between centroids of top and bottom chords [see figure 5A-5 (a)]
= 30.22 inches

$P = 4{,}144/30.22 = 137.1$ kips

l = panel length = 51 inches

Slenderness ratio, $l/r_x = 51/0.92 = 55.4 < C_c$

where $C_c = (2\pi^2 E/f_{ds})^{1/2} = 95$ (Equation 5-41)

$F_a = 23.5$ ksi for $f_y = 50$ ksi (Table 3-50, AISC Specification)

$1.31\ (23.5) = 30.8$ ksi for $f_{ds} = 65{,}450$ psi

$P_u = 1.7AF_a = 1.7 \times 3.56 \times 30.8 = 186.4$ kips (Equation 5-42)

Considering the first panel as a fixed, simply supported beam, the maximum moment in the panel is

$$M = r_u L^2/12 = \frac{1{,}105\ (51)^2}{12 \times 12 \times 1{,}000} = 19.96 \text{ in-kips}$$

The effective slenderness ratio of the top chord in the first panel is

$Kl_b/r_x = (1.0 \times 51)/0.92 = 55.4$

$$F'_{ex} = \frac{12\pi^2 E}{23(Kl_b/r_x)^2} = \frac{12\pi^2 \times 29{,}000}{23\ (55.4)^2} = 48.7 \text{ ksi}$$

$P_{ex} = (23/12)AF'_{ex} = 23/12 \times 3.56 \times 48.7 = 333$ kips

$(1 - P/P_{ex}) = (1.0 - 126.6/333) = 0.62$

To determine M_m, the plastic moment M_p is needed and the value of Z_x has to be computed. The neutral axis for a fully plastic section is located at a distance x from the flange.

$$3\bar{x} = (3 - 5/16)\ 5/16 + 3\ (5/16 - \bar{x})$$

$$= (43)\ 5/(16 \times 16) + 15/16 - 3\bar{x}$$

$\bar{x} = 455/(6 \times 256) = 0.296$ inch

The plastic section modulus, Z_x, is found to be

$$Z_x = 2\left[\frac{(0.296)^2}{2} \times 3 + (3.0 - 0.3125)\ \frac{(0.3125 - 0.296)^2}{2} + \frac{(3 - 0.296)^2}{2} \times 0.3125\right] = 0.263 + 0.0007 + 2.285 = 2.549 \text{ in}^3$$

$M_{px} = f_{ds}Z_x = 65.45 \times 2.549 = 166.8$ in-kips (Equation 5-8)

$$M_{mx} = \left[1.07 - \frac{(1/r_y)\ (f_{ds})^{1/2}}{3160}\right] M_{px} \leq M_{px} \qquad \text{(Equation 5-37)}$$

where r_y is least radius of gyration = 0.92

$= [1.07 - (55.4/391)]\ 166.8 = 154.8$ in-kips

$C_m = 0.85$ (Section 1.6.1, AISC)

$P/P_u + C_mM/[(1 - P/P_{ex})M_{mx}] \leq 1.10$ (Equation 5-44)

$$\frac{137.1}{186.4} + \frac{0.85\ (19.96)}{0.62\ (154.8)} \leq 1.0 = 0.736 + 0.176 = 0.912 < 1.0 \text{ O.K.}$$

Step 7. Check bottom chord for rebound.

a. Calculate required resistance in rebound.

$T/T_N = 0.33$

From figure 5-13, 100% rebound

$\bar{r}/r_u = 1.0$

$\bar{r} = r_u = 1{,}105$ lb/ft

b. Moment and axial forces in rebound

$M = (\bar{r} \times L^2)/8 = 4{,}144$ in-kips

Maximum axial force in bottom chord

$P = M/d = 137.1$ kips (compression)

c. Ultimate axial load capacity

Stability in vertical direction (about x-axis)

$l = 51$ inches $r_x = 0.945$

$l/r = 51/0.945 = 54.0 < C_c$

where $C_c = [(2\pi^2 E)/f_{ds}]^{1/2} = 95$

$F_a = 23.72$ ksi for $f_y = 50$ ksi (Table 3-50, AISC Specification)

$1.31(23.72) = 31.1$ ksi for $f_{ds} = 65{,}450$ psi

$P_u = 1.7AF_a = 1.7 \times 2.62 \times 31.1 = 138.5$ kips

d. Check bracing requirements.

$P = 137.1 < P_u = 138.5$ O.K.

Adding a vertical member between panel joints of bottom chord would have been required had $P > P_u$. This additional bracing would have been needed in mid-span but may be spared at the joist ends.

Stability in the lateral direction (about y-axis)

$P_u = 137.1$ kips

$r_y = 1.28$ inches, $A = 2.62\ in^2$

$F_a = P_u/(1.7 \times 1.31A) = 137.1/(1.7 \times 1.31 \times 2.62) = 23.5$ ksi

For a given $F_a = 23.5$, the corresponding slenderness ratio is

$l/r \approx 55$ (Table 3-50, AISC Specification)

Therefore, the maximum unbraced length in mid-span is

$l_b = 55 \times 1.28 = 70.4$ inches

Use lateral bracing at panel points, i.e., 51 inches at midspan. The unbraced length may be increased at joist ends, but not greater than specified for bridging requirements in the joist specification.

Example 5A-5 (b) Analysis of Existing Open-Web Steel Joist

Required: Analyze a simply supported, open-web steel joist whose capacity is controlled by shear.

Solution:

Step 1. Given:

a. Pressure-time loading [figure 5A-5 (b)]

Joist 22H11

b. Clear span = 32'-0"

Spacing of joists = 6'-0"

Weight of decking = 4 psf

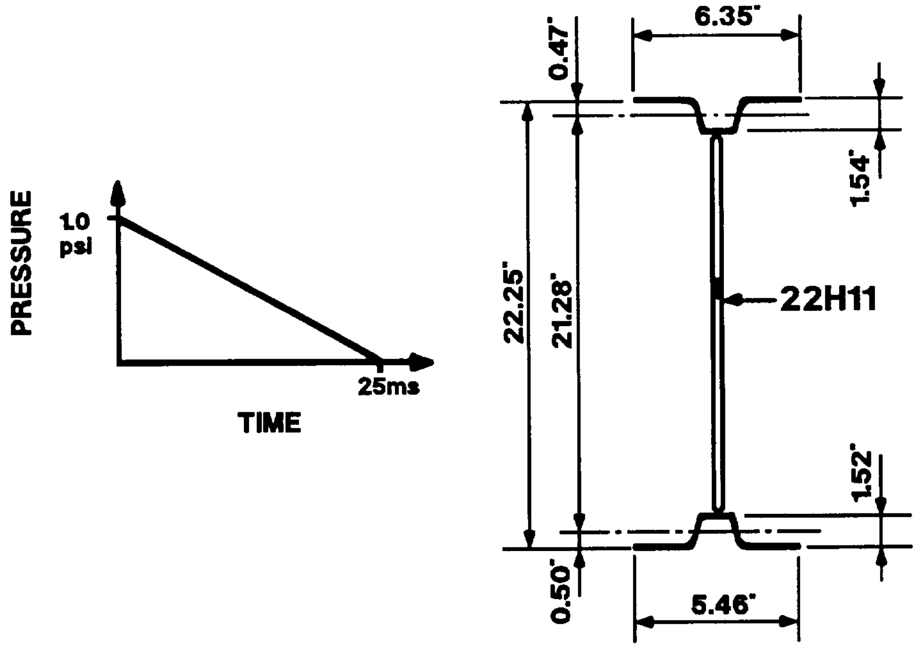


Figure 5A-5(b) Joist cross-section and loading, Example 5A-5(b)

c. Properties of structural steel:

Chords $f_y = 50,000$ psi

Web $f_y = 36,000$ psi

Dynamic increase factor (chords only) $c = 1.19$ (Table 5-2 for A588)

Dynamic design stress, $f_{ds} = c \times a \times f_y$

Chords $f_{ds} = 1.19 \times 1.1 \times 50,000 = 65,450$ psi

d. Design criteria (Section 5-35)

For members controlled by shear:

$\mu_{max} = 1.0$

$\theta_{max} = 1°$

Step 2. a. Assume the DLF = 1.25

b. Overpressure load on joist

$w_1 = 1.25 \times 1.0 \times 144 \times 6 = 1{,}080$ lb/ft

c. Equivalent service load

$w_2 = w_1/1.7 \times a \times c = 1{,}080/2.23 = 485$ lb/ft

d. From the "Standard Specifications and Load Tables" of the Steel Joist Institute:

Total load-carrying capacity (including dead load) = 506 lb/ft.

Approximate weight of joist plus decking =

17 + (6 x 4) = 41 lb/ft

Total load-carrying capacity (excluding dead load) =

506 - 41 = 465

From the steel joist catalog, the following are the section properties of Joist 22H11 [figure 5A-5 (b)]:

Panel length = 24 inches

Top Chord:

$A = 1.935$ in^2

$I_x = 0.455$ in^4

$r_x = 0.485$ in.

$r_y = 1.701$ in.

Bottom Chord:

$A = 1.575$ in^2

$I_x = 0.388$ in^4

$r_x = 0.497$ in.

$r_y = 1.469$ in.

I_{xx} for joist = 396.0 in^4

Step 3. Resistance per unit length

$$r_u = 2.23 \times 465 = 1{,}035 \text{ lb/ft}$$

Step 4.

$$K_E = \frac{384EI}{5L^3} = \frac{384 \times 29 \times 10^6 \times 396}{5 \times (12 \times 32)^3} = 15{,}580 \text{ lb/in} \qquad \text{(Table 3-8)}$$

$$X_E = r_uL/K_E = \frac{1{,}035 \times 32}{15{,}580} = 2.13 \text{ inches}$$

Step 5. Mass of joist plus decking

$$M = \frac{41 \times 32 \times 10^6}{386} = 3.4 \times 10^6 \text{ lb-ms}^2\text{/in}$$

Effective mass $M_e = K_{LM}M$

$$= 0.78 \times 3.4 \times 10^6 = 2.65 \times 10^6 \text{ lb-ms}^2\text{in}$$

Natural period $T_N = 2\pi(M_e/K_E)^{1/2}$

$$= 2\pi(2{,}650{,}000/15{,}580)^{1/2} = 81.8 \text{ ms}$$

Behavior controlled by shear. Use Step 6b of the procedure.

Step 6b.

a. $T/T_N = 25/81.8 = 0.305$

$$P/r_u = \frac{6 \times 144 \times 1.0}{1{,}035} = 0.835$$

b. From Figure 3-64a of Chapter 3:

$\mu = X_m/X_E < 1.0$; elastic, O.K.

$\tan \Theta = X_m/(L/2)$

$$= 2.13/(16.0 \times 12) = 0.0111$$

$\Theta = 0.64° < 1°$ O.K.

c. Check selection of DIF from figure 3-49 of Chapter 3, for $T/T_N = 0.305$, $t_m/T = 1.12$; $t_m = 1.12 \times 25 = 28$ ms

Find $\dot{\epsilon}$

$\dot{\epsilon} = f_{ds}/E_s t_e \quad (t_E - t_m)$ (Equation 5-1)

$= 65.45/30 \times 10^3 \quad \times 0.028 = 0.078$ in/in/sec

From figure 5-2, (average of A36 and A514)

DIF = 1.18 = 1.19 assumed, O.K.

d. Check of top chord as a beam-column is not necessary.

Step 7. Check bottom chord in rebound.

a. For $\mu = 1$ and $T/T = 0.305$, rebound is 100% (Figure 5-13)

$\bar{r} = r_u$

b. Determine axial load in bottom chord, $P = M/d$.

For an elastic response, $\mu < 1.0$, where $T/T_N = 0.305$, the DLF = 0.87 (Figure 3-49)

Equivalent static load, w

$w = DLF \times b \times p = 0.87 \times 12 \times 12 \times 6 \times 1.0 = 751$ lb/ft

Maximum moment in rebound, $M = wL^2/8$

$M = [751 \times (32)^2/8]\ 12 = 1{,}155{,}000$ in-lb

$P = M/d = 1{,}155{,}000/21.28 = 54{,}300$ lb $= 54.3$ kips

c. Check bracing requirements.

(1) Vertical bracing of bottom chord:

Panel length = 24 inches

$r_x = 0.497, \quad r_y = 1.469, \quad A = 1.575$ in^2

$l/r_x = 24/0.497 = 48.3$

Allowable $P = 1.7 \times a \times c \times A \times F_a$

$= 1.7 \times 1.1 \times 1.19 \times 1.575 \times 24.6 = 86.2$ kips > 54.3 kips
(Table 3-50, AISC Specification)

No extra bracing required.

(2) Lateral bracing of bottom chord:

$P = 54.3$ kips, $A = 1.575$ in^2

$F_a = P/1.7A = 54.3/(1.7 \times 1.575 \times 1.1 \times 1.19) = 15.5$ ksi

For $f_y = 50$ ksi and $F_a = 15.5$ ksi

$l/r = 96$

$l = 96 \times 1.469 = 141$ inches
(Table 3-50, AISC Specification)

Therefore, use lateral bracing at every fifth panel point close to mid-span. The unbraced length may be increased at joist ends, but not greater than specified for bridging requirements in the joist specification.

Problem 5A-6 Design of Single-Story Rigid Frames for Pressure-Time Loading

Problem: Design a single-story, multi-bay rigid frame subjected to a pressure-time loading.

Procedure:

Step 1. Establish the ratio α between the design values of the horizontal and vertical blast loads.

Step 2. Using the recommended dynamic load factors presented in Section 5-41.3 establish the magnitude of the equivalent static load w for:

a. local mechanisms of the roof and blastward column

b. panel or combined mechanisms for the frame as a whole.

Step 3. Using the general expressions for the possible collapse mechanisms from Table 5-13 and the loads from Step 2, assume values of the moment capacity ratios C and C_1 and proceed to establish the required design plastic moment M_p considering all possible mechanisms. In order to obtain a reasonably economical design, it is desirable to select C and C_1 so that the least resistance (or the required value of M_p) corresponds to a combined mechanism. This will normally require several trials with assumed values of C and C_1.

Step 4. Calculate the axial loads and shears in all members using the approximate method of Section 5-41.4.

Step 5. Design each member as a beam-column using the ultimate strength design criteria of Sections 5-37.3, 38, and 39. A numerical example is presented in Section 5A-4.

Step 6. Using the moments of inertia from Step 5, calculate the sidesway natural period using Table 5-14 and Equations 5-50 and 5-51. Enter the response charts in Chapter 3 with the ratios of T/T_N and P/R_u. In this case, P/R_u is the reciprocal of the panel or sidesway mechanism dynamic load factor used in the trial design. Multiply the ductility ratio by the elastic deflection given by equation 5-53 and establish the peak deflection X_m from equation 5-54. Compare the maximum sidesway deflection X_m with the criteria of Section 5-35. Note that the sidesway deflection δ in Table 5-8 is X_m.

Step 7. Repeat the procedure of Step 6 for the local mechanisms of the roof and blastward column. The stiffness and natural period may be obtained from Table 3-8 of Chapter 3 and equation 5-15, respectively. The resistance of the roof girder and the blastward column may be obtained from Table 5-13 using the values of M_p and CM_p determined in Step 3. Compare the ductility ratio and rotation with the criteria of Section 5-35.

Step 8. a. If the deflection criteria for both sidesway and beam mechanisms are satisfied, then the member sizes from Step 5 constitute the results of this preliminary design. These members would then be used in a more rigorous dynamic frame analysis. Several computer programs are available through the repositories listed in Section 5-4.

b. If the deflection criterion for a sidesway mechanism is exceeded, then the resistance of all or most of the members should be increased.

c. If the deflection criterion for a beam mechanism of the front wall or roof girder is exceeded, then the resistance of the member in question should be increased. The member sizes to be used in a final analysis should be the greater of those determined from Steps 8b and 8c.

Example 5A-6 Design of a Rigid Frame for Pressure-Time Loading

Required: Design a four-bay, single-story, reusable, pinned-base rigid frame subjected to a pressure-time loading in its plane.

Given:

a. Pressure-time loading (Figure 5A-6)

b. Design criteria: It is required to design the frame structure for more than one incident. The deformation limits shall be half that permitted for personnel protection, that is:

$$\delta = H/50 \text{ and}$$

$$\theta_{max} = 1° \text{ for individual members}$$

c. Structural configuration (figure 5A-6)

d. A36 steel

e. Roof purlins spanning perpendicular to frame ($b_v = b_n$, Figure 5-26)

f. Frame spacing, b = 17 ft

g. Uniform dead load of deck, excluding frame

Step 1. Determine α: (Section 5-41.1)

$$b_h = b_v = 17 \text{ ft}$$

$$q_h = 5.8 \times 17 \times 12 = 1{,}183 \text{ lb/in}$$

$$q_v = 2.5 \times 17 \times 12 = 510 \text{ lb/in}$$

$$\alpha = q_h/q_v = 2.32$$

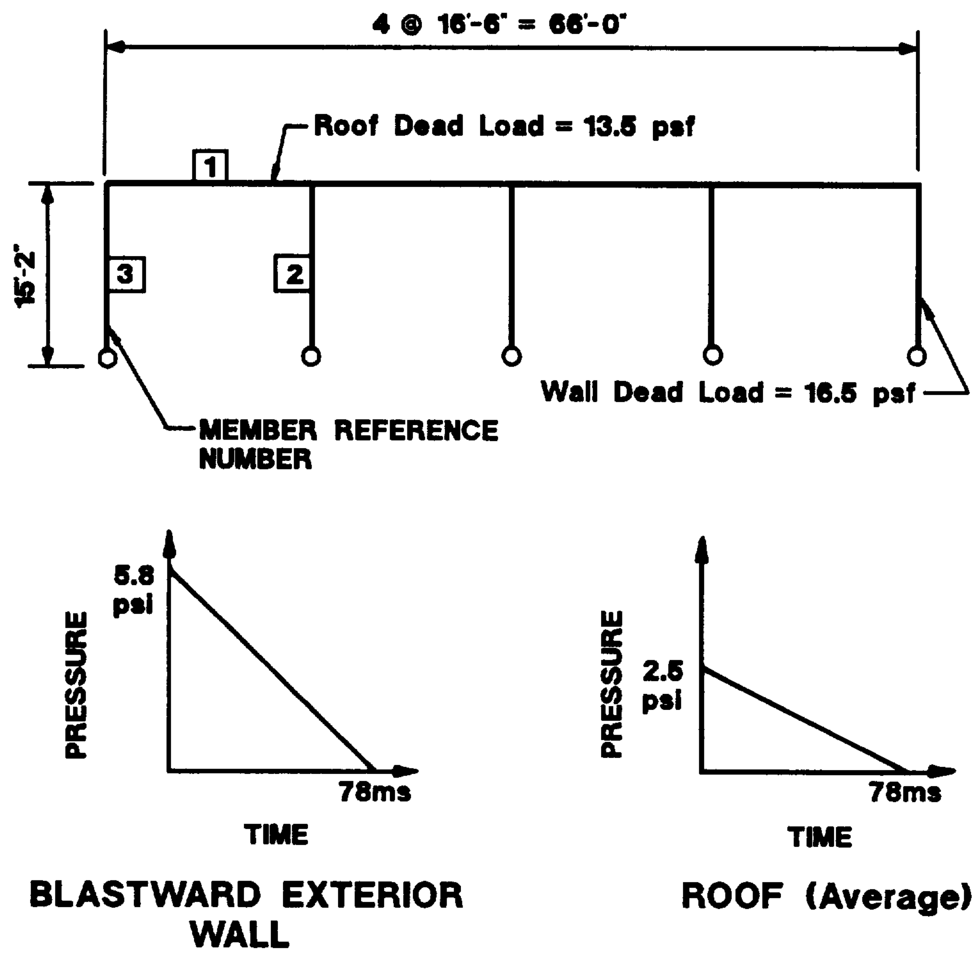


Figure 5A-6. Preliminary design of four-bay, single-story rigid frame, Example 5A-6

Step 2. Establish equivalent static loads (Section 5-41.3)

a. Local beam mechanism, $w = DLF \times q_v$

$$w = \frac{1.25 \times 510 \times 12}{1,000} = 7.65 \text{ k/ft}$$

b. Panel or combined mechanism, $w = DLF \times q_h$

$$w = \frac{0.625 \times 510 \times 12}{1,000} = 3.83 \text{ k/ft}$$

Step 3. **The required plastic moment capacities for the frame members are determined from Table 5-13 based upon rational assumptions for the moment capacity ratios C_1 and C. In general, the recommended starting values are C_1 equal to 2 and C greater than 2. From Table 5-13. for $n = 4$, $\alpha = 2.32$, $H = 15.167$ ft, $L = 16.5$ ft and pinned bases, values of C_1 and C were substituted and after a few trials, the following solution is obtained:**

$M_p = 130$ kip-ft, $C_1 = 2.0$ and $C = 3.5$.

The various collapse mechanisms and the associated values of M_p are listed below:

Collapse Mechanism	w (k/ft)	M_p (k-ft)
1	7.65	130
2	7.65	128
3a, 3b	3.83	128
4	3.83	129
5a, 5b	3.83	110
6	3.83	116

The plastic design moments for the frame members are established as follows:

Girder, $M_p = 130$ k-ft

Interior column, $C_1M_p = 260$ k-ft

Exterior column, $CM_p = 455$ k-ft

Step 4. a. Axial loads and shears due to horizontal blast pressure.

$w = 3.83$ k/ft

From figure 5-27, $R = \alpha wH = 2.32 \times 3.83 \times 15.167 = 135$ kips

1. Member 1, axial load

   $P_1 = R/2 = 67$ kips

2. Member 2, shear force

   $V_2 = R/2(4) = 135/8 = 16.8$ kips

3. Member 3, shear force

   $V_3 = R/2 = 67$ kips

b. Axial loads and shears due to vertical blast pressure,

$w = 7.65$ k/ft

1. Member 1, shear force

   $V_1 = w \times L/2 = 7.65 \times 16.5/2 = 63.1$ kips

2. Member 2, axial load

   $P_2 = w \times L = 7.65 \times 16.5 = 126.2$ kips

3. Member 3, axial load

   $P_3 = w \times L/2 = 63.1$ kips

Note:
The dead loads are small compared to the blast loads and are neglected in this step.

Step 5. The members are designed using the criteria of Sections 5-37.3, 5-38, and 5-39 with the following results:

Member	M_p (k-ft)	P (k)	V (k)	Use	I_x (in^4)
1	130	67.0	63.1	w12x35	285
2	260	126.2	16.8	w14x61	640
3	455	63.1	67.0	w14x74	796

Step 6. Determine the frame stiffness and sway deflection.

$$I_{ca} = \frac{(3 \times 640) + (2 \times 796)}{5} = 702 \text{ in}^4 \qquad \text{(Table 5-14)}$$

$$I_g = 285 \text{ in}^4$$

$$\beta = 0$$

$$D = \frac{I\ /L_g}{0.75 I_{ca}/H} = \frac{285/16.8}{(0.75)\ (702/15.167)} = 0.498 \qquad \text{(Table 5-14)}$$

$$C_2 = 4.65$$

$$K = \frac{EI_{ca}\ C_2}{H^3}\ [1 + (0.7 - 0.1\beta)\ (n-1)] \qquad \text{(Table 5-14)}$$

$$= \frac{(30)\ (10)^3\ (702)\ (4.65)\ [1 + 0.7(3)]}{(15.167 \times 12)^3} = 50.2 \text{ k/in}$$

$$K_L = 0.55\ (1 - 0.25\beta) = 0.55 \qquad \text{(Equation 5-51)}$$

Calculate dead weight, W:

$$W = b[(4Lw_{dr}) + (1/3)\ (Hw_{dw})] + (35 \times 66)$$
$$+ 1/3\ (15.167)\ [(3 \times 61) + (2 \times 74)] = 20{,}548 \text{ lb}$$

$$m_e = W/g = 20{,}548/32.2 = 638 \text{ lb-sec}^2\text{/ft} = 638 \times 10^6 \text{ lb-ms}^2\text{/ft}$$

$$T_N = 2\pi[m_e/KK_L]^{1/2} \qquad \text{(Equation 5-50)}$$

$$= 2\pi[(638 \times 10^6)/(50.2 \times 12 \times 10^3 \times 0.55)]^{1/2} = 276 \text{ ms}$$

$$T/T_N = 78/276 = 0.283$$

$P/R_u = 1.6$

$\mu = X_m/X_E = 1.40$ (Figure 3-64a)

$$X_E = R_u/K_E = \alpha wH/K_E = \frac{2.32 \times 3.83 \times 15.167}{50.2} = 2.68 \text{ inches}$$

(Equation 5-52 and 5-53)

$X_m = \delta = 1.40 \times 2.68 = 3.75$ inches (Equation 5-54)

$\delta = 3.75/(15.167)\ (12)H = 0.0206H = H/48.5$

Step 7. Check deflection of possible local mechanisms.

a. Roof girder mechanism (investigate W12 x 35 from Step 5)

$T_N = 2\pi\ (m_e/K_E)^{1/2}$ (Equation 5-15)

$m_e = K_{LM} \times m$

For an elasto-plastic response, take the average load-mass factor for the plastic and elastic response, or:

$K_{LM} = (0.77 = 0.66)/2 = 0.715$ (Table 3-12)

$\therefore m = 0.715 \times W/g$

$w = (13.5 \times 17) + 36 = 265$ lb/ft

$W = w \times L = 265 \times 16.5 = 4372$ lb.

$m_E = 0.715 \times 4372/368 = 8.1\ \text{lb-sec}^2/\text{in.}$

$K_E = 307\ EI/L^3$ (Table 3-8)

$$\therefore K = \frac{307\ (30)\ (10^6)\ (285)}{(16.5 \times 12)^3} = 332{,}000 \text{ lb/in.}$$

$\therefore T_N = 2\pi\ (8.1/332{,}000)^{1/2} \times 1{,}000$

$= 31.0$ ms

$T/T_N = 78/31.0 = 2.52$

$R_u = 16M_p/L = (16 \times 130)/16.5 = 126$ kips (Table 5-13)

$P = pbL = (2.5)\ (17)\ (144)\ (16.5)/1{,}000 = 101$ kips

$P/R_u = 101/126 = 0.80$

$\mu = X_m/X_E = 1.80$ (Figure 3-64a)

Check end rotation of girder.

$X_E = R_u/K_E = 126/332 = 0.380$ inch

$X_m = 1.80 \times 0.380 = 0.69$ inch

$X_m/(L/2) = 0.69/(8.25)\ (12) = 0.069 = \tan \theta,$

$\theta = 0.40° < 1°$ O.K.

b. Exterior column mechanism (investigate W14 x 74 from Step 5).

$T_N = 2\pi\ (m_e/K_E)^{1/2}$ (Equation 5-15)

$$m_e = K_{LM} \times m = \left[\frac{0.78 + 0.66}{2} \right] \frac{w}{g}$$ (Table 3-12)

$= 0.72\ w/g$

$w = (16.5 \times 17) + 74 = 354$ lb/ft

$W = 354 \times 15.167 = 5369$ lb

$M_e = 0.72\ (5369)/386 = 10.0$ lb-sec^2/in.

$K_E = 160\ EI/L^4$ (Table 3-8)

$$K_E = \frac{(160)\ (30)\ (10)\ (796)}{(182)^3} = 632{,}000 \text{ lb/in}$$

$$T_N = 2\pi(10.0/632{,}000)^{1/2} \times 1000 = 25.0 \text{ ms}$$

$$T/T_N = 78/25.0 = 3.12$$

$$R_u = \frac{4M_p(2C + 1)}{H} = \frac{4\ (130)\ [(2 \times 3.5) + 1]}{15.167} = 275 \text{ kips}$$

(Table 5-13)

$$P = (2.32)\ (7.65/1.25)\ (15.167) = 215 \text{ kips}$$

$$P/R_u = 215/275 = 0.78$$

$$\mu = X_m/X_E = 1.80$$ (Figure 3-64a)

Check end rotation of columns.

$$X_E = R_u/K = 275/632 = 0.435 \text{ inch}$$

$$X_m = 1.80 \times 0.435 = 0.78 \text{ inch}$$

$$X_m/(L/2) = 0.78/(7.58)\ (12) = 0.0086 = \tan \theta$$

$$\theta = 0.49^\circ < 1^\circ \qquad \text{O.K.}$$

Step 8. The deflections of the local mechanisms are within the criteria. The sidesway deflection is acceptable.

Summary: The member sizes to be used in a computer analysis are as follows:

Member	Size
1	W12 x 35
2	W14 x 61
3	W14 x 74

Problem 5A-7 Design of Doors for Pressure-Time Loading

Problem: Design a steel-plate blast door subjected to a pressure-time loading.

Procedure:

Step 1. Establish the design parameters

a. Pressure-time load

b. Design criteria: Establish support rotation, θ_{max}, and whether seals and rebound mechanisms are required.

c. Structural configuration of the door including geometry and support conditions

d. Properties of steel used:

Minimum yield strength, f_y, for door components (Table 5-1)

Dynamic increase factor, c (Table 5-2)

Step 2. Select the thickness of the plate.

Step 3. Calculate the elastic section modulus, S, and the plastic section modulus, Z, of the plate.

Step 4. Calculate the design plastic moment, M_p, of the plate (equation 5-7).

Step 5. Compute the ultimate dynamic shear, V_p (equation 5-16).

Step 6. Calculate maximum support shear, V, using a dynamic load factor of 1.25 and determine V/V_p. If V/V_p is less than 0.67, use the plastic design moment as computed in Step 4 (Section 5-31). If V/V_p is greater than 0.67, use Equation 5-23 to calculate the effective M_p.

Step 7. Calculate the ultimate unit resistance of the section (Table 3-1), using the equivalent plastic moment as obtained in Step 4 and a dynamic load factor of 1.25.

Step 8. Determine the moment of inertia of the plate section.

Step 9. Compute the equivalent elastic unit stiffness, K_E, of the plate section (Table 3-8).

Step 10. Calculate the equivalent elastic deflection, X_E, of the plate as given by $X_E = r_u/K_E$.

Step 11. Determine the load-mass factor K_{LM} and compute the effective unit mass, m_e.

Step 12. Compute the natural period of vibration, T_N.

Step 13. Determine the door plate response using the values of P/r_u and T/T_N and the response charts of Chapter 3. Determine X_m/X_E and T_E.

Step 14. Determine the support rotation,

$$\text{Tan } \theta = (X_m) / (L/2)$$

Compare θ with the design criteria of step 1b.

Step 15. Determine the strain rate, ϵ, using equation 5-1. Determine the dynamic increase factor using figure 5-2 and compare with the DIF selected in Step 1d.

If the criteria of Step 1 is not satisfied, repeat Steps 2 to 15 with a new plate thickness.

Step 16. Design supporting flexural element considering composite action with the plate (if so constructed).

Step 17. Calculate elastic and plastic section moduli of the combined section.

Step 18. Follow the design procedure for a flexural element as described in Section 5A-1.

Example 5A-7 (a) Design of a Blast Door for Pressure-Time Loading

Required: Design a double-leaf, built-up door (6 ft by 8 ft) for the given pressure-time loading.

Step 1. Given:

a. Pressure-time loading (Figure 5A-7)

b. Design criteria: This door is to protect personnel from exterior loading. Leakage into the structure is permitted but the maximum end rotation of any member is limited to 2° since panic hardware must be operable after an accidental explosion.

c. Structural configuration (Figure 5A-7)

NOTE:
This type of door configuration is suitable for low-pressure range applications.

d. Steel used: A36

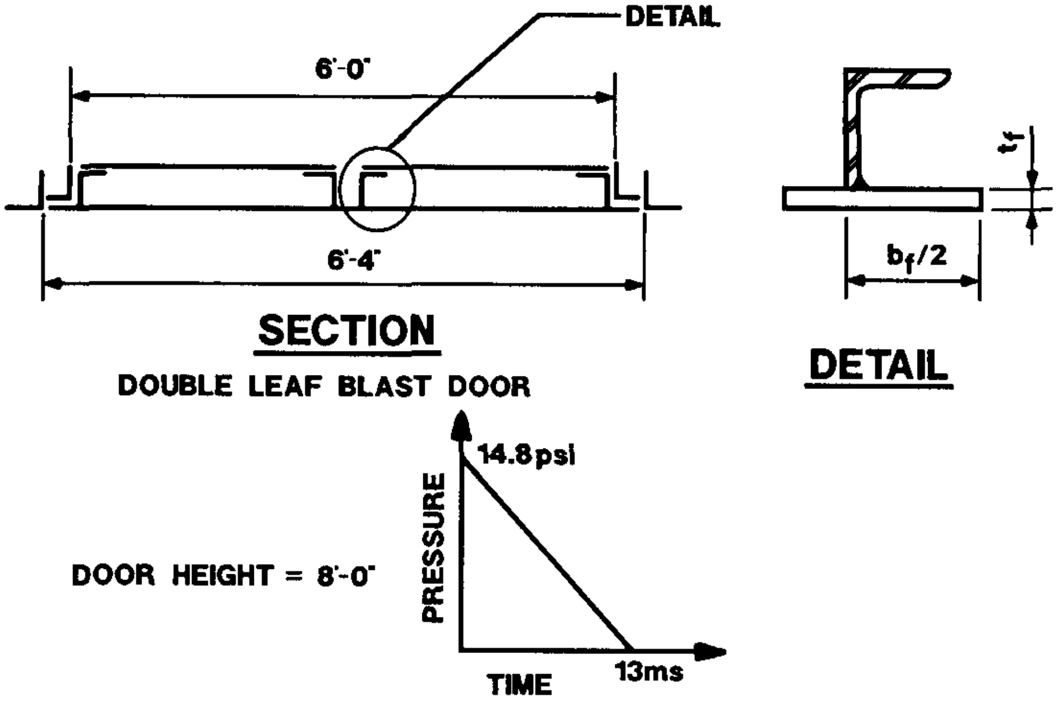


Figure 5A-7(a) Door configuration and loading, Example 5A-7(a)

Yield strength, $f_y = 36$ ksi (Table 5-1)

Dynamic increase factor, $c = 1.29$ (Table 5-2)

Average yield strength increase factor, $a = 1.1$ (Section 5-12.1)

Hence, the dynamic design stress,

$f_{ds} = 1.1 \times 1.29 \times 36 = 51.1$ ksi (Equation 5-2)

and the dynamic yield stress in shear,

$f_{dv} = 0.55\ f_{ds} = 0.55 \times 51.1 = 28.1$ ksi (Equation 5-4)

Step 2. Assume a plate thickness of 5/8 inch.

Step 3. Determine the elastic and plastic section moduli (per unit width).

$$S = \frac{bd^2}{6} = \frac{1 \times (5/8)^2}{6} = 6.515 \times 10^{-2} \text{ in}^3/\text{in}$$

$$Z = \frac{bd^2}{4} = \frac{1 \times (5/8)^2}{4} = 9.765 \times 10^{-2} \text{ in}^3/\text{in}$$

Step 4. Calculate the design plastic moment, M_p.

$$M_p = f_{ds} (S + Z)/2 = 51.1 [(6.515 \times 10^{-2}) + (9.765 \times 10^{-2})]/2 = 51.1 \times 8.14 \times 10^{-2} = 4.16 \text{ in-k/in} \quad \text{(Equation 5-7)}$$

Step 5. Calculate the dynamic ultimate shear capacity, V_p, for a 1-inch width.

$$V_p = f_{dv}A_w = 28.1 \times 1 \times 5/8 = 17.56 \text{ kips/in} \quad \text{(Equation 5-16)}$$

Step 6. Evaluate the support shear and check the plate capacity. Assume DLF = 1.25

$$V = DLF \times P \times L/2 = \frac{1.25 \times 14.8 \times 36 \times 1}{2} = 333 \text{ lb/in} = 0.333 \text{ kip/in}$$

$$V/V_p = 0.333/13.61 = 0.0245 < 0.67 \quad \text{(Section 5-31)}$$

No reduction in equivalent plastic moment is necessary.

NOTE:
When actual DLF is determined, reconsider Step 6.

Step 7. Calculate the ultimate unit resistance, r_u, (assuming the plate to be simply-supported at both ends).

$$r_u = 8M_p/L^2 = \frac{8 \times 4.16 \times 10^3}{(36)^2} = 25.7 \text{ psi} \quad \text{(Table 3-1)}$$

Step 8. Compute the moment of inertia, I, for a 1-inch width.

$$I = \frac{bd^3}{12} = \frac{1 \times (5/8)^3}{12} = 0.02035 \text{ in}^4/\text{in}$$

Step 9. Calculate the equivalent elastic stiffness, K_E.

$$K_E = 384EI/5bL^4 = \frac{384 \times 29 \times 106 \times .02035}{5 \times 1 \times (36)^4} = 27.0 \text{ psi/in}$$

(Table 3-8)

Step 10. Determine the equivalent elastic deflection, X_E.

$$X_E = r_u/K_E = 25.7/27.0 = 0.95 \text{ inch}$$

Step 11. Calculate the effective mass of element.

a. K_{LM} (average elastic and plastic) $= (0.78 + 0.66)/2 = 0.72$

b. Unit mass of element, m

$$m = w/g = \frac{5/8 \times 1 \times 1 \times 490 \times 10^6}{1,728 \times 32.2 \times 12} = 458.0 \text{ psi-ms}^2\text{/in}$$

c. Effective unit of mass of element, m_e

$$m_e = K_{LM}m = 0.72 \times 458.0$$

$$= 330 \text{ psi-ms}^2\text{/in}$$

Step 12. Calculate the natural period of vibration, T_N.

$$T_N = 2\pi\ (330/27.0)^{1/2} = 22 \text{ ms}$$

Step 13. Determine the door response.

Peak overpressure	$P = 14.8$ psi
Peak resistance	$r_u = 25.7$ psi
Duration	$T = 13.0$ ms
Natural period of vibration	$T_N = 22$ ms

$P/r_u = 14.8/25.7 = 0.58$

$T/T_N = 13.0/22.0 = 0.59$

From Figure 3-64a of Chapter 3,

$X_m/X_E < 1$

Since the response is elastic, determine the DLF from Figure 3-49 of Chapter 3.

DLF = 1.3 for $T/T_N = 0.59$

Step 14. Determine the support rotation.

$$X_m = \frac{1.3 \times 14.8 \times 0.95}{25.7} = 0.713 \text{ inch}$$

$$\tan \Theta = X_m/(L/2) = 0.713/(36/2) = 0.0396$$

$$\Theta = 2.27° > 2° \qquad \text{N.G.}$$

Step 15. Evaluate the selection of the dynamic increase factor.

Since this is an elastic response, use figure 3-49(b) of Chapter 3 to determine t_m. For $T/T_N = 0.59$, $t_m/T = 0.7$ and $t_m = 9.1$ ms. The strain rate is:

$$\dot{\epsilon} = f_{ds}/E_s t_E \qquad \text{(Equation 5-1)}$$

Since the response is elastic,

$$f_{ds} = 51.1 \times \frac{X_m}{X_E} = 51.1 \times \frac{0.713}{0.95} = 38.4 \text{ ksi}$$

and $t_E = t_m = 0.0091$ sec. Hence,

$$\dot{\epsilon} = 38.4/29.6 \times 10^3 \times 0.0091 = .142 \text{ in/in/sec}$$

Using figure 5-2, DIF = 1.31. The preliminary selection of DIF = 1.29 is acceptable.

Since the rotation criterion is not satisfied, change the thickness of the plate and repeat the procedure. Repeating these calculations, it can be shown that a 3/4-inch plate satisfies the requirements.

Step 16. Design of the supporting flexural element.

Assume an angle L4 x 3 x 1/2 and attached to the plate as shown in Figure 5A-7(b).

Determine the effective width of plate which acts in conjunction with the angle

$b_f/2t_f \leq 8.5$ (Section 5-24)

where $b_f/2$ is the half width of the outstanding flange or overhang and t_f is the thickness of the plate.

With t_f = 3/4 inch, $b_f/2 \leq 8.5 \times 3/4$, i.e., 6.38 inches

Use overhang of 6 inches.

Hence, the effective width = 6 + 2 = 8 inches. The angle together with plate is shown in Figure 5A-7(b).

Step 17. Calculate the elastic and plastic section moduli of the combined section.

Let $\bar{y}$ be the distance of c.g. of the combined section from the outside edge of the plate as shown in Figure 5A-7(b), therefore

$$\bar{y} = \frac{(8 \times 3/4 \times 3/8) + (4 + 3/4 - 1.33) \times 3.25}{(8 \times 3/4) + 3.25} = 1.445 \text{ inches}$$

Let y_p be the distance to the N.A. of the combined section for full plasticity.

$$y_p = \frac{1}{8 \times 2} [(8 \times 3/4) + 3.25] = 0.578 \text{ inch}$$

$$I = \frac{8 \times (3/4)^3}{12} + 8 \times 3/4 \times (1.45 - 3/8)^2$$

$$+ 5.05 + 3.25 (4 + 3/4 - 1.33 - 1.445)^2 = 24.881 \text{ in}^4$$

Hence, $S_{min} = 24.881/(4.75 - 1.445) = 7.54 \text{ in}^3$

$$Z = 8 (0.578)^2/2 + 8 (0.75 - 0.578)^2/2$$

$$+ 3.25 (4.75 - 1.33 - 0.578) = 10.69 \text{ in}^3$$

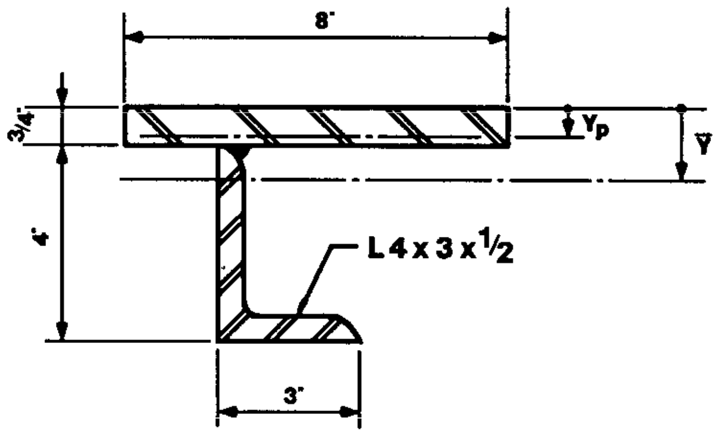


Figure 5A-7(b) Detail of composite angle/plate supporting element, Example 5A-7(b)

Step 18. Follow the design procedure for the composite element using steps 4 through 13. Calculate the design plastic moment M_p of the supporting flexural element.

$M_p = 51.1\ (7.54 + 10.69)/2 = 465.8$ in-kips (Equation 5-7)

Calculate the ultimate dynamic shear capacity, V_p.

$V_p = f_{dv}A_w = 28.1\ (4.0 - 1/2)\ 1/2 = 49.2$ kips (Equation 5-16)

Calculate support shear and check shear capacity.

L = 8'-0" = 96 inches

$V_p = (14.8 \times 36/2 \times 96)/2 = 12{,}790$ lb $= 12.79$ kips $< V$ O.K. (Section 5-23)

Calculate the ultimate unit resistance, r_u.

Assuming the angle to be simply supported at both ends:

$r_u = 8M_p/L^2 = (8 \times 465.8 \times 1{,}000)/(96)^2 = 405$ lb/in (Table 3-1)

Calculate the unit elastic stiffness, K_E.

$$K_E = 384EI/5L^4 = \frac{384 \times 29 \times 10^6 \times 24.881}{5 \times (96)^4} = 652.5 \text{ lb/in}^2$$

(Table 3-8)

Determine the equivalent elastic deflection, X_E.

$X_E = r_u/K_E = 405/652.5 = 0.620$ inch

Calculate the effective mass of the element.

$K_{LM} = 0.72$

$$w = \frac{11.1}{12} + \frac{3}{4} \times 18 \times \frac{490}{1{,}728} = (0.925 + 3.825) = 4.750 \text{ lb/in}$$

Effective unit mass of element,

$$m_e = 0.72 \times \frac{4.75 \times 10^6}{32.2 \times 12} = 0.89 \times 10^4 \text{ lb-ms}^2/\text{in}^2$$

Calculate the natural period of vibration, T_N.

$T_N = 2\pi[(89 \times 10^2)/652.5]^{1/2} = 23.2$ ms

Determine the response parameters. (Figure 3-64a)

Peak overpressure $P = 14.8 \times 36/2 = 266.5$ lb/in

Peak resistance $r_u = 405$ lb/in

Duration $T = 13.0$ ms

Natural period of vibration, $T_N = 23.2$ ms

$P/r_u = 266.5/405 = 0.658$

$T/T_N = 13/23.2 = 0.56$

From Figure 3-64a,

$$\mu = X_m/X_E < 1$$

From Figure 3-49 for $T/T_N = 0.56$,

DLF = 1.28

$$\text{Hence, } X_m = \frac{1.28 \times 14.8 \times 36/2}{652.5} = 0.522 \text{ in}$$

$$\tan \theta = X_m/(L/2) = 0.522/48 = 0.0109$$

$$\theta = 0.69° < 2° \qquad \text{O.K.}$$

Check stresses at the connecting point.

$$\sigma = My/I = 355 \times 10^3 \times (1.445 - 0.75)/24.881$$

$$= 9{,}900 \text{ psi } (M = \frac{X_m}{X_E} \times M_p = \frac{0.522}{0.62} \times 405 = 341)$$

$$\tau = VQ/Ib = \frac{12.79 \times 103 \times 8 \times 3/4 \times (1.445 - 0.75/2)}{24.881 \times 1/2} = 5{,}321 \text{ psi}$$

Effective stress at the section $= (\sigma^2 + \tau^2)^{1/2}$

$= 10^3 \times (9.9^2 + 5.321^2)^{1/2} = 11{,}239$ psi < 39.600 psi O.K.

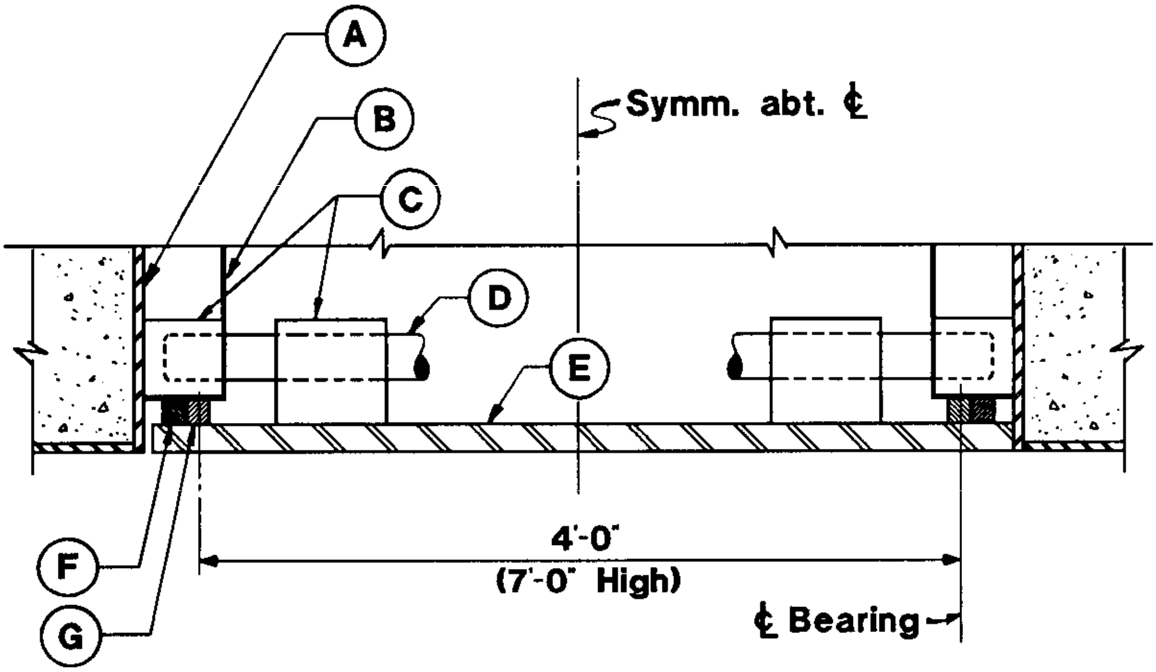
Example 5A-7 (b) Design of a Plate Blast Door for Pressure-Time Loading

Required: Design a single-leaf door (4 ft by 7 ft) for the given pressure-time loading.

Step 1. Given:

a. Pressure-time loading [figure 5A-7 (c)]

b. Design criteria: Door shall be designed to contain blast pressures from an internal explosion. Gasket and reversal



PLAN / SECTION

Legend:

A - Steel frame embedded in concrete
B - Steel sub-frame
C - Reversal bolt housing
D - Reversal bolt
E - Blast door plate
F - Continuous gasket
G - Continuous bearing block

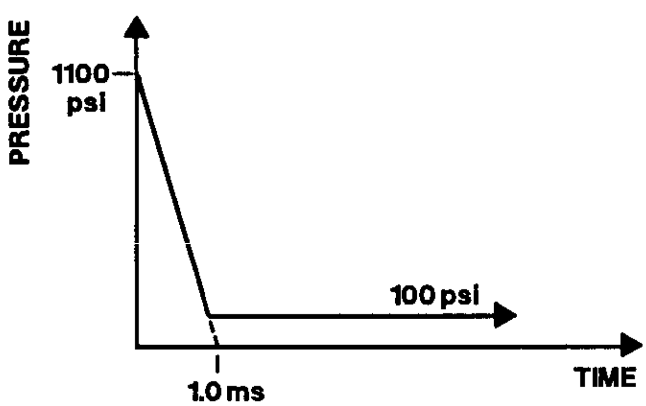


Figure 5A-7(c)Door configuration and loading, Example 5A-7(b) mechanisms shall be provided. Support rotation shall be limited to 3°

c. Structural configuration [see figure 5A-7 (c)]

Note: This type of door is suitable for high pressure range applications.

d. Steel used: ASTM A588

Yield strength, $f_y = 50$ ksi (Table 5-1)
Dynamic increase factor, $c = 1.24$ (preliminary, Table 5-2)

Average strength increase factor, $a = 1.1$ (Section 5-12.1)

Hence, the dynamic design stress,

$f_{ds} = 1.1 \times 1.24 \times 50 = 68.2$ ksi (Equation 5-2)

Note:
It is assumed, for the limited design rotation of 3° , that $\mu < 10$, and, therefore, that equation 5-3 does not govern.

The dynamic design stress in shear

$f_{dv} = 0.55\ f_{ds} = 37.5$ ksi (Equation 5-4)

Step 2. Assume a plate thickness of 2 inches

Step 3. Determine the elastic and plastic section moduli (per unit width)

$$S = \frac{bd^2}{6} = \frac{1\ (2)^2}{6} = 0.667\ \text{in}^3/\text{in}$$

$$Z = \frac{bd^2}{4} = \frac{1\ (2)^2}{4} = 1.0\ \text{in}^3/\text{in}$$

Step 4. Calculate the design plastic moment, M_p

$M_p = f_{ds}\ (S + Z)/2 = 68.2\ (0.667 + 1.0)/2 = 56.8$ in-k/in
(Equation 5-7)

Step 5. Calculate the dynamic ultimate shear capacity, V_p, for a 1-inch width.

$V_p = f_{dv}A_w = 37.5 \times 2 = 75.0$ kips/in (Equation 5-16)

Step 6. Evaluate the support shear and check the plate shear capacity.

Example 5A-7(b)

Assume DLF = 1.0 (Table 5-4)

For simplicity, assume the plate is a one-way member, hence:

$V = DLF \times P \times L/2 = 1.0 \times 1100 \times 48/2 = 26{,}400$ lb/in

$= 26.4$ kips/in

$V/V_p = 26.4/75. = 0.352 < 0.67$ (Section 5-31)

No reduction in equivalent plastic moment is necessary.

Step 7. Calculate the ultimate unit resistance, r_u.

For a plate, simply-supported on four sides (direct load)

$r_u = 5M_p/X^2$ (Table 3-2)

where $M_{HP} = M_p$ and $M_{HN} = 0$ and

$$\frac{X}{L} = 0.35 \quad \text{for} \quad \frac{L}{H} = 1.75$$ (Figure 3-17)

thus, $X = 0.35 \times 12 \times 7 = 29.4$ in

$r_u = 5 \times 56.8/(29.4)^2 = 329$ psi

Step 8. Compute the moment of inertia, I, for a 1-inch width

$$I = \frac{bd^3}{12} = \frac{1 \times 2^3}{12} = 0.667 \text{ in}^4/\text{in}$$

Step 9. Calculate the equivalent elastic stiffness, K_E

$K_E = r/x = D/\gamma\ H^4$ (Figure 3-36)

where

$\gamma = 0.0083$ (for H/L = 0.57)

$D = EI/b\ (1 - \nu^2)$ (Equation 3-33)

$D = 29.6 \times 10^6 \times 0.667/1\ (1 - .3^2) = 2.17 \times 10^7$

$K_E = 2.17 \times 10^7/0.0083 \times 48^4 = 492$ psi/in

Step 10. Determine the equivalent elastic deflection, X_E.

$X_E = r_u/K_E = 329/492 = 0.669$ in

Step 11. Calculate the effective mass of the element

a. K_{LM} (average elastic and plastic)

$= (0.78 + 0.66)/2 = 0.72$

b. Unit mass of element,

$$m = w/g = \frac{2 \times 1 \times 1 \times 490 \times 10^6}{1{,}728 \times 32.2 \times 12} = 1{,}468\ \frac{\text{psi-ms}^2}{\text{in}}$$

c. Effective unit mass of element, m_e

$$m_e = K_{LM} \times m = 0.72 \times 1{,}468 = 1{,}057\ \frac{\text{psi-ms}^2}{\text{in}}$$

Step 12. Calculate the natural period of vibration, T_N

$T_N = 2\pi\ (1{,}057/492)^{1/2} = 9.2$ ms

Step 13. Determine the door plate response for:

$P/r_u = 1100/329 = 3.34$

$T/T_N = 1.0/9.2 = 0.109$

$C_1 = 100/1100 = 0.091$ (Figure 3-62, Region C)

$C_2 > 1000$

Using Figure 3-253,

$X_m/X_E = 1.5$

$X_m = 1.5 \times 0.669 = 1.00$ in

Step 14. Determine the support rotation.

$\theta = \tan^{-1}(1/24) = 2.39° < 3°$

Step 15. Evaluate the selection of the dynamic increase factor.

$\dot{\epsilon} = f_{ds}/E_s t_E$ (Equation 5-1)

$t_E/T = 1.8, \quad t_E = 1.8$ ms (Figure 3-253)

$\dot{\epsilon} = 68.2/29.6 \times 10^3 \times .0018 = 1.28$ in./in./sec.

DIF = 1.3 (figure 5-2, average of A36 and A514)

Initial selection of DIF = 1.24 is adequate.

Since the support rotation criteria has been satisfied and the preliminary selection of the DIF is acceptable, a 2 inch thick plate is used in design.

Steps 15 through 18 These steps are bypassed since the door plate has no stiffening elements.

Problem 5A-8 Design of Doubly Symmetric Beams Subjected to Inclined Pressure-Time Loading

Problem: Design a purlin or girt as a flexural member which is subjected to a transverse pressure-time load acting in a plane other than a principal plane.

Procedure:

Step 1. Establish the design parameters.

a. Pressure-time load

b. Angle of inclination of the load with respect to the vertical axis of the section

c. Design criteria: Maximum support rotation limited to 2°.

d. Member spacing, b

e. Type and properties of steel used:

Minimum yield strength for the section (Table 5-1)

Dynamic increase factor, c (Table 5-2)

Step 2. Preliminary sizing of the beam.

a. Determine the equivalent static load, w, using a preliminary dynamic load factor equal to 1.0.

$$\therefore w = 1.0 \times p \times b$$

b. Using the appropriate resistance formula from Table 3-1 and the equivalent static load derived in Step 2a, determine the required M_p.

c. Determine the required section properties using equation 5-7. Select a larger section since the member is subjected to unsymmetrical bending.

Note that for a load inclination of 10°, it is necessary to increase the required average section modulus, (1/2) (S + Z), by 40 percent.

Step 3. Check local buckling of the member (Section 5-24).

Step 4. Calculate the inclination of the neutral axis (equation 5-24).

Step 5. Calculate the elastic and plastic section moduli of the section (equation 5-25).

Step 6. Compute the design plastic moment, M_p, (equation 5-6).

Step 7. Calculate ultimate unit resistance, r_u, of the member.

Step 8. Calculate elastic deflection, δ (Section 5-32.3).

Step 9. Determine the equivalent elastic unit stiffness, K_E, of the beam section using δ from Step 8.

Step 10. Compute the equivalent elastic deflection, X_E, of the member as given by $X_E = r_u/K_E$.

Step 11. Determine the load-mass factor, K_{LM}, and obtain the effective unit mass, m_e, of the element.

Step 12. Evaluate the natural period of vibration, T_N.

Step 13. Determine the dynamic response of the beam. Evaluate P/r_u and T/T_N, using the response charts of Chapter 3 to obtain X_m/X_E and θ. Compare with criteria.

Step 14. Determine the ultimate dynamic shear capacity, V_p, (equation 5-16) and maximum support shear, V, using Table 3-9 of Chapter 3 and check adequacy.

Example 5A-8 Design an I-Shaped Beam for Unsymmetrical Bending Due to Inclined Pressure-Time Loading

Required: Design a simply-supported I-shaped beam subjected to a pressure-time loading acting at an angle of 10° with respect to the principal vertical plane of the beam. This beam is part of a structure designed to protect personnel.

Step 1. Given:

a. Pressure-time loading [figure 5A-8 (a)]

b. Design criteria: The structure is to be designed for more than one "shot." A maximum end rotation = 1°, is therefore assigned

c. Structural configuration [figure 5A-8(a)]

d. Steel used: A36

Yield strength, $f_y = 36$ ksi (Table 5-1)

Dynamic increase factor, $c = 1.29$ (Table 5-2)

Average yield strength increase factor, $a = 1.1$ (Section 5-12.1)

Dynamic design strength, $f_{ds} = 1.1 \times 1.29 \times 36 = 51.1$ ksi (equation 5-2)

Dynamic yielding stress in shear, $f_{dv} = 0.55\ f_{ds} = 0.55 \times 51.1 = 28.1$ ksi (equation 5-4)

Modulus of elasticity, E = 29,000,000 psi

Step 2. Preliminary sizing of the member.

a. Determine equivalent static load.

Select DLF = 1.2 (Section 5-22.3)

$$w = 1.25 \times 4.5 \times 4.5 \times 144/1,000 = 3.65 \text{ k/ft}$$

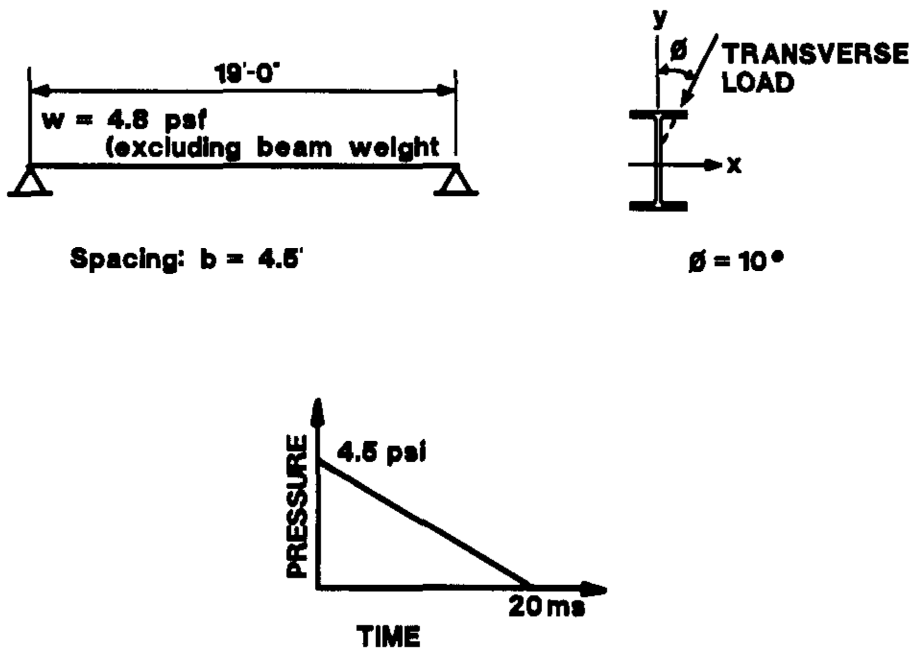


Figure 5A-8(a) Beam configuration and loading, Example 5A-8(a)

b. Determine minimum required M_p

$M_p = (wL^2)/8 = (3.65 \times 19^2)/8 = 165$ k-ft (Table 3-1)

c. Selection of a member.

For a load acting in the plane of the web,

$(S + Z) = 2M_p/f_{ds} = (2 \times 165 \times 12)/51.1$ (Equation 5-7)

$(S + Z) = 77.5 \text{ in}^3$

$(S + Z)$ required $1.4 \times 77.5 = 109 \text{ in}^3$

Try W14 x 38, $S_x = 54.6\ in^3$, $Z_x = 61.5\ in^3$

$(S + Z) = 116.1\ in^3$, $I_x = 385\ in^4$

$I_y = 26.7\ in^4$

Step 3. Check against local buckling.

For W14 x 38,

$d/t_w = 45.5 < (412 / (36)^{1/2})\ (1 - 1.4 \times P/P_y) = 68.66$ O.K. (Equation 5-17)

$b_f/2t_f = 6.6 < 8.5$ O.K. (Section 5-24)

Step 4. Inclination of elastic and plastic neutral axes with respect to the x-axis.

$\tan \alpha = (I_x/I_y) \tan \phi = (385/26.7) \tan 10° = 2.546$ (Equation 5-24)

$\alpha = 68.5°$

Calculate the equivalent elastic section modulus.

$S = (S_xS_y)/(S_y\cos \phi + S_x\sin \phi)$

$S_x = 54.6\ in^3$, $S_y = 7.88\ in^3$, $\phi = 10°$

$\sin 10° = 0.174$, $\cos 10° = 0.985$

$S = (54.6)\ (7.88)/(7.88 \times 0.985 + 54.7 \times 0.174) = 24.9\ in^3$

Step 5. Calculate the plastic section modulus, Z.

$Z = A_cm_1 + A_tm_2$ (Equation 5-6)

$A_c = A_t = A/2 = 11.2/2 = 5.6\ in^2$

Let $\bar{y}$ be the distance of the c.g. of the area of cross section in compression from origin as shown in Figure 5A-8 (b).

$$\bar{y} = \frac{1}{5.6}\Bigg[6.770 \times 0.515 \times \Big(\frac{14.10}{2} - \frac{0.515}{2}$$

$$+ \frac{1}{2}(14.10 - 2 \times 0.515) \times 0.310$$

$$\times \frac{1}{2}\Big(\frac{14.10}{2} - 0.515\Big)\Bigg] = 5.42 \text{ inches}$$

$m_1 = m_2 = \bar{y} \sin\alpha = 5.42 \sin 68° \; 30' = 5.05$ inches

$Z = 2A_c m_1 = 11.2 \times 5.05 = 56.5 \text{ in}^3$

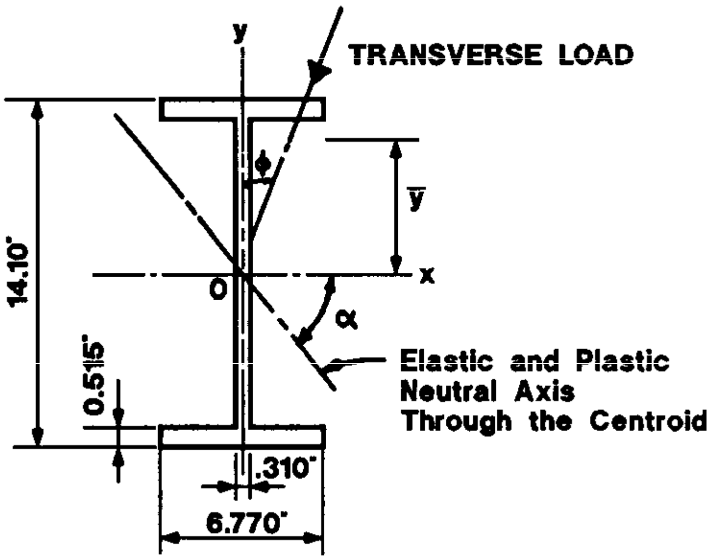


Figure 5A-8(b) Loading on beam section, Example 5A-8(b)

Step 6. Determine design plastic moment, M_p.

$M_p = f_{ds}(S + Z)/2 = 51.1(24.9 + 56.5)/2$ (Equation 5-7)

$= 51.1 \times 40.7 = 2{,}080$ in-kips

Step 7. Calculate ultimate unit resistance, r_u.

$r_u = 8M_p/L^2 = (8)\ (2{,}080)\ (1{,}000)/(19 \times 12)^2 = 320$ lb/in (Table 3-1)

Step 8. Compute elastic deflection, δ.

$\delta = [(\delta_x{}^2 + \delta_y{}^2)]^{1/2}$ (Section 5-32.3)

$$\delta_y = \frac{5w \cos\phi\ L^4}{384EI_x}$$

$$\delta_x = \frac{5w \sin\phi\ L^4}{384EI_y}$$

w = equivalent static load + dead load

$$= 2.92 + \frac{(4.8 \times 4.5)\ + 38}{1{,}000}\ \text{kips/ft} = 2.94\ \text{kips/ft}$$

$$\delta = \left[\frac{[\ 5w\sin\phi L^4\ ]^2}{384EI_y} + \frac{[\ 5w\cos\phi L^4\ ]^2}{384EI_x}\right]^{1/2}$$

$$= \frac{5wL^4}{38{,}400E}\ [(0.652)^2 + (0.256)^2\ ]^{1/2} = 2.08\ \text{inches}$$

Step 9. Calculate the equivalent elastic unit stiffness, K_E.

$$K_E = w/\delta = \frac{2.94 \times 1{,}000 \times 1}{12 \times 2.085} = 117.8\ \text{lb/in}^2$$

(Get w from Step 8)

Step 10. Determine the equivalent elastic deflection, X_E.

$X_E = r_u/K_E = 320/117.8 = 2.72$ inches

Step 11. Calculate the effective mass of the element, m_e.

a. Load-mass factor, K_{LM} (Table 3-12)

K_{LM} (average elastic and plastic)

$= (0.78 + 0.66)/2 = 0.72$

b. Unit mass of element, m

$$m = w/g = \frac{[(4.5 \times 4.8) + 38] \times 10^6}{32.2 \times 12 \times 12} = 1.286 \times 10^4 \text{ lb-ms}^2/\text{in}^2$$

c. Effective unit mass of element, m_e

$$m_e = K_{LM}m = 0.72 \times 1.286 \times 10^4 = 0.93 \times 10^4 \text{ lb-ms}^2/\text{in}^2$$

Step 12. Calculate the natural period of vibration, T_N.

$T_N = 2\pi\ [(93 \times 10^2)/117.8]^{1/2} = 55.8$ ms (Section 5-22.2)

Step 13. Determine the beam response.

Peak overpressure $P = 4.5 \times 4.5 \times 12 = 243$ lb/in

Peak resistance $r_u = 320$ lb/in

Duration $T = 20$ ms

Natural period of vibration, $T_N = 55.8$ ms

$P/r_u = 243/320 = 0.76$

$T/T_N = 20/55.8 = 0.358$

From figure 3-64a

$X_m/X_E < 1$

From figure 3-49, for $T/T_N = 0.358$,

DLF = 0.97

Hence, X_m = 0.97 x 4.5 x 4.5 x 12/117.8 = 2.0 in

Find end rotation, θ.

tan θ = X_m/(L/2) = 2.0/[(19 x 12)/2] = 0.0175

θ = 1.0°   O.K.

Step 14. Calculate the dynamic ultimate shear capacity, V_p, and check for adequacy.

$V_p = f_{dv}A_w$ = 28.1 (14.10 - 2 x 0.515) (0.310) = 113.9 kips (Equation 5-16)

V = DLF x P x b x L/2

= 0.97 x 4.5 x 4.5 x 19 x 144/(2 x 1,000)

= 26.9 kips < 89.2 kips < V_p O.K.   (Table 3-9)

APPENDIX 5B

LIST OF SYMBOLS

a	yield stress increase factor

A	Area of cross section (in^2)

A_b	Area of bracing member (in^2)

A_c	Area of cross section in compression (in^2)

A_t	Area of cross section in tension (in^2)

A_w	Web area (in^2)

b	Width of tributary loaded area (ft)

b_f	Flange width (in)

b_h	Tributary width for horizontal loading (ft)

b_v	Tributary width for vertical loading (ft)

c,DIF (1)	Dynamic increase factor
c	(2)	Distance from neutral axis to extreme fiber of cross-section in flexure (in)

C, C_1	Coefficients indicating relative column to girder moment capacity (Section 5-42.1)

C_b	Bending coefficient defined in Section 1.5.1.4.5 of the AISC Specification

C_c	Column slenderness ratio indicating the transition from elastic to inelastic buckling

C_{mx}, C_{my}	Coefficients applied to the bending terms in interaction formula (AISC Specification Section 1.6.1)

C_2	Coefficient in approximate expression for sidesway stiffness factor (Table 5-14)

D	Coefficient indicating relative girder to column stiffness (Table 5-14)

DLF	Dynamic load factor

d	(1)	Web depth (in)
	(2)	Diameter of cylindrical portion of fragment, in.

E	Young's modulus of elasticity (psi)

f	(1)	Maximum bending stress (psi)
	(2)	Shape factor, S/Z

f_a Axial stress permitted in the absence of bending moment from Section 5-37.3 (psi)

f_b Bending stress permitted in the absence of axial force (psi)

f_{cr} Web buckling stress (psi)

f_d Maximum dynamic design stress for connections (psi)

f_{ds} Dynamic design stress for bending, tension and compression (psi)

f_{dv} Dynamic yielding shear stress (psi)

f_{dv} Dynamic ultimate stress (psi)

f_{dy} Dynamic yield stress (psi)

F'_{ex}, F'_{ey} Euler buckling stresses divided by safety factor (psi)

F_H Horizontal component of force in bracing member (lb)

F_s Allowable static design stress for connections (psi)

f_u Ultimate tensile stress (psi)

f_y Minimum static yield stress (psi)

g Acceleration due to gravity (386 in/sec^2)

H Story height (ft)

h Web depth for cold-formed, light gauge steel panel sections (in)

I Moment of inertia (in^4)

I_{ca} Average column moment of inertia for single-story multi-bay frame(in^4)

I_{20} Effective moment of inertia for cold-formed section at a service stress of 20 ksi (in^4 per foot width)

I_x Moment of inertia about the x-axis (in^4)

I_y Moment of inertia about the y-axis (in^4)

K (1) Effective length factor for a compression member
(2) Stiffness factor for rigid single-story, multi-bay frame from Table 5-14

K_b Horizontal stiffness of diagonal bracing (lb/ft)

K_E Equivalent elastic stiffness (lb/in or psi/in)

K_L Load factor

K_{LM} Load-mass factor

K_M Mass factor

L (1) Span length (ft or in)
(2) Frame bay width (ft)

l Distance between cross section braced against twist or lateral displacement of compression flange or distance between points of lateral support for beams or columns

l/r Slenderness ratio

l_b Actual unbraced length in the plane of bending (in)

l_{cr} Critical unbraced length (in)

M Total effective mass ($lb\text{-}ms^2/in$)

M_{mx}, M_{my} Moments about the x- and y-axis that can be resisted by member in the absence of axial load

M_p Design plastic moment capacity

M_1, M_2 Design plastic moment capacities (figures 5-6 and 5-10)

M_{px}, M_{py} Plastic bending moment capacities about the x- and y-axes

M_{pu} Ultimate dynamic moment capacity

M_{up} Ultimate positive moment capacity for unit width of panel

M_{un} Ultimate negative moment capacity for unit width of panel

M_y Moment corresponding to first yield

m (1) Unit mass of panel ($psi\text{-}ms^2/in$)
(2) Number of braced bays in multi-bay frame

m_e Effective unit mass ($psi\text{-}ms^2/in$)

m_1 Distance from plastic neutral axis to the centroid of the area in compression in a fully plastic section (in)

m_2 Distance from plastic neutral axis to the centroid of the area in tension in a fully plastic section (in)

N Bearing length at support for cold-formed steel panel (in)

n Number of bays in multi-bay frame

P (1) Applied compressive load (lb)

(2) Peak pressure of equivalent triangular loading function, (psi) [when used with r_u], or peak total blast load (lb) [when used with R_u].

P_{ex}, P_{ey} Euler buckling loads about the x- and y-axes

P_p Ultimate capacity for dynamic axial load, Af_{dy} (lb)

P_u Ultimate axial compressive load (lb)

P_y Ultimate capacity for static axial load Af_y (lb)

P_h Reflected blast pressure on front wall (psi)

P_v Blast overpressure on roof (psi)

Q_u Ultimate support capacity (lb)

q_h Peak horizontal load on rigid frame (lb/ft)

q_v Peak vertical load on rigid frame (lb/ft)

R Equivalent total horizontal static load on frame (lb)

R_u Ultimate total flexural resistance (lb)

r_b Radius of gyration of bracing member (in)

r_T Radius of gyration, equation 5-22 (in)

r_u Ultimate unit flexural resistance (psi)

r_x, r_y Radii of gyration about the x- and y-axes (in)

$\bar{r}$ Required resistance for elastic behavior in rebound (psi)

S Elastic section modulus (in^3)

S_x, S_y Elastic section modulus about the x- and y-axes (in^3)

S^+ Effective section modulus of cold-formed section for positive moments (in^3)

S^- Effective section modulus of cold-formed section for negative moments (in^3)

T Load duration (sec)

T_N Natural period of vibration (sec)

t (1) Thickness of plate element (in)
(2) Thickness of panel section (in)

t_E Time to yield (sec)

t_f Flange thickness (in)

t_m Time to maximum response (sec)

t_w Web thickness (in)

V Support shear (lb)

V Ultimate shear capacity (lb)

V_r Residual velocity of fragment (fps)

V_s Striking velocity of fragments, (fps)

V_x Critical perforation velocity of fragment (fps)

W Total weight (lb)

W_c Total concentrated load (lb)

W_E External work (lb-in)

W_f Fragment weight (oz.)

W_I Internal work (lb-in)

w (1) Flat width of plate element (in)
(2) Load per unit area (psi)
(3) Load per unit length (lb/ft)

X_o Deflection at design ductility ratio (figure 5-12)

X_E Equivalent elastic deflection (in)

X_m Maximum deflection (in)

x Depth of penetration of steel fragments (in)

Z Plastic section modulus (in^3)

Z_x, Z_y Plastic section moduli about the x- and y-axes (in^3)

α (1) Angle between the horizontal principal plane of the cross section and the neutral axis (deg)
(2) Ratio of horizontal to vertical loading on a frame

β (1) Base fixity factor (Table 5-14)
(2) Support condition coefficient (Section 5-34.3)
(3) Critical length for bracing correction factor (Section 5-26.3)

γ Angle between bracing member and a horizontal plane (deg)

δ (1) Total transverse elastic deflection (in)
(2) Lateral (sidesway) deflection (in)

ϵ Strain (in/in)

$\dot{\epsilon}$ Average strain rate (in/in/sec)

θ (1) Member end rotation (deg)
(2) Plastic hinge rotation (deg)

θ_{max} Maximum permitted member end rotation

μ Ductility ratio

μ_{max} Maximum permitted ductility ratio

ϕ Angle between the plane of the load and the vertical principal plane of the cross section (deg)

APPENDIX 5C

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CHAPTER 6. SPECIAL CONSIDERATIONS IN EXPLOSIVE FACILITY DESIGN

CHAPTER 6

SPECIAL CONSIDERATIONS IN EXPLOSIVE FACILITY DESIGN

INTRODUCTION

6-1. Purpose

The purpose of this manual is to present methods of design for protective construction used in facilities for development, testing, production, storage, maintenance, modification, inspection, demilitarization, and disposal of explosive materials.

6-2. Objective

The primary objectives are to establish design procedures and construction techniques whereby propagation of explosion (from one structure or part of a structure to another) or mass detonation can be prevented and to provide protection for personnel and valuable equipment.

The secondary objectives are to:

(1) Establish the blast load parameters required for design of protective structures.
(2) Provide methods for calculating the dynamic response of structural elements including reinforced concrete, and structural steel.
(3) Establish construction details and procedures necessary to afford the required strength to resist the applied blast loads.
(4) Establish guidelines for siting explosive facilities to obtain maximum cost effectiveness in both the planning and structural arrangements, providing closures, and preventing damage to interior portions of structures because of structural motion, shock, and fragment perforation.

6-3. Background

For the first 60 years of the 20th century, criteria and methods based upon results of catastrophic events were used for the design of explosive facilities. The criteria and methods did not include a detailed or reliable quantitative basis for assessing the degree of protection afforded by the protective facility. In the late 1960's quantitative procedures were set forth in the first edition of the present manual, "Structures to Resist the Effects of Accidental Explosions". This manual was based on extensive research and development programs which permitted a more reliable approach to current and future design requirements. Since the original publication of this manual, more extensive testing and development programs have taken place. This additional research included work with materials other than reinforced concrete which was the principal construction material referenced in the initial version of the manual.

Modern methods for the manufacture and storage of explosive materials, which include many exotic chemicals, fuels, and propellants, require less space for a given quantity of explosive material than was previously needed. Such concentration of explosives increases the possibility of the propagation of accidental explosions. (One accidental explosion causing the detonation of other explosive materials.) It is evident that a requirement for more accurate design techniques is essential. This manual describes rational design methods to provide the required structural protection.

These design methods account for the close-in effects of a detonation including the high pressures and the nonuniformity of blast loading on protective structures or barriers. These methods also account for intermediate and far-range effects for the design of structures located away from the explosion. The dynamic response of structures, constructed of various materials, or combination of materials, can be calculated, and details are given to provide the strength and ductility required by the design. The design approach is directed primarily toward protective structures subjected to the effects of a high explosive detonation. However, this approach is general, and it is applicable to the design of other explosive environments as well as other explosive materials as mentioned above.

The design techniques set forth in this manual are based upon the results of numerous full- and small-scale structural response and explosive effects tests of various materials conducted in conjunction with the development of this manual and/or related projects.

6-4. Scope

It is not the intent of this manual to establish safety criteria. Applicable documents should be consulted for this purpose. Response predictions for personnel and equipment are included for information.

In this manual an effort is made to cover the more probable design situations. However, sufficient general information on protective design techniques has been included in order that application of the basic theory can be made to situations other than those which were fully considered.

This manual is applicable to the design of protective structures subjected to the effects associated with high explosive detonations. For these design situations, the manual will apply for explosive quantities less than 25,000 pounds for close-in effects. However, this manual is also applicable to other situations such as far- or intermediate-range effects. For these latter cases the design procedures are applicable for explosive quantities in the order of 500,000 pounds which is the maximum quantity of high explosive approved for aboveground storage facilities in the Department of Defense manual, "Ammunition and Explosives Safety Standards", DOD 6055.9-STD. Since tests were primarily directed toward the response of structural steel and reinforced concrete elements to blast overpressures, this manual concentrates on design procedures and techniques for these materials. However, this does not imply that concrete and steel are the only useful materials for protective construction. Tests to establish the response of wood, brick blocks, and plastics, as well as the blast attenuating and mass effects of soil are contemplated. The results of these tests may require, at a later date, the supplementation of these design methods for these and other materials.

Other manuals are available to design protective structures against the effects of high explosive or nuclear detonations. The procedures in these manuals will quite often complement this manual and should be consulted for specific applications.

Computer programs, which are consistent with procedures and techniques contained in the manual, have been approved by the appropriate representative of the US Army, the US Navy, the US Air Force and the Department of Defense Explosives Safety Board (DDESB). These programs are available through the following repositories:

(1) Department of the Army
Commander and Director
U.S. Army Engineer
Waterways Experiment Station
Post Office Box 631
Vicksburg, Mississippi 39180-0631
Attn: WESKA

(2) Department of the Navy
Commanding Officer
Naval Civil Engineering Laboratory
Port Hueneme, California 93043
Attn: Code L51

(3) Department of the Air Force
Aerospace Structures
Information and Analysis Center
Wright Patterson Air Force Base
Ohio 45433
Attn: AFFDL/FBR

If any modifications to these programs are required, they will be submitted for review by DDESB and the above services. Upon concurrence of the revisions, the necessary changes will be made and notification of the changes will be made by the individual repositories.

6-5. Format

This manual is subdivided into six specific chapters dealing with various aspects of design. The titles of these chapters are as follows:

Chapter 1	Introduction
Chapter 2	Blast, Fragment, and Shock Loads
Chapter 3	Principles of Dynamic Analysis
Chapter 4	Reinforced Concrete Design
Chapter 5	Structural Steel Design
Chapter 6	Special Considerations in Explosive Facility Design

When applicable, illustrative examples are included in the Appendices.

Commonly accepted symbols are used as much as possible. However, protective design involves many different scientific and engineering fields, and, therefore, no attempt is made to standardize completely all the symbols used. Each symbol is defined where it is first used, and in the list of symbols at the end of each chapter.

CHAPTER CONTENTS

6-6. General

This chapter contains procedures for the design of blast resistant structures other than above ground, cast-in-place concrete or structural steel structures, as well as the design of other miscellaneous blast resistant components. Included herein is the design of reinforced and non-reinforced masonry walls, recast elements both prestressed and conventionally reinforced, pre-engineered buildings, suppressive shielding, blast resistant windows, underground structures, earth-covered arch-type magazines, blast valves and shock isolation systems.

MASONRY

6-7. Application

Masonry units are used primarily for wall construction. These units may be used for both exterior walls subjected to blast overpressures and interior walls subjected to inertial effects due to building motions. Basic variations in wall configurations may be related to the type of masonry unit such as brick, clay tile or solid and hollow concrete masonry units (CM), and the manner in which these units are laid (running bond, stack bond, etc.), the number of wythes of units (single or double), and the basic lateral load - carrying mechanism (reinforced or non-reinforced, one or two-way elements).

In addition to their inherent advantages with respect to fire protection, acoustical and thermal insulation, structural mass and resistance to flying debris, masonry walls when properly designed and detailed can provide economical resistance to relatively low blast pressures. However, the limitation on their application includes a limited capability for large deformations, reduced capacity in rebound due to tensile cracking in the primary phase of the response as well as the limitations on the amount and type of reinforcement which can be provided. Because of these limitations, masonry construction in this Manual is limited to concrete masonry unit (CM) walls placed in a running bond and with single or multiple wythes. However, because of the difficulty to achieve the required interaction between the individual wythes, the use of multiple wythes should be avoided.

Except for small structures (such as tool sheds, garage, etc.) where the floor area of the building is relatively small and interconnecting block walls can function as shear walls, masonry walls will usually require supplementary framing to transmit the lateral forces produced by the blast forces to the building foundation. Supplementary framing is generally classified into two categories (depending on the type of construction used); namely (1) flexible type supports such as structural steel framing, and (2) rigid supports including reinforced concrete frames or shear wall slab construction. The use of masonry walls in combination with structural steel frames is usually limited to incident over-pressures of 2 psi or less while masonry walls when supported by rigid supports may be designed to resist incident pressures as high as 10 psi. Figures 6-1 and 6-2 illustrate these masonry support systems.

Depending on the type of construction used, masonry walls may be classified into three categories: namely (1) cavity walls, (2) solid walls, and (3) a combination of cavity and solid walls. The cavity walls utilize hollow load - bearing concrete masonry units conforming to ASTM C90. Solid walls use solid load-bearing concrete masonry units conforming to ASTM C145 or hollow units whose cells and voids are filled with grout. The combined cavity and solid walls utilize the combination of hollow and solid units. Masonry walls may be subdivided further depending on the type of load-carrying mechanism desired: (1) joint reinforced masonry construction, (2) combined joint and cell reinforced masonry construction, and (3) non-reinforced masonry construction.

Joint reinforced masonry construction consists of single or multiple wythes walls and utilizes either hollow or solid masonry units. The joint reinforced wall construction utilizes commercially available cold drawn wire assemblies (see Figure 6-3), which are placed in the bed joints between the rows of the masonry units. Two types of reinforcement are available; truss and ladder types. The truss reinforcement provides the more rigid system and, therefore, is recommended for use in blast resistance structures. In the event that double wythes are used, each wythe must be reinforced independently. The wythes must also be tied together using wire ties. Joint reinforced masonry construction is generally used in combination with flexible type supports. The cells of the units located at the wall supports must be filled with grout. Typical joint reinforced masonry construction is illustrated in Figure 6-4.

Combined joint and cell reinforcement masonry construction consists of single wythe walls which utilize both horizontal and vertical reinforcement. The horizontal reinforcement may consist either of the joint reinforcement previously discussed or reinforcing bars. Where reinforcing bars are used, special masonry units are used which permit the reinforcement to sit below the joint (Figure 6-5). The vertical reinforcement consists of reinforcing bars which are positioned in one or more of the masonry units cells. All cells, which contain reinforcing bars, must be filled with grout. Depending on the amount of reinforcement used, this type of construction may be used with either the flexible or rigid type support systems.

Non-reinforced masonry construction consists of single wythe of hollow or solid masonry units. This type of construction does not utilize reinforcement for strength but solely relies on the arching action of the masonry units formed by the wall deflection and support resistance (Figure 6-6). This form of construction is utilized with the rigid type support system and, in particular, the shear wall and slab construction system.

6-8. Design Criteria for Reinforced Masonry Walls

6-8.1. Static Capacity of Reinforced Masonry Units

Figure 6-7 illustrates typical shapes and sizes of concrete masonry units which are commercially available. Hollow masonry units shall conform to ASTM C90, Grade N. This grade is recommended for use in exterior below and above grade and for interior walls. The minimum dimensions of the components of hollow masonry units are given in Table 6-1.

The specific compressive strength (f'_m) for concrete masonry units may be taken as:

Type of Unit	Ultimate Strength (f'_m)
Hollow Units	1350 psi
Hollow Units filled with grout	1500 psi
Solid Units	1800 psi

while the modulus of elasticity (E_m) of masonry units is equal to:

$$E_m = 1000 f'_m \qquad 6\text{-}1$$

The specific compressive strength and the modulus of elasticity of the mortar may be assumed to be equal to that of the unit.

Joint reinforcement shall conform to the requirements of ASTM A82 and, therefore, it will have a minimum ultimate (f_{un}) and yield (f_m) stresses equal to 80 ksi and 70 ksi respectively. Reinforcing bars shall conform to ASTM

A615 (Grade 60) and have minimum ultimate stress (f_{un}) of 90 ksi and minimum yield stress (f_m) of 60 ksi. The modulus of elasticity of the reinforcement is equal to 29,000,000 psi.

6-8.2 Dynamic Strength of Material

Since design for blast resistant structures is based on ultimate strength, the actual yield stresses of the material, rather than conventional design stresses or specific minimum yield stresses, are used for determining the plastic strengths of members. Further, under the rapid rates of straining that occur in structures loaded by blast forces, materials develop higher strengths than they do in the case of statically loaded members. In calculating the dynamic properties of concrete masonry construction it is recommended that the dynamic increase factor be applied to the static yield strengths of the various components as follows:

Concrete

Flexure	$1.19\ f'_m$
Shear	$1.00\ f'_m$
Compression	$1.12\ f'_m$

Reinforcement

Flexure	$1.17\ f_m$

6-8.3 Ultimate Strength of Reinforced Concrete Masonry Walls

The ultimate moment capacity of joint reinforced masonry construction may be conservatively estimated by utilizing the horizontal reinforcement only and neglecting the compressive strength afforded by the concrete. That is the reinforcement in one face will develop the tension forces while the steel in the opposite face resists the compression stresses. The ultimate moment relationship may be expressed for each horizontal joint of the wall as follows:

$$M_u = A_s f_{dy} d_c \qquad \text{6-2}$$

where:

A_s = area of joint reinforcement at one face

f = dynamic yield strength of the joint reinforcement

d_c = distance between the centroids of the compression and tension reinforcement

M_u = ultimate moment capacity

On the contrary, the ultimate moment capacity of the cell reinforcement (vertical reinforcement) in a combined joint and cell reinforced masonry construction utilizes the concrete strength to resist the compression forces.

The method of calculating ultimate moment of the vertical reinforcement is the same as that presented in Chapter 4 of this manual which is similar to that presented in the American Concrete Institute Standard Building Code Requirements for Reinforced Concrete.

The ultimate shear stress in joint reinforced masonry walls is computed by the formula:

$$v_u = V_u / A_n \qquad \text{6-3}$$

where:

v_u = unit shear stress

V_u = total applied design shear at $d_c/2$ from the support

A_n = net area of section

In all cases, joint reinforced masonry walls, which are designed to resist blast pressures, shall utilize shear reinforcement which shall be designed to carry the total shear stress. Shear reinforcement shall consist of; (1) bars or stirrups perpendicular to the longitudinal reinforcement, (2) longitudinal bars bent so that the axis or inclined portion of the bent bar makes an angle of 45 degrees or more with the axis of the longitudinal part of the bar; or (3) a combination of (1) and (2) above. The area of the shear reinforcement placed perpendicular to the flexural steel shall be computed by the formula:

$$A_v = \frac{v_u b s}{\phi f_m} \qquad \text{6-4}$$

where:

A_v = area of shear reinforcement

b = unit width of wall

s = spacing between stirrups

f_m = yield stress of the shear reinforcement

ϕ = strength reduction factor equal to 0.85

When bent or inclined bars are used, the area of shear reinforcement shall be calculated using:

$$A_v = \frac{v_u\, bs}{\phi f_m (\sin\alpha + \cos\alpha)} \qquad \text{6-5}$$

where:

α = angle between inclined stirrup and longitudinal axis of the member.

Shear reinforcement in walls shall be spaced so that every 45 degree line extending from mid depth ($d_c/2$) of a wall to the tension bars, crosses at least one line of shear reinforcement.

Cell reinforced masonry walls essentially consist of solid concrete elements. Therefore, the relationships, for reinforced concrete as presented in Chapter 4 of this manual may also be used to determine the ultimate shear stresses in cell reinforced masonry walls. Shear reinforcement for cell reinforced walls may only be added to the horizontal joint similar to joint reinforced masonry walls.

6-8.4. Dynamic Analysis

The principles for dynamic analysis of the response of structural elements to blast loads are presented in Chapter 3 of this manual. These principles also apply to blast analyses of masonry walls. In order to.perform these analyses, certain dynamic properties must be established as follows:

Load-mass factors, for masonry walls spanning in either one direction (joint reinforced masonry construction) or two directions (combined joint and cell reinforced masonry construction) are the same as those load-mass factors which are listed in Tables 3-12 and 3-13. The load-mass factors are applied to the actual mass of the wall. The weights of masonry wall can be determined based on the properties of hollow masonry units previously described and utilizing a concrete unit weight of 150 pounds per cubic foot. The values of the loadmass factors K_{LM}, will depend in part on the range of behavior of the wall; i.e., elastic, elasto-plastic, and plastic ranges. An average value of the elastic and elasto-plastic value of K_{LM} is used for the elasto-plastic range while an average value of the average K_{LM} for the elasto-plastic range and K_{LM} of the plastic range is used for the wall behavior in the plastic range.

The resistance-deflection function is illustrated in Figure 3-1. This figure illustrates the various ranges of behavior previously discussed and defines the relationship between the wall's resistances and deflections as well as presents the stiffness K in each range of behavior. It may be noted in Figure 3-1, that the elastic and elasto-plastic ranges of behavior have been idealized forming a bilinear (or trilinear) function. The equations for defining these functions are presented in Section 3-13.

The ultimate resistance r_u, of a wall varies; (1) as the distribution of the applied load, (2) geometry of the wall (length and width), (3) the amount and distribution of the reinforcement, and (4) the number and type of supports. The ultimate resistances of both one and two-way spanning walls are given in Section 3-9.

Recommended maximum deflection criteria for masonry walls subjected to blast loads is presented in Table 6-2. This table includes criteria for both reusable and non-reusable conditions as well as criteria for both one and two-way spanning walls.

When designing masonry walls for blast loads using response chart procedures of Chapter 3 the effective natural period of vibration is required. This effective period of vibration when related to the duration of the blast loading of given intensity and a given resistance of the masonry wall determines the maximum transient deflection X_m of the wall. The expression for the natural period of vibration is presented in equation 3-60, where the effective unit mass m_e has been described previously and the equivalent unit stiffness K_E is obtained from the resistance-deflection function. The equivalent stiffness of one way beams is presented in Table 3-8. This table may be used for one way spanning walls except that a unit width shall be used. Methods for determining the stiffnesses and period of vibrations for two-way walls are presented in Sections 3-11 through 3-13. Determining the stiffness in the elastic and elasto-plastic range is complicated by the fact that the moment of inertia of the cross section along the masonry wall changes continually as cracking progresses, and further by the fact that the modulus of elasticity changes as the stress increases. It is recommended that computations for deflections and therefore, stiffnesses be based on average moments of inertia I_a as follows:

$$I_a = \frac{I_n + I_c}{2} \qquad \text{6-6}$$

In Equation 6-6, I_n is the moment of inertia of the net section and I_c is the moment of inertia of the cracked section. For solid masonry units the value of I_n is replaced with the moment of inertia of the gross section. The values of I_n and I_g for hollow and solid masonry units used in joint reinforced masonry construction are listed in Table 6-3. The values of I_g for solid units may also be used for walls which utilize combined joint and cell masonry construction. The values of I_c for both hollow and solid masonry construction may be obtained using:

$$I_c = 0.005\, bd_c^3 \qquad \text{6-7}$$

6-8.5. Rebound

Vibratory action of a masonry wall will result in negative deflections after the maximum positive deflection has been attained. This negative deflection is associated with negative forces which will require tension reinforcement to be positioned at the opposite side of the wall from the primary reinforcement. In addition, wall ties are required to assure that the wall is supported by the frame (Figure 6-8). The rebound forces are a function of the maximum resistance of the wall as well as the vibratory properties of the wall and the load duration. The maximum elastic rebound of a masonry wall may be obtained from Figure 3-268.

6-9. Non-Reinforced Masonry Walls

The resistance of non-reinforced masonry walls to lateral blast loads is a function of the wall deflection, mortar compression strength and the rigidity of the supports.

6-9.1. Rigid Supports

If the supports are completely rigid and the mortar's strength is known, a resistance function can be constructed in the following manner.

Both supports are assumed to be completely rigid and lateral motion of the top and bottom of the wall is prevented. An incompletely filled joint is assumed to exist at the top as shown in Figure 6-9a. Under the action of the blast load the wall is assumed to crack at the center. Each half then rotates as a rigid body until the wall takes the position shown in Figure 6-9b. During the rotation the midpoint m has undergone a lateral motion X_c in which no resistance to motion will be developed in the wall, and the upper corner of the wall (point o) will be just touching the upper support. The magnitude of X_c can be found from the geometry of the wall in its deflected position:

$$\left[T - X_c\right]^2 = L^2 - \left[h/2 + (h' - h)/2\right]^2 \qquad 6\text{-}8$$

$$= L^2 - (h'/2)^2$$

where:

$$X_c = T - \left[L^2 - (h'/2)^2\right]^{1/2} \qquad 6\text{-}9$$

and

$$L = \left[(h/2)^2 + T^2\right]^{1/2} \qquad 6\text{-}10$$

All other symbols are shown in Figure 6-9.

For any further lateral motion of point m, compressive forces will occur at points m and o. These compressive forces form a couple that produces a resistance to the lateral load equal to:

$$r_u = 8M_u / h^2 \qquad 6\text{-}11$$

where all symbols have previously been defined. When point m deflects laterally to a line n-o (Figure 6-9c), the moment arm of the resisting couple will be reduced to zero and the wall will become unstable with no further resistance to deflection. In this position the diagonals o-m and m-n will be shortened by an amount:

$$L - h'/2 \qquad 6\text{-}12$$

The unit strain in the wall caused by the shortening will be:

$$\epsilon_m = (L - h'/2)/L \qquad 6\text{-}13$$

where:

ϵ_m = unit strain in the mortar

All the shortening is assumed to occur in the mortar joints and therefore:

$$f_m = E_n \epsilon_m \qquad 6\text{-}14$$

where:

E_n = modulus of elasticity of the mortar

f_m = compressive stress corresponding to the strain ϵ

In most cases f_m will be greater than the ultimate compressive strength of the mortar f_m, and therefore cannot exist. Since for walls of normal height and thickness each half of the wall undergoes a small rotation to obtain the position shown in Figure 6-9c, the shortening of the diagonals o-m and m-n can be considered a linear function of the lateral displacement of point m. The deflection at maximum resistance X_1, at which a compressive stress f_m exists at points m, n and o can therefore be found from the following:

$$\frac{X_1 - X_c}{T - X_c} = \frac{f'_m}{f_m} = \frac{f'_m}{E_n \epsilon_m} \qquad \text{6-15a}$$

or

$$X_1 = \frac{(T - X_c)\, f'_m}{(E_n \epsilon_m)} + X_c \qquad \text{6-15b}$$

The resisting moment that is caused by a lateral deflection X_1 is found by assuming rectangular compression stress blocks to exist at the supports (points o and n) and at the center (point m) as shown in Figure 6-10a. The bearing width a is chosen so that the moment M_u is a maximum, that is, by differentiating M_u with respect to a and setting the derivative equal to zero, which for a solid masonry unit will result in:

$$a = 0.5\,(T - X_1) \qquad \text{6-16}$$

and the corresponding ultimate moment and resistance (Figure 6-10b) are equal to:

$$M_u = 0.25\, f'_m \left[T - X_1\right]^2 \qquad \text{6-17}$$

and

$$r_u = (2/h^2)\, f'_m \left[T - X_1\right]^2 \qquad \text{6-18}$$

When the mid span deflection is greater than X_1 the expression for the resistance as a function of the displacement is:

$$r = (2/h^2) f'_m (T - X)^2 \qquad \text{6-19}$$

As the deflection increases the resistance is reduced until r is equal to zero and maximum deflection X_m is reached (Figure 6-10b). Similar expressions can be derived for hollow masonry units. However, the maximum value of a can not exceed the thickness of the flange width.

6-9.2. Non-rigid Supports

For the case where the wall is supported by elastic supports at the top and/or bottom, the resistance curve cannot be constructed based on the value of the compression force (af'_m) which is determined solely on geometry of the wall.

Instead the resistance curve is a function of the stiffness of the supports. Once the magnitude of the compression force is determined, equations similar to those derived for the case of the rigid supports can be used.

6-9.3. Simply Supported Walls

If the supports offer no resistance to vertical motion, the compression in the wall will be limited by the wall weight above the floor plus any roof load which may be carried by the wall. If the wall carries no vertical loads, then the wall must be analyzed as a simply supported beam, the maximum resisting moment being determined by the modulus of rupture of the mortar.

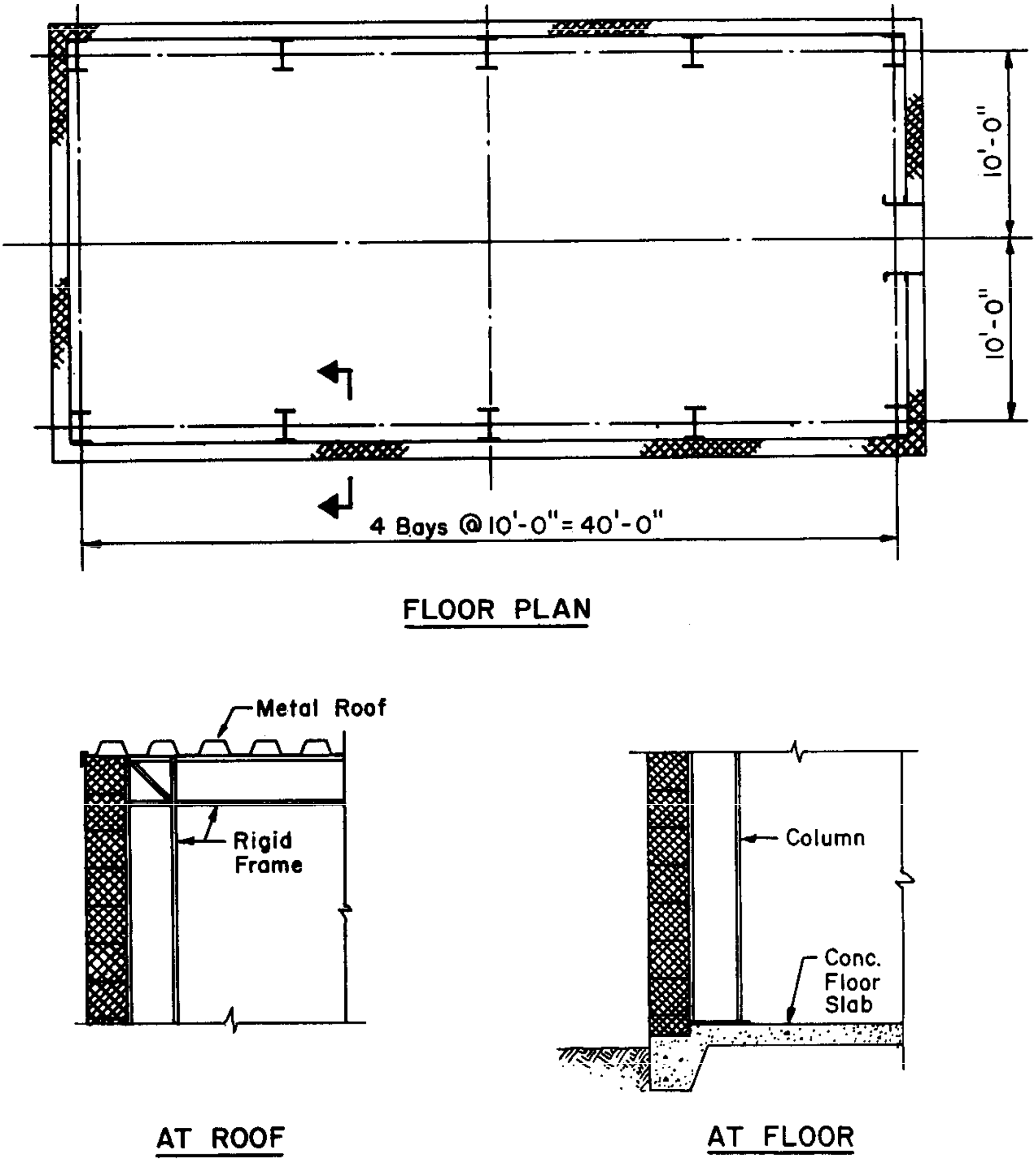


Figure 6-1 Masonry wall with flexible support

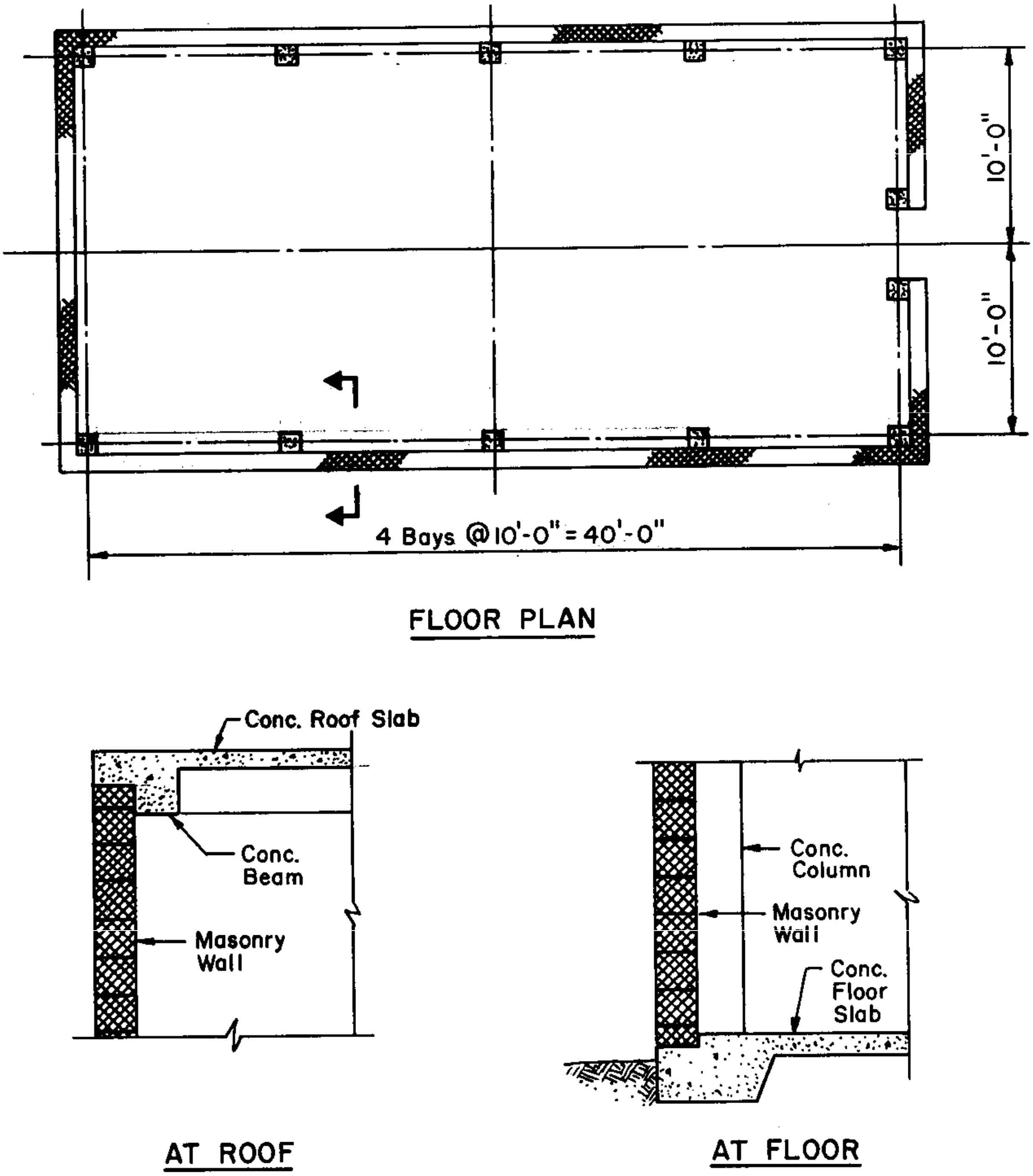


Figure 6-2   Masonry wall with rigid support

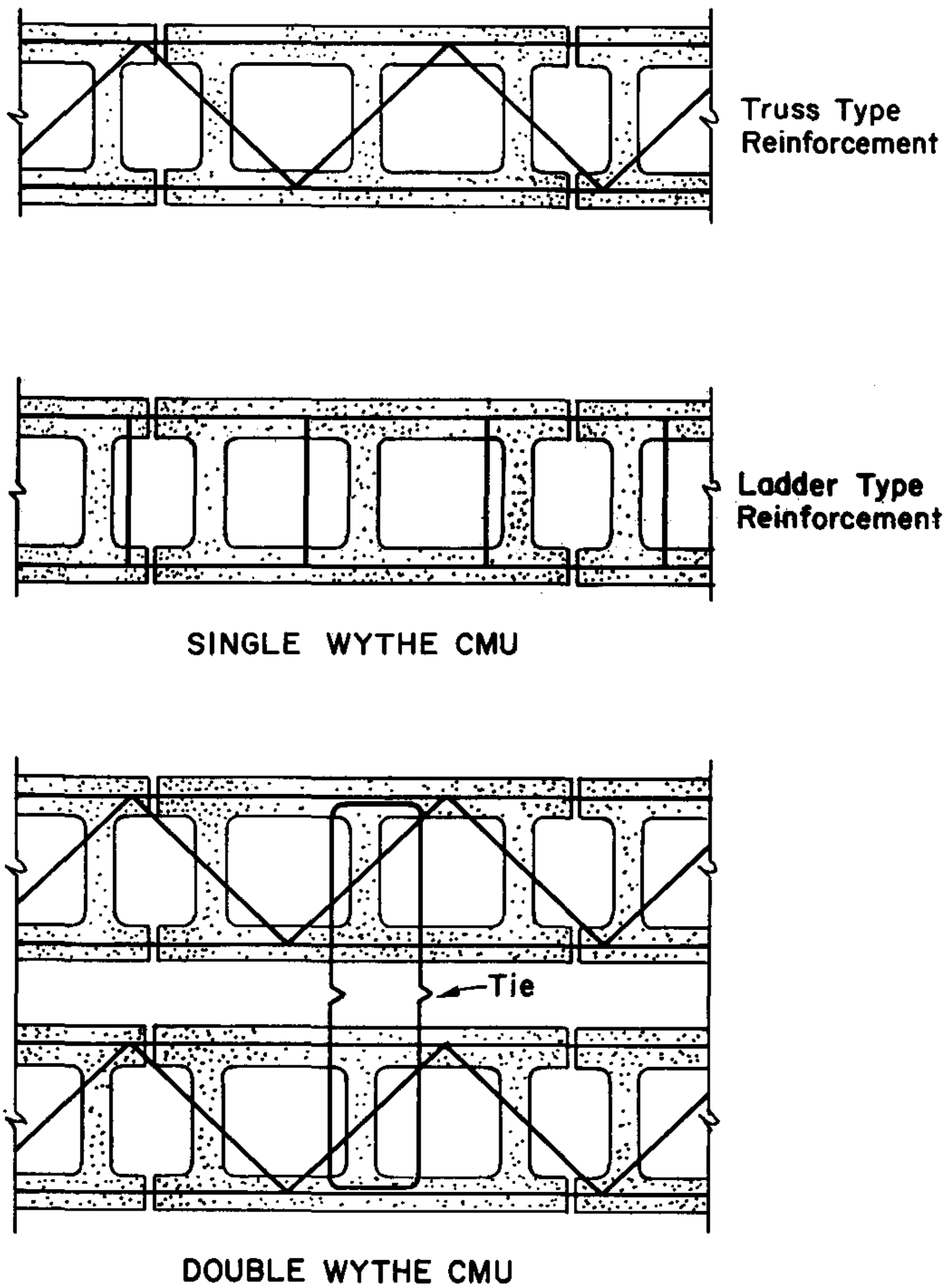
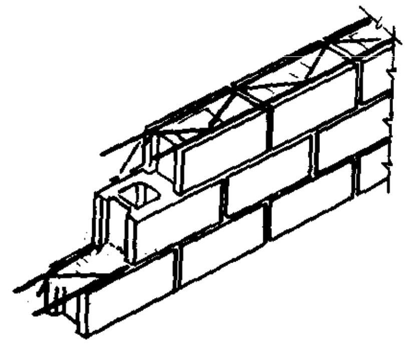
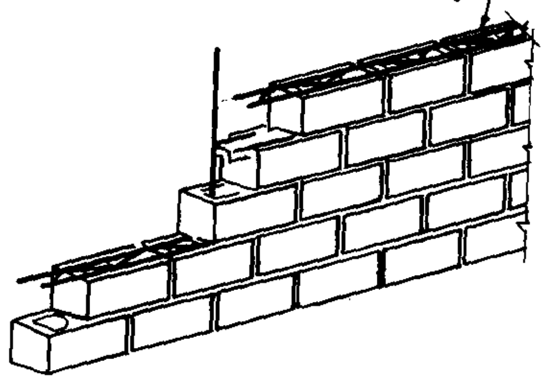


Figure 6-3 Concrete masonry walls



JOINT REINFORCED MASONRY CONSTRUCTION



COMBINED JOINT AND CELL REINFORCED MASONRY CONSTRUCTION

Figure 6-4 Typical joint reinforced masonry construction

THE SPECIAL PROVISIONS FOR REINFORCEMENT PLACEMENT AS SHOWN, ARE AVAILABLE IN MANY OF THE BLOCK CONFIGURATIONS ILLUSTRATED IN FIG. 6-7.

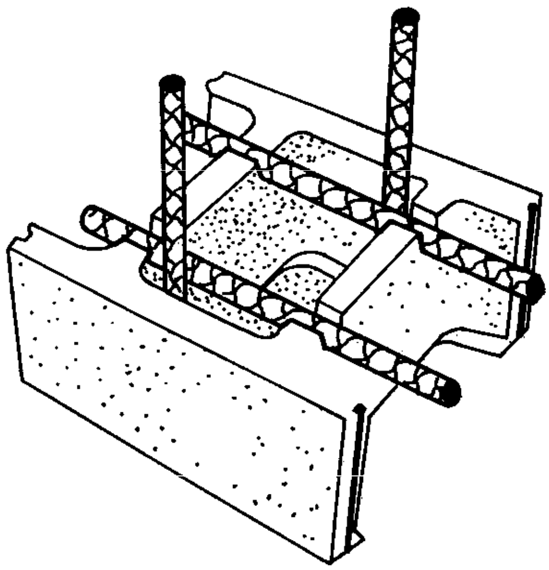


Figure 6-5 Special masonry unit for use with reinforcing bars

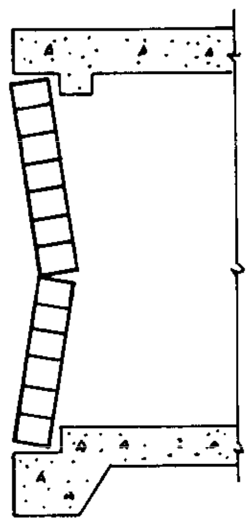


Figure 6-6 Arching action of non-reinforced masonry wall

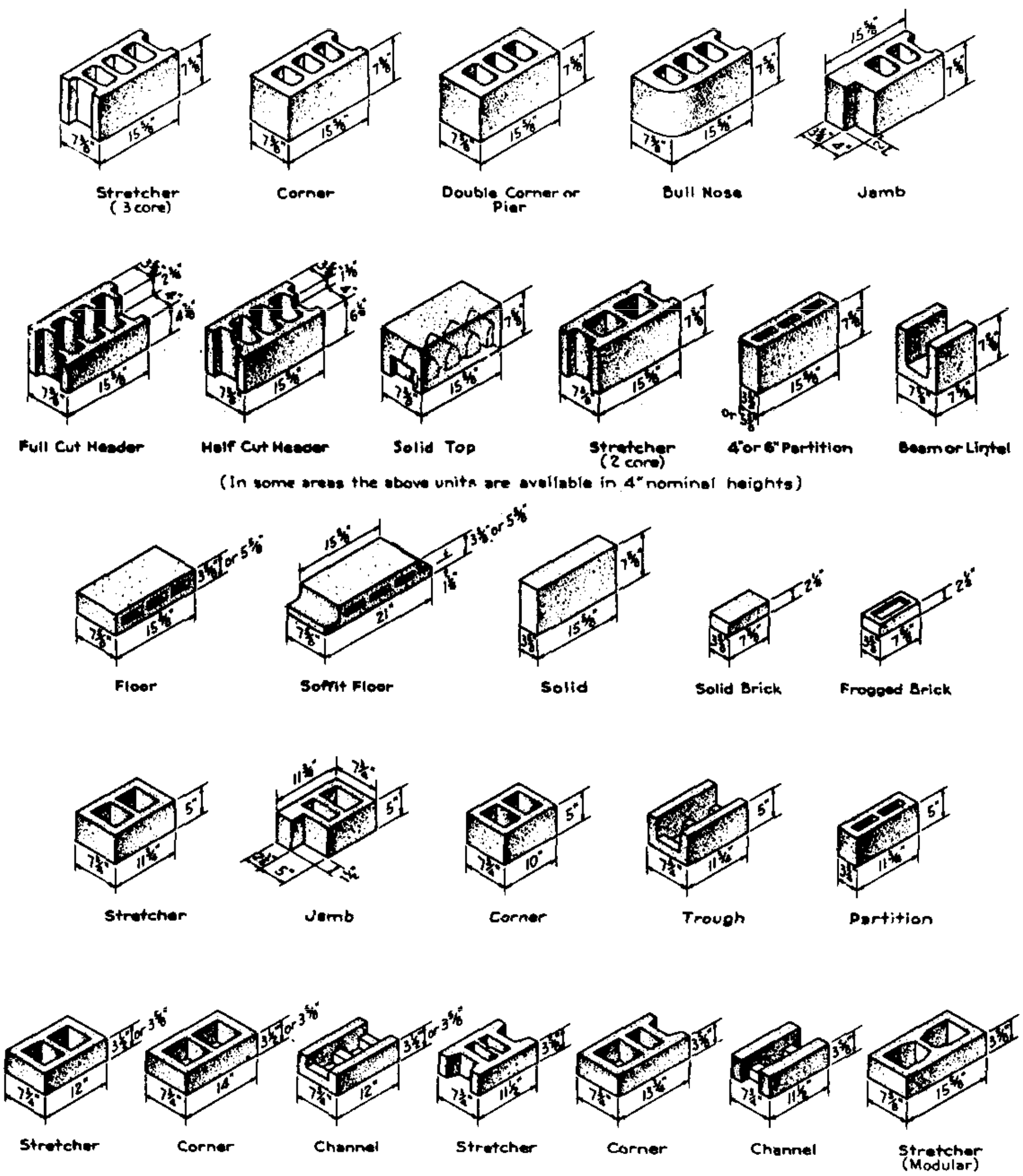
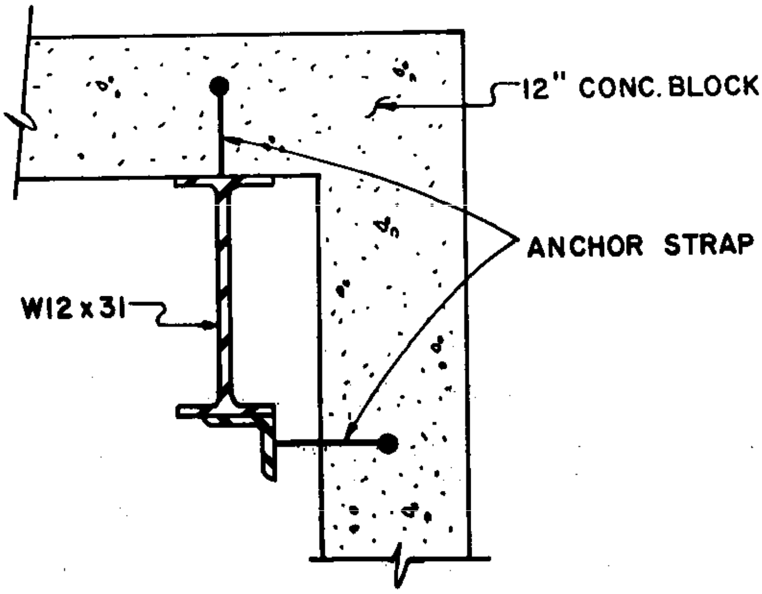
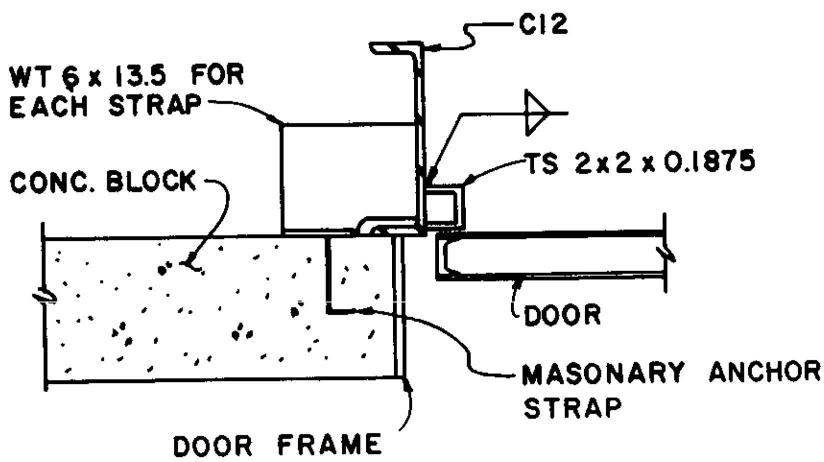


Figure 6-7 Typical concrete masonry units



(a) Masonry Anchor Straps at Corners



(b) Masonry Anchor Strap Detail at Door.

Figure 6-8 Connection details for rebound and/or negative overpressures

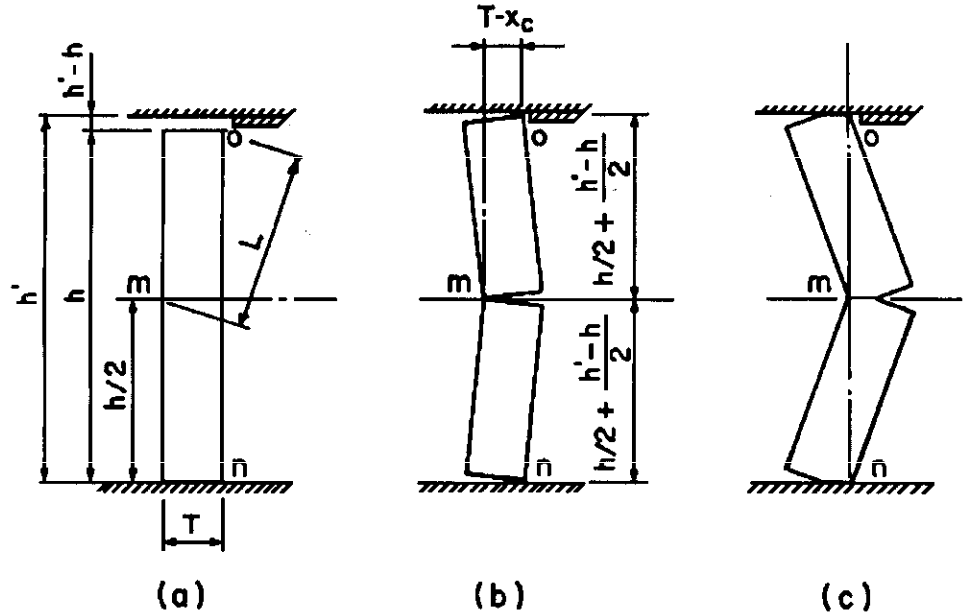
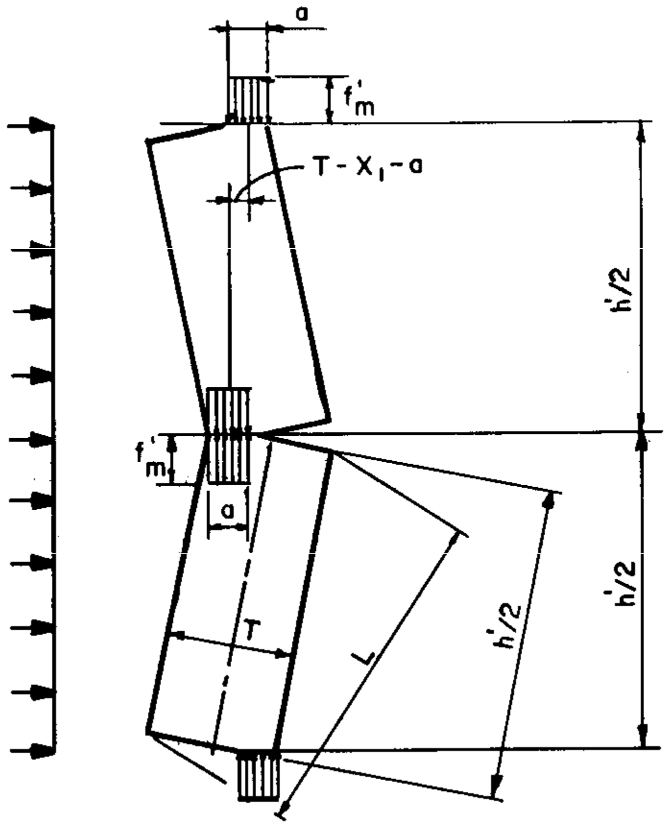
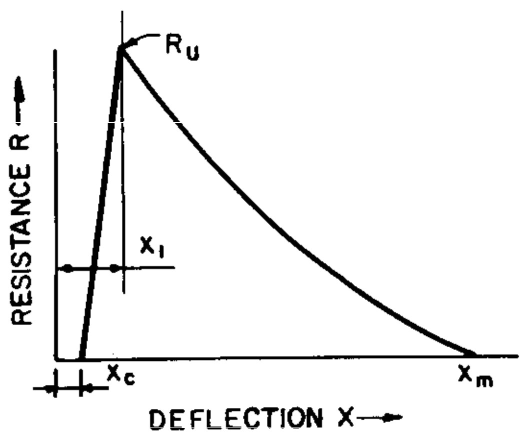


Figure 6-9 Deflection of non-reinforced masonry walls



(a)
ARCHING BEHAVIOR



(b)
RESISTANCE - DEFLECTION FUNCTION

Figure 6-10 Structural behavior of non-reinforced solid masonry panel with rigid supports

Table 6-1 **Properties of Hollow Masonry Units**

Nominal Width of Units (in)	Face-Shell Thickness (in)	Equiv. Web Thickness (in)
3 and 4	0.75	1.625
6	1.00	2.25
8	1.25	2.25
10	1.375	2.50
12	1.500	2.50

Table 6-2 **Deflection Criteria for Masonry Walls**

Wall Type	Support Type	Support Rotation
Reusable	One-way	0.5°
	Two-way	0.5°
Non-Reusable	One-way	1.0°
	Two-way	2.0°

Table 6-3 **Moment of Inertia of Masonry Walls**

Type of Unit	Width of Unit (in)	Moment of Inertia (in^4)
Hollow	3	2.0
	4	4.0
	6	12.7
	8	28.8
	10	51.6
	12	83.3
Solid	3	2.7
	4	5.3
	6	18.0
	8	42.7
	10	83.0
	12	144.0

PRECAST CONCRETE

6-10. Applications

Precast concrete construction can consist of either prestressed or conventionally reinforced members. Prestressing is advantageous in conventional construction, for members subjected to high flexural stresses such as long span or heavily loaded slabs and beams. Other advantages of precast concrete construction include: (1) completion time for precast construction will be significantly less than the required for cast-in-place concrete, (2) precast construction will provide protection against primary and secondary fragments not usually afforded by steel construction and (3) precast work is generally more economical than cast-in-place concrete construction especially when standard precast shapes can be used. The overriding disadvantage of precast construction is that the use of precast members is limited to buildings located at relatively low pressure levels of 1 to 2 psi. For slightly higher pressure levels, cast-in-place concrete or structural steel construction becomes the more economical means of construction. However, for even higher pressures, cast-in-place concrete is the only means available to economically withstand the applied load.

Precast structures are of the shear wall type, rigid frame structures being economically impractical (see the discussion of connections, Section 6-16 below). Conventionally designed precast structures may be multi-story, but for blast design it is recommended that they be limited to single story buildings. Some of the most common precast sections are shown in Figure 6-11. The single tee and double tee sections are used for wall panels and roof panels. All the other sections are beam and girder elements. In addition, a modified flat slab section will be used as a wall panel around door openings. All of the sections shown can be prestressed or conventionally reinforced. In general though, for blast design, beams and roof panels are prestressed, while columns and wall panels are not. For conventional design, prestressing wall panels and columns is advantageous in tall multi-story building, and thus of no benefit for blast resistant design which uses only single story buildings. In fact, in the design of a wall panel, the blast load is from the opposite direction of conventional loads and hence prestressing a wall panel decreases rather than increases the capacity of section.

6-11. Static Strength of Materials

6-11.1. Concrete

Generally the minimum compressive strength of the concrete, f_c, used in precast elements is 4000 to 5000 psi. High early-strength cement is usually used in prestressed elements to ensure adequate concrete strength is developed before the prestress is introduced.

6-11.2. Reinforcing Bars

Steel reinforcing bars are used for rebound and shear reinforcement in prestressed members as well as for flexural reinforcement in non-prestressed members. For use in blast design, bars designated by the American Society for Testing and Materials (ASTM) as A 615, grade 60, are recommended. As only small deflections are permitted in precast members, the reinforcement is not stressed into its strain hardening region and thus the static design strength of the reinforcement is equal to its yield stress (f_m = 60,000 psi).

6-11.3. Welded Wire Fabric

Welded wire fabric, designated as A 185 by ASTM, is used to reinforce the flanges of tee and double tee sections. In conventional design welded wire fabric is sometimes used as shear reinforcement, but it is not used for blast design, which requires closed ties. The static design strength f_m, of welded wire fabric is equal to its yield stress, 65,000 psi.

6-11.4. Prestressing Tendons

There are several types of reinforcement that can be used in prestressing tendons. They are designated by ASTM as A 416, A 421 or A 722, with A 416, grade 250 or grade 270, being the most common. The high strength steel used in these types of reinforcement can only undergo a maximum elongation of 3.5 to 4 percent of the original length before the ultimate strength is reached. Furthermore, the high strength steel lacks a well defined yield point, but rather exhibits a slow continuous yielding with a curved stress-strain relationship until ultimate strength is developed (see Figure 6-12). ASTM specifies a fictitious yield stress f_{py}, corresponding to a 1 percent elongation. The minimum value of f_{py} depends on the ASTM designation, but it ranges from 80 to 90 percent of the ultimate strength, f_{pu}.

6-12. Dynamic Strength of Materials

Under the rapid rate of straining of blast loads, most materials develop higher strengths than they do when statically loaded. An exception, is the high strength steel used in prestressing tendons. Researchers have found that there was very little increase in the upper yield stress and ultimate tensile strengths of high strength steels under dynamic loading.

The dynamic design strength is obtained by multiplying the static design strength by the appropriate dynamic increase factor DIF, which is as follows:

(a)	Concrete:	Compression	DIF = 1.19
		Diagonal tension	DIF = 1.00
		Direct shear	DIF = 1.10
		Bond	DIF = 1.00
(b)	Non-prestressed Steel Reinforcement;		
		Flexure	DIF = 1.17
		Shear	DIF = 1.00
(c)	Welded Wire Fabric:		DIF = 1.10
(d)	Prestressed Reinforcement		DIF = 1.00

6-13. Ultimate Strength of Precast Elements

The ultimate strength of non-prestressed precast members is exactly the same as cast-in-place concrete members and as such is not repeated here. For the ultimate strength of non-prestressed precast elements, see Chapter 4 of this manual.

6-13.1 Ultimate Dynamic Moment Capacity of Prestressed Beams

The ultimate dynamic moment capacity M_u of a prestressed rectangular beam (or of a flanged section where the thickness of the compression flange is greater than or equal to the depth of the equivalent rectangular stress block, a) is as follows:

$$M_u = A_{ps} f_{ps} (d_p - a/2) + A_s f_{dy} (d-a/2) \qquad 6\text{-}20$$

and

$$a = \frac{(A_{ps} f_{ps} + A_s f_{dy})}{.85 f'_{dc}\, b} \qquad 6\text{-}21$$

where:

M_u = ultimate moment capacity

A_{ps} = total area of prestress reinforcement

f_{ps} = average stress in the prestressed reinforcement at ultimate load

d_p = distance from extreme compression fiber to the centroid of the prestressed reinforcement

a = depth of equivalent rectangular stress block

A_s = total area of non-prestressed tension reinforcement

f_{dy} = dynamic design strength of non-prestressed reinforcement

d = distance from extreme compression fiber to the centroid of the non-prestressed reinforcement

f'_{dc} = dynamic compressive strength of concrete

b = width of the beam for a rectangular section or width of the compression flange for a flanged section

The average stress in the prestressed reinforcement at ultimate load f_{ps} must be determined from a trial-and-error stress-strain compatibility analysis. This may be tedious and difficult especially if the specific stress-strain curve of the steel being used is unavailable. In lieu of such a detailed analysis, the following equations may be used to obtain an appropriate value of f_{ps}:

For members with bonded prestressing tendons:

$$f_{ps} = f_{pu}\left[1 - \frac{\gamma_p}{\beta_1}\right]\left[p_p \frac{f_{pu}}{f'_{dc}} + \frac{df_{dy}}{d_p f'_{dc}}(p-p')\right] \qquad 6\text{-}22$$

and

$$p_p = A_{ps} / bd_p \qquad 6\text{-}23$$

$$p = A_s / bd \qquad 6\text{-}24$$

$$p' = A_s' / bd \qquad 6\text{-}25$$

where:

f_{pu} = specified tensile strength of prestressing tendon

γ_p = factor for type of prestressing tendon

= 0.40 for $f_{py} / f_{pu} \geq 0.80$

= 0.28 for $f_{py} / f_{pu} \geq 0.90$

f_{py} = "fictitious" yield stress of prestressing tendon corresponding to a 1 percent elongation

β_1 = 0.85 for f'_{dc} up to 4000 psi and is reduced 0.05 for each 1000 psi in excess of 4000 psi

p_p = prestressed reinforcement ratio

p = ratio of non-prestressed tension reinforcement

p' = ratio of compression reinforcement

A_s = total area of compression reinforcement

If any compression reinforcement is taken into account when calculating f_{ps} then the distance from the extreme compression fiber to the centroid of the compression reinforcement must be less than $0.15d_p$ and

$$p_p \frac{f_{pu}}{f'_{dc}} + \frac{df_{dy}}{d_p f'_{dc}}(p-p') \geq 0.17 \qquad 6\text{-}26$$

If there is no compression reinforcement and no non-prestressed tension reinforcement, Equation 6-22 becomes:

$$f_{ps} = f_{pu}\left[1 - \frac{\gamma_p}{\beta_1} p_p \frac{f_{pu}}{f'_{dc}}\right] \qquad 6\text{-}27$$

For members with unbonded prestressing tendons and a span-to-depth ratio less than or equal to 35:

$$f_{ps} = f_{se} + 10{,}000 + f'_{dc}/(100 p_p) \le f_{py} \qquad 6\text{-}28a$$

and

$$f_{ps} \le f_{se} + 60{,}000 \qquad 6\text{-}28b$$

where:

f_{se} = effective stress in prestressed reinforcement after allowances for all prestress losses

For members with unbonded prestressing tendons and a span-to-depth ratio greater than 35:

$$f_{ps} = f_{se} + 10{,}000 + f'_{dc}/(300\, p_p) \le f_{py} \qquad 6\text{-}29a$$

and

$$f_{ps} \le f_{se} + 30{,}000 \qquad 6\text{-}29b$$

To insure against sudden compression failure the reinforcement ratios for a rectangular beam, or for a flanged section where the thickness of the compression flange is greater than or equal to the depth of the equivalent rectangular stress block will be such that:

$$\frac{p_p f_{ps}}{f'_{dc}} + \frac{d f_{dy}}{d_p f'_{dc}} (p - p') \le 0.36\beta_1 \qquad 6\text{-}30$$

When the thickness of the compression flange of a flanged section is less than the depth of the equivalent rectangular stress block, the reinforcement ratios will be such that

$$\frac{p_{pw} f_{ps}}{f'_{dc}} + \frac{d f_{dy}}{d_p f'_{dc}} (p_w - p'_w) \le 0.36\beta_1 \qquad 6\text{-}31$$

p_{pw}, p_w, p'_w = reinforcement ratios for flanged sections computed as for p_p, p and p' respectively except that b shall be the width of the web and the reinforcement area will be that required to develop the compressive strength of the web only.

6-13.2. Diagonal Tension and Direct Shear of Prestressed Elements

Under conventional service loads, prestressed elements remain almost entirely in compression, and hence are permitted a higher concrete shear stress than non-prestressed elements. However at ultimate loads the effect of prestress is lost and thus no increase in shear capacity is permitted. The shear capacity of a precast beam may be calculated using the equations of Chapter 4 of this manual. The loss of the effect of prestress also means that d is the actual distance to the prestressing tendon and is not limited to 0.8h as it is in the ACI code. It is obvious then that at the supports of an element with draped tendons, d and thus the shear capacity are greatly reduced. Draped tendons also make it difficult to properly anchor shear reinforcement at the supports, exactly where it is needed most. Thus it is recommended that only straight tendons be used for blast design.

6-14. Dynamic Analysis

The dynamic analysis of precast elements uses the procedures described in Chapter 3 of this manual.

Since precast elements are simply supported, the resistance-deflection curve is a one-step function (see Figure 3-39a). The ultimate unit resistance for various loading conditions is presented in Table 3-1. As precast structures are subject to low blast pressures, the dead load of the structures become significant, and must be taken into account.

The elastic stiffness of simply supported beams with various loading conditions is given in Table 3-7. In determining the stiffness, the effect of cracking is taken into account by using an average moment of inertia I_a, as follows:

$$I_a = (I_g + I_c)/2 \qquad 6\text{-}32$$

where:

I_g = moment of inertia of the gross section

I_c = moment of inertia of the cracked section

For non-prestressed elements, the cracked moment of inertia can be determined from Chapter 4. For prestressed elements the moment of inertia of the cracked section may be approximated by:

$$I_c = n\, A_{ps}\, d_p^{\,2} \left[1 - (p_p)^{1/2} \right] \qquad 6\text{-}33$$

where n is the ratio of the modulus of elasticity of steel to concrete. The load-mass factors, used to convert the mass of the actual system to the equivalent mass, are given in Table 3-12 for prestressed elements the load-mass factor in the elastic range is used. An average of the elastic and plastic range load-mass factors is used in the design of non-prestressed elements.

The equivalent single-degree-of-freedom system is defined in terms of its ultimate resistance r_u, equivalent elastic deflection X_E, and natural period of vibration T_N. The dynamic load is defined by its peak pressure P and duration T. The figures given in Chapter 3 may be used to determine the response of an element in terms of its maximum deflection X_m, and the time to reach maximum deflection t_m.

Recommended maximum deflection criteria for precast elements is as follows:

(1) For prestressed flexural members:
$\theta_{max} \leq 2°$ or $\mu_{max} \leq 1$, whichever governs

(2) For non-prestressed flexural members
$\theta_{max} \leq 2°$ or $\mu_{max} \leq 3$, whichever governs

(3) For compression members
$\mu_{max} \leq 1.3$

where θ_{max} = maximum support ratio

μ_{max} = maximum ductility ratio

6-15. Rebound

Precast elements will vibrate under dynamic loads, causing negative deflections after the maximum deflection has been reached. The negative forces associated with these negative deflections may be predicted using Figure 3-268.

6-15.1. Non-prestressed elements

The design of non-prestressed precast elements for the effects of rebound is the same as for cast-in-place members. See Chapter 4 for a discussion of rebound effects in concrete elements.

6-15.2. Prestressed elements

In prestressed elements, non-prestressed reinforcement must be added to what is the compression zone during the loading phase to carry the tensile forces of the rebound phase. The rebound resistance will be determined from Figure 3-268, but in no case will it be less than one-half of the resistance available to resist the blast load.

The moment capacity of a precast element in rebound is as follows:

$$M_u^- = A_s^- f_{dy} (d^- - a^-/2) \qquad 6\text{-}34$$

where:

M_u^- = ultimate moment capacity in rebound

A_s^- = total area of rebound tension reinforcement

f_{dy} = dynamic design strength of reinforcement

d^- = distance from extreme compression fiber to the centroid of the rebound reinforcement

a^- = depth of the equivalent rectangular stress block

It is important to take into account the compression in the concrete due to prestressing and reduce the strength available for rebound. For a conservative design, it may be assumed that the compression in the concrete due to prestressing is the maximum permitted by the ACI code, i.e. 0.45 f'_c. Thus the concrete strength available for rebound is

$$0.85\, f'_{dc} - 0.45\, f_c' = 0.85 f'_{dc} - 0.45 f'_{dc}/DIF = 0.47 f'_{dc} \qquad 6\text{-}35$$

A more detailed analysis may be performed to determine the actual concrete compression due to prestress. In either case the maximum amount of rebound reinforcement added will be

$$A_s^- \le \left[\frac{(0.85 f'_{dc} - d_1)\beta}{f_{dy}}\right]\left[\frac{(87000 - nf)\, bd^-}{(87000 - nf + f_{dy})}\right] \qquad 6\text{-}36$$

where f is the compression in the concrete due to prestressing and all the other terms have been defined previously. If available concrete strength is assumed to be $0.47 f'_{dc}$, equation 6-36 becomes:

$$A_s^- \le \frac{(0.47 f'_{dc}\beta_1)}{f_{dy}}\left[\frac{(87{,}000 - 0.378 n f'_{dc})\, bd^-}{(87000 - 0.378 n f'_{dc} + f_{dy})}\right] \qquad 6\text{-}37$$

6-16. Connections

6-16.1. General

One of the fundamental differences between a cast-in-place concrete structure and one consisting of precast elements is the nature of connections between members. For precast concrete structures, as in the case of steel structures, connections can be detailed to transmit gravity loads only, gravity and lateral loads, or moments in addition to these loads. In general though, connectors of precast members should be designed so that blast loads are transmitted to supporting members through simple beam action. Moment-resisting connections for blast resistant structures would have to be quite heavy and expensive because of the relatively large rotations, and hence induced stresses, permitted in blast design.

In the design of connections the capacity reduction factor ϕ, for shear and bearing stresses on concrete are as prescribed by ACI code, i.e. 0.85 and 0.7 respectively. No capacity reduction factor is used for moment calculations and no dynamic increase factors are used in determining the capacity of a connector. Capacity of the connection should be at least 10 percent greater than the reaction of the member being connected to account for the brittleness of the connection. In addition the failure mechanism should be controlled by tension or bending stress of the steel, and therefore the pullout strength of the concrete and the strength of the welds should be greater than the steel strength.

The following connections are standard for use in blast design but they are not intended to exclude other connection details. Other details are possible but they must be able to transmit gravity and blast loads, rebound loads and lateral loads without inducing moments.

6-16.2. Column-to-Foundation Connection

The standard PCI column-to-foundation connection may be used for blast design without modification. However anchor bolts must be checked for tension due to rebound in order to prevent concrete pullout.

6-16.3. Roof Slab-to-Girder Connection

Figure 6-13 shows the connection detail of a roof panel (tee section) framing into a ledger beam. The bearing pads transmit gravity loads while preventing the formation of moment couples. The bent plate welded to the plate embedded in the flange of the tee transmits lateral loads but is soft enough to deform when the roof panel tries to rotate. The angle welded to the embedded plate in the web of the tee restricts the panel, through shear action, from lifting off the girder during the rebound loading. The effects of dimensional changes due to creep, shrinkage and relaxation of prestress should be considered in this type of connection.

6-16.4. Wall Panel-to-Roof Slab Connection

The basic concepts employed in the roof slab-to-girder connection apply to the wall panel-to-roof slab connection shown in Figure 6-14. The roof panel instead of bearing on the girder, bears on a corbel cast with the tee section. The angle that transmits lateral loads has been moved from the underside of the flange to the top of the flange to facilitate field welding.

6-16.5. Wall Panel-to-Foundation Connection

The wall panel ln Figure 6-15 is attached to the foundation by means of angles welded to plates cast in both the wall panel and the foundation. It is essential to provide a method of attachment to the foundation that is capable of taking base shear in any direction, and also a method of levelling and aligning the wall panel. Non-shrinking grout is used to fill the gap between the panel and the foundation so as to transmit the loads to the foundation.

6-16.6. Panel Splice

Since precast structures are of the shear wall type, all horizontal blast loads are transferred by diaphragm action, through wall and roof slabs to the foundations. The typical panel splice shown in Figure 6-16 is used for transferring the horizontal loads between panels.

6-16.7. Reinforcement Around Door Openings

A standard double tee section cannot be used around a door opening. Instead a special panel must be fabricated to satisfy the requirements for the door opening. The design of the reinforcement around the door opening and the door frame is discussed in Chapter 4.

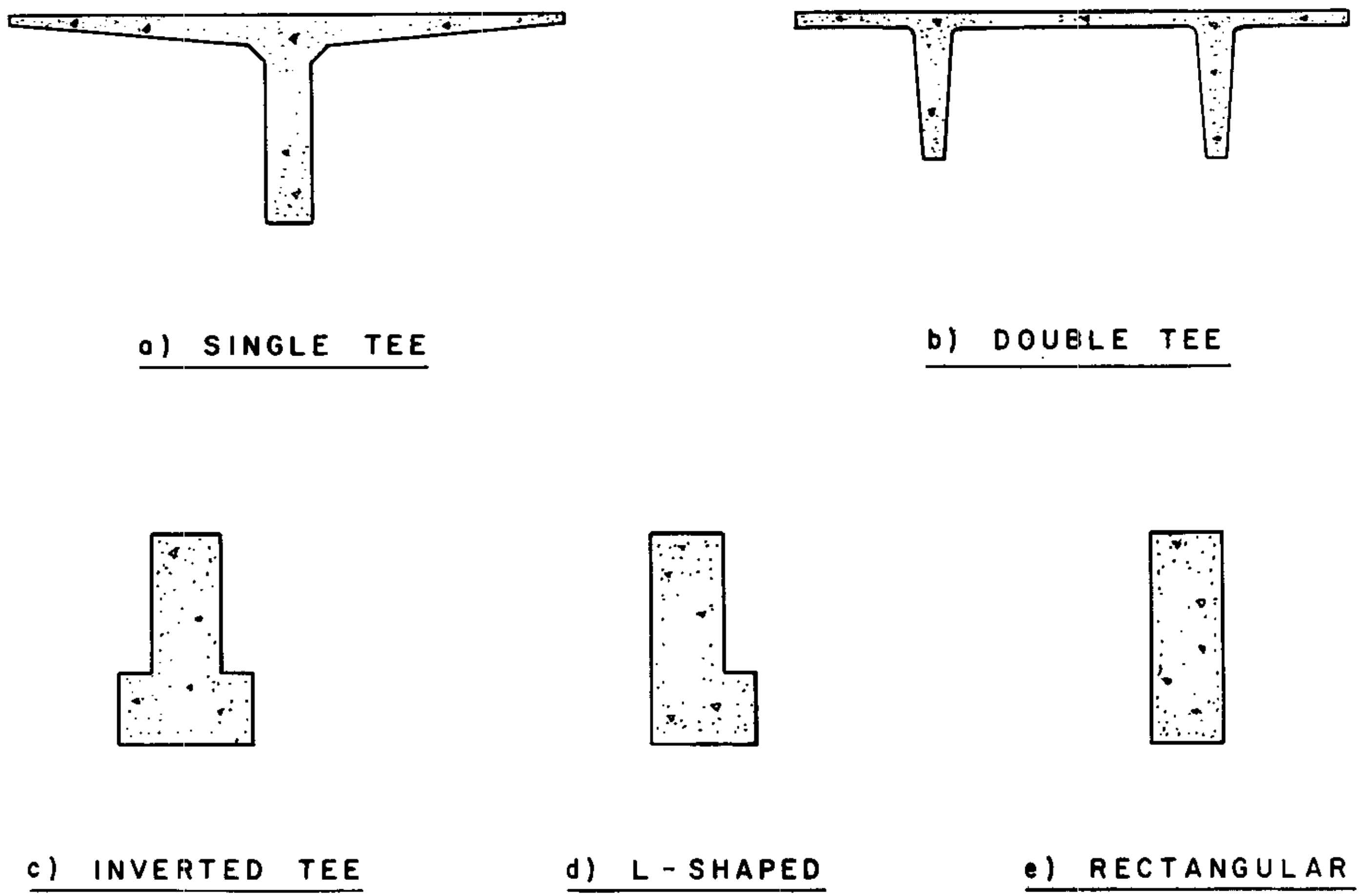


Figure 6-11 Common precast elements

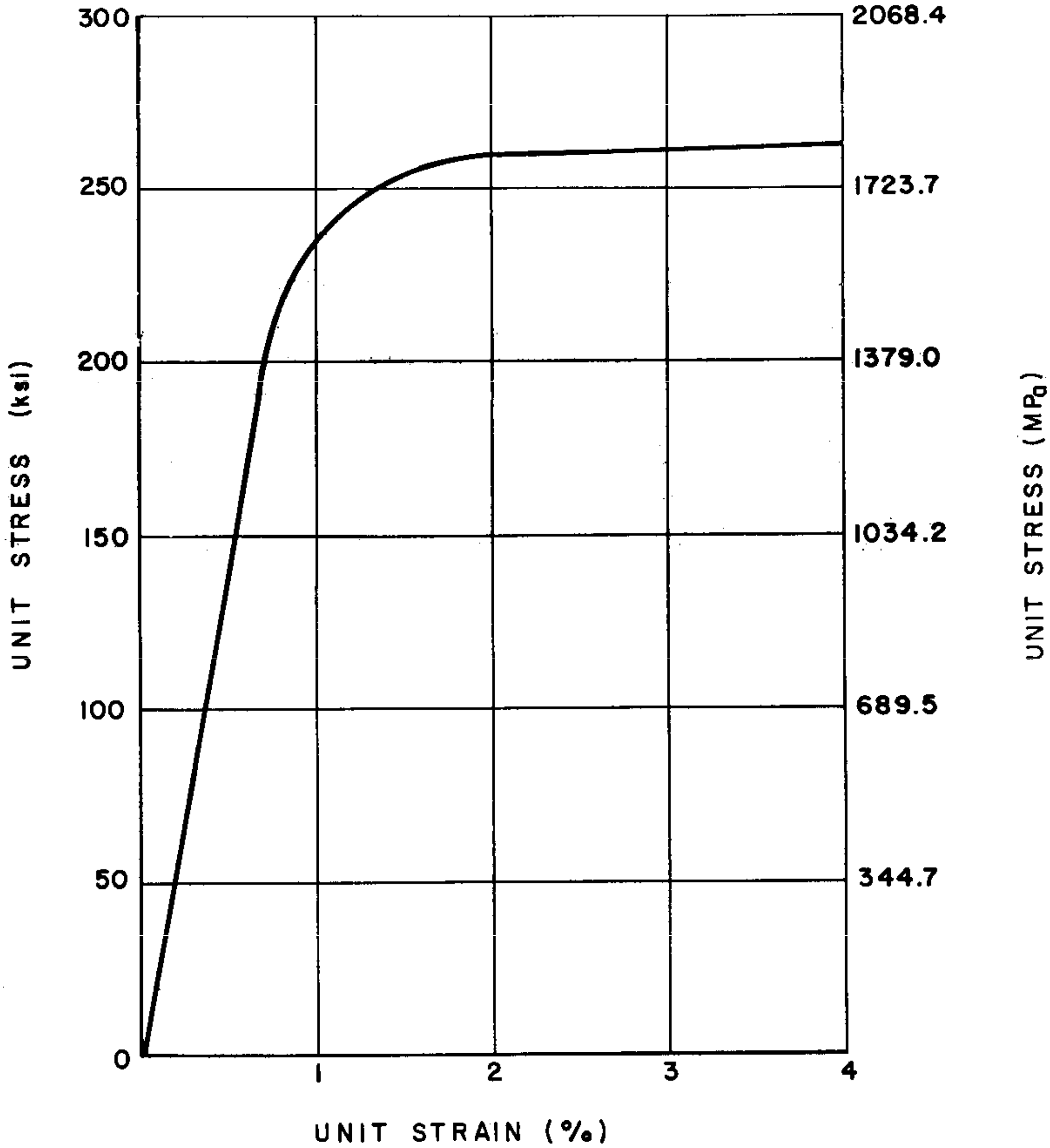


Figure 6-12 Typical stress-strain curve for high strength wire

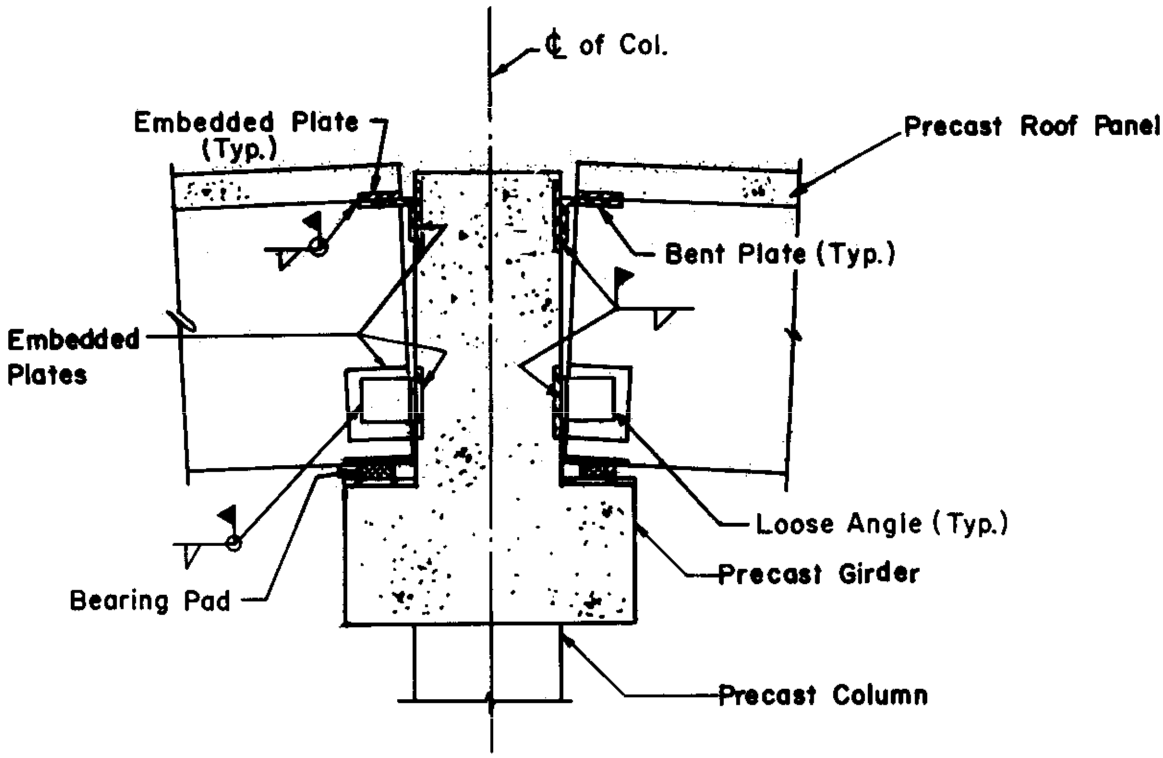


Figure 6-13 Roof slab-to-girder connection

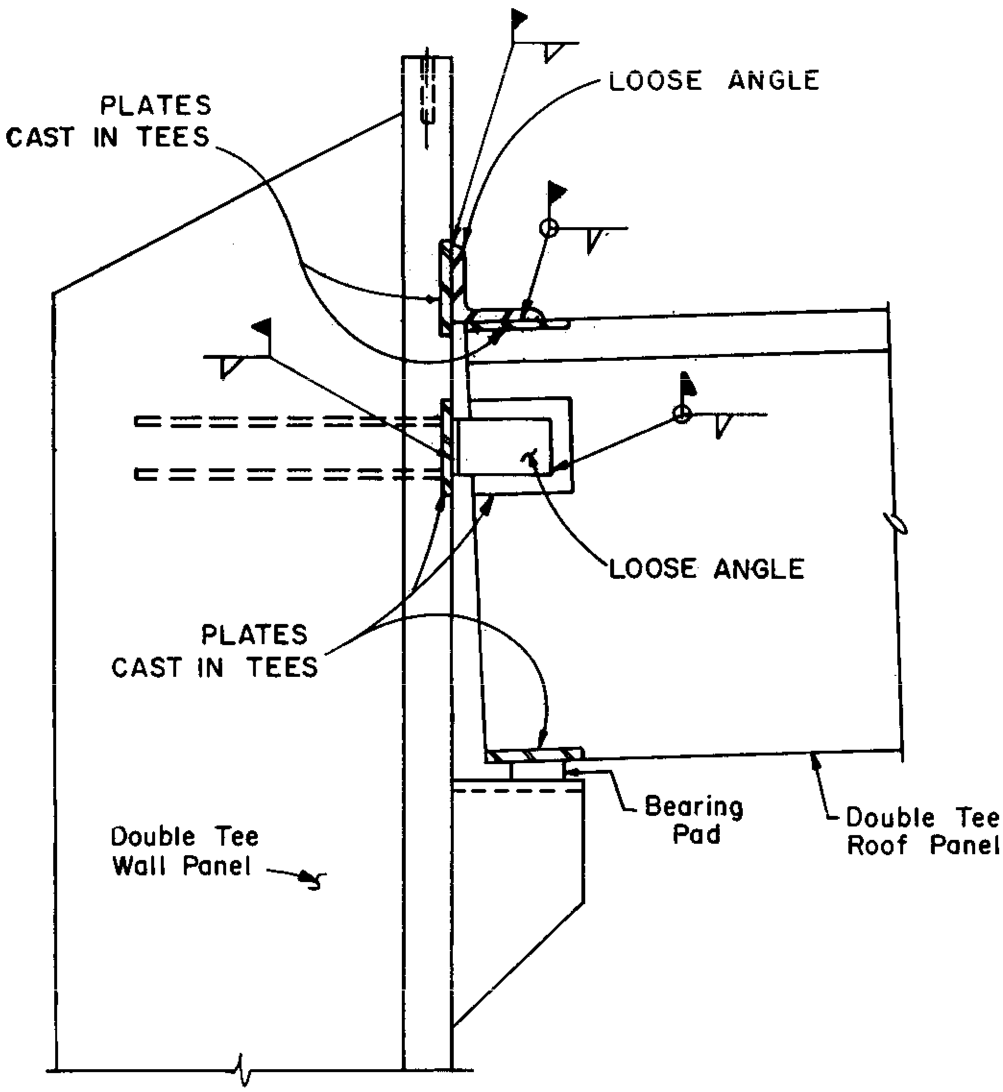


Figure 6-14 Typical wall panel-to-roof slab connection

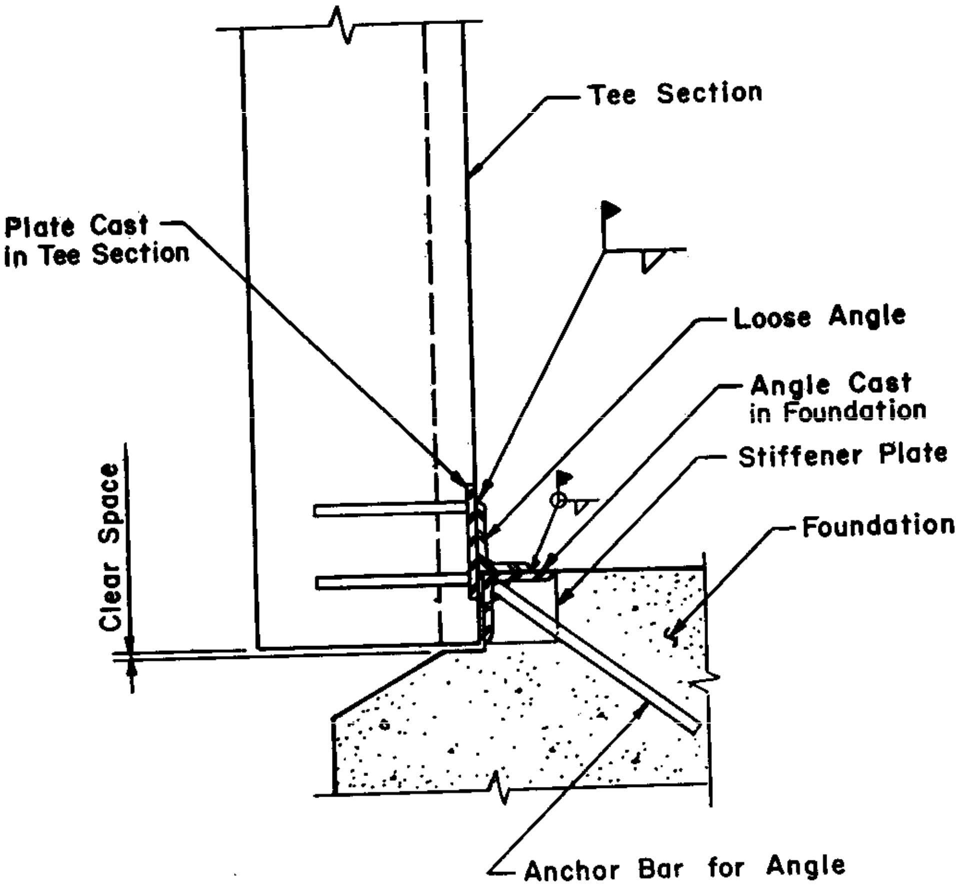


Figure 6-15 Wall panel-to-foundation connection

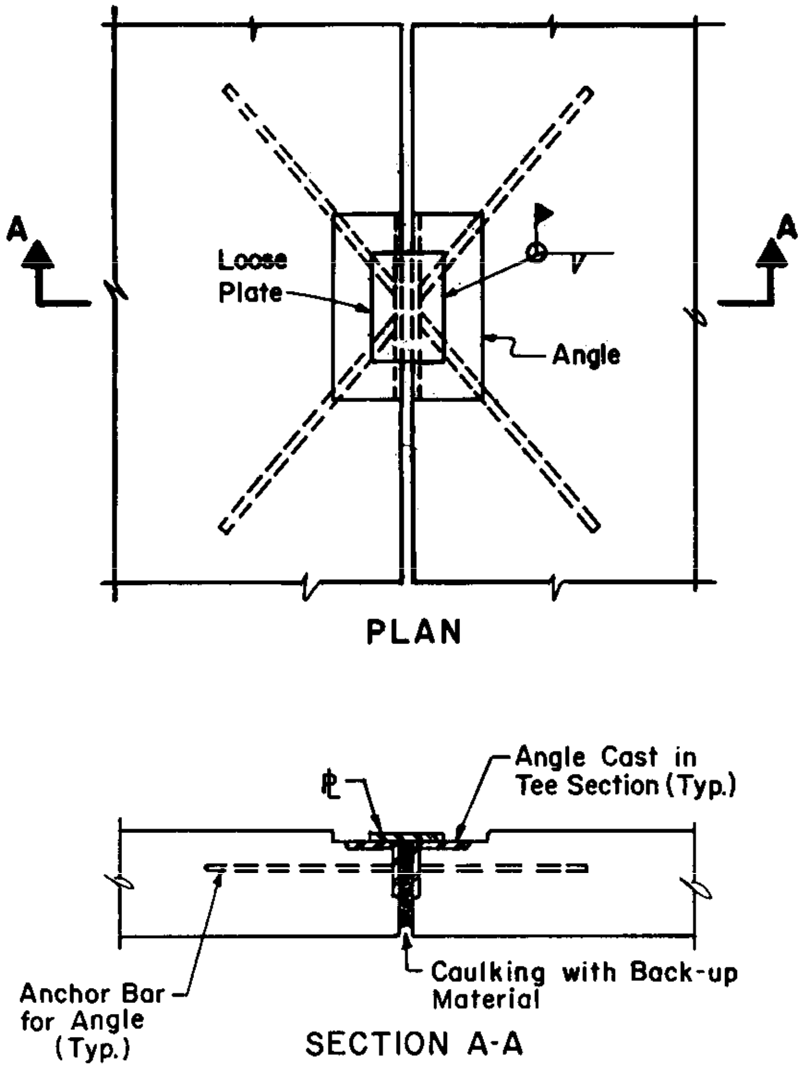


Figure 6-16 Typical panel splice

SPECIAL PROVISIONS FOR PRE-ENGINEERED BUILDING

6-17. General

Standard pre-engineered buildings are usually designed for conventional loads (live, snow, wind and/or seismic). Blast resistant pre-engineered buildings are also designed in the same manner as standard structures. However, the conventional loadings, which are used for the latter designs, are quite large to compensate for effects of blast loads. Further, as with standard buildings, pre-engineered structures, which are designed for blast, are designed elastically for the conventional loadings with the assumption that the structure will sustain plastic deformations due to the blast. The design approach will require a multi-stage process, including: preparation of general layouts and partial blast designs by the design engineer; preparation of the specifications, by the engineer including certain features as recommended herein; design of the building and preparation of shop drawings by the pre-engineered building manufacturers; and the final blast evaluation of the structure by design engineer utilizing the layouts on the previously mentioned shop drawings. At the completion of the analysis some slight modifications in building design may be necessary. However, if the following procedures are used, then the required modifications will be limited and in some cases eliminated for blast overpressures upward to 2 psi.

6-18. General Layout

The general layout of pre-engineered buildings is based on both operational and blast resistant requirements. Figure 6-17 illustrates a typical general layout of the pre-engineered building. The general requirements for structural steel, concrete, wall and roof coverings and connections are given below.

6-18.1. Structural Steel

In order for a pre-engineered building to sustain the required blast loading, structural steel layout must conform to the following requirements:

1. The maximum spacing between main transverse rigid frames (bay width) shall not exceed 20 feet.

2. The maximum spacing between column supports for rigid frames shall not exceed 20 feet while the overall height of frames shall be 30 feet or less.

3. Slope of the roof shall not exceed four horizontal to one vertical. However, the roof slope shall be as shallow as physically possible and be in compliance with the requirements of the Metal Building Manufacturers' Association.

4. Spacing between girts shall not exceed 4 feet while the space between purlins shall not be greater than 5 feet.

5. Primary members, including frames and other main load carrying members, shall consist of hot rolled structural steel shapes. The shapes must be doubly-symmetrical and have a constant depth. They may be wide-flange sections, I-sections, structural tubes, or

welded shapes built-up from hot rolled steel sheet, strips or plates. Secondary structural framing, such as girts, roof purlins, bridging, eave struts and other miscellaneous secondary framing, may consist of either hot rolled or cold-formed structural steel. All main secondary members (purlins, girts, etc.) shall be doubly-symmetrical sections of constant depth (e.g. wide flange, "I"-shaped, structural tubing).

6. Primary structural framing connections shall be either shop welded or bolted or field bolted assemblies. ASTM A 325 bolts with appropriate nuts and washers shall be used for connecting of all primary members; where as secondary members may use bolts conforming to ASTM A 307. A minimum of two bolts shall be used for each connection while bolts for primary and secondary members shall not be less than 3/4 and 1/2-inch in diameter, respectively.

7. Base plates for columns shall be rolled and set on grout bed of 1-inch minimum thickness. ASTM A 307 steel bolts shall be used to anchor all columns.

6-18.2. Foundations

Concrete floor and foundation slabs shall be monolithic in construction and shall be designed to transfer all horizontal and vertical loads from the pre-engineered superstructure to the foundation soil. Minimum slab thickness shall be 6 inches with edge beams thickened to meet local frost conditions.

6-18.3. Roof and Walls

Roof and wall coverings must meet the following requirements:

1. Roof and wall coverings shall conform to ASTM A 446, G 90, have a minimum depth of 1-1/2 inches corrugation and have a material thickness of 22 gauge.

2. Conventional side laps are not usually sufficient to resist the effects of blast loads. The construction details required to strengthen those joints .depend upon the type of decking employed. Chapter 5 gives the required panel-to-panel attachments for various types of decking.

3. Insulation retainers or sub girts shall be designed to transmit all external loads (listed below) which act on the metal cover to the structural steel framing.

4. Roof and wall liners shall be a minimum of 24 gauge and shall be formed to prevent waviness, distortion or failure as a result of the impact by external loads.

6-18.4. Connections for Roof and Wall Coverings

The connections used in a blast resistant structure are especially critical. To ensure full development of structural steel and the roof and wall panels, connections must meet the following criteria:

1. Fasteners for connecting roof and wall coverings to structural steel supports shall be designed to support the external loads (listed below) and shall consist of either self-tapping screws, self-drilling and self-tapping screws, bolts and nuts, self-locking rivets, self-locking bolts, end welded studs, or welds. Fasteners of covering to structural steel shall be located at valleys of the covering and shall have a minimum of one fastener per valley.

2. Fasteners which do not provide positive locking such as self-tapping screws, etc. shall not be used at side laps and for fastening accessories to panels. At least one fastener for side laps shall be located in each valley and at a maximum spacing along the valley of 8 inches.

3. Self tapping screws shall not have a diameter smaller than a no. 14 screw while the minimum diameter of a self-drilling and self-tapping type shall be equal to or greater than a no. 12 screw. Automatic welded studs shall be shouldered type and have a shank diameter of at least 3/16 inch. Fasteners for use with power actuated tools shall have a shank diameter of not less than 1/2 inch. Blind rivets shall be stainless steel type and have a minimum diameter of 1/8 inch. Rivets shall be threaded stem type if used for other than fastening trim and if of the hollow type shall have closed ends. Bolts shall not be less than 1/4 inch in diameter and will be provided with suitable nuts and washers.

   If suction and/or rebound loads dictate, provide oversized washers with a maximum outside diameter of 2 inches or a 22 gauge thick metal strip along each valley.

6-19. Preparation of Partial Blast Analysis

A partial blast analysis of a pre-engineered building shall be performed by the design engineer. This analysis shall include the determination of the minimum size of the roof and wall panels which is included in the design specifications and the design of the building foundation and floor slab. The foundation and floor slab shall be designed monolithically and have a minimum thickness as previously stated. The slab shall be designed for a foundation load equal to either 1.3 times the yield capacity of the building roof equivalent blast load or the static roof and floor loads listed below. Quite often the foundation below the building columns must be thickened to distribute the column loads. For the blast analysis of the building foundation and floor slab, the dynamic capacity of the soil below the foundation slab can conservatively be assumed to be equal to twice the static soil capacity. The resistance of the roof of the building can be determined in accordance with the procedures given in Chapter 5. The front panel of the building is designed in the same manner as the roof panel. The blast loads for determining the capacities of the roof and wall panels can be determined from Chapter 2.

6-20. Pre-Engineered Building Design

Design of the pre-engineered building shall be performed by the pre-engineered building manufacturer using static loads and conventional stresses.

Conventional stresses are listed in "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings with Commentary". Static design loads shall be as follows:

1. Floor live loads shall be as specified in the report titled "American National Standard Building Code Requirements for Minimum Design Loads in Buildings and Other Structures" (hereafter referred to as ANSI) but not less than 150 pounds per square foot.

2. Roof live loads shall be as specified ANSI.

3. Dead loads are based on the materials of construction.

4. Wind pressure shall be as computed in accordance with ANSI for exposure "C" and a wind speed of 100 miles per hour.

5. Seismic loads will be calculated according to the Uniform Building Code for the given area. If this load is greater than the computed wind pressure, than the seismic load will be substituted for wind load in all load combinations.

6. Auxiliary and collateral loads are all design loads not listed above and include suspended ceilings, insulation, electrical systems, mechanical systems, etc.

Combinations of design loads shall include the following (a) dead loads plus live loads; (b) dead loads plus wind loads, and (c) 75 percent of the sum of dead, live and wind loads.

6-21. Blast Evaluation of the Structure

Blast evaluation of the structure utilizing the shop drawings prepared in connection with the above design shall be performed by the design engineer. A dynamic analysis which describes the magnitude and direction of the elasto-plastic stresses developed in the main frames and secondary members as a result of the blast loads, shall be performed using the methods described in Chapter 5. This evaluation should be made at the time of the shop drawing review stage.

6-22. Recommended Specification for Pre-Engineered Buildings

Specifications for pre-engineered buildings shall be consistent with the recommended design changes set forth in the preceding Section. These example specifications are presented using the Construction Specification Institute (CSI) format and shall contain as a minimum the following:

1. APPLICABLE PUBLICATIONS. The following publications of the issues listed below, but referred to thereafter by basic designation only, form a part of this specification to the extent indicated by the reference thereto:

1.1 American Society of Testing and Materials (ASTM)

A 36 Structural Steel

A 307-80 Standard Specification for Carbon Steel Externally and Internally Threaded Standard Fasteners

A 325 Standard Specification for High Strength Bolts for Structural Steel Joint Including Suitable Nuts and Plain Hardened Washers

A 446 Specification for Steel Sheet, Zinc Coated (Galvanized) by the Hot-Dip Process, Physical (Structural) Quantity

A 501 Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing

A 529 Standard Specification for Structural Steel with 42,000 psi Minimum Yield Point

A 570 Standard Specification for Hot-Rolled Carbon Steel Sheet and Strip, Structural Quality

A 572 Specification of High-Strength Low-Allow Columbium-Vanadium Steels of Structural Quality

1.2 American Iron and Steel Institute (AISI)

Specification for the Design of Cold-Formed Steel Structural Members and Commentary

1.3 American National Standards Institute (ANSI)

A58.1 Minimum Design Loads for Buildings and Other Structures

B18.22.1 Plain Washers

1.4 American Institute of Steel Construction (AISC)

Specification for the Design Fabrication and Erection of Structural Steel for Buildings with Commentary Research Council on Riveted and Bolted Structural Joints (RCRBSJ) Specification for Structural Joints Using ASTM A 325 or A 490 Bolts

1.5 American Welding Society (AWS)

D1.1 Structural Welding Code

1.6 Metal Building Manufacturers' Association (MBMA)

Metal Buildings Systems Manual

1.7 Uniform Building Code

2. GENERAL.

2.1 This section covers the manufacture and erection of pre-engineered metal structures. The structure manufacturer shall be regularly engaged in the fabrication of metal structures.

2.2 The structure shall include the rigid framing, which are spaced at a maximum of 20 feet on center, roof and wall covering, trim, closures, and accessories as indicated on the drawings. Minor alterations in dimensions shown on the drawings will be considered .in order to comply with the manufacturer's standards building system, provided that all minimum clearances indicated on the drawings are maintained. Such changes shall be submitted for review and acceptance prior to fabrication.

2.3 Drawings shall indicate extent and general assembly details of the metal roofing and sidings. Members and connections not indicated on the drawings shall be designed.by the Contractor in accordance with the manufacturer's standard details. The Contractor shall comply with the dimensions, profile limitations, gauges and fabrication details shown on the drawings. Modification of details will be permitted only when approved by the Owner. Should the modifications proposed by the Contractor be accepted by the Owner, the Contractor shall be fully responsible for any re-design and re-detailing of the building construction effected.

3. DEFINITIONS.

3.1 Low Rigid Frame. The building shall be single gable type with the roof slope not to exceed one on four.

3.2 Framing.

3.2.1 Primary Structural Framing. The primary structural framing includes the main transverse frames and other primary load carrying members and their fasteners.

3.2.2 Secondary Structural Framing. The secondary structural framing includes the girts, roof purlins, bridging, eave struts, and other miscellaneous secondary framing members and their fasteners.

3.2.3 Roof and Wall Covering. The roof and wall covering includes the exterior ribbed metal panel having a minimum depth of one and one-half inches, neoprene closure, fasteners and sealant.

3.3 Building Geometry.

3.3.1 Roof Slope. The roof of the building shall have a maximum slope not to exceed one on four.

3.3.2 Bay Spacing. The bay spacing shall not exceed 20 feet.

3.4 Column Shape. Main frame columns shall be doubly symmetrical members of constant depth; tapered columns will not be permitted.

3.5 Calculations. The Contractor shall submit for review complete design calculations for all work, sealed by a registered professional engineer.

4. STRUCTURAL DESIGN.

4.1 Structural Analysis. The structural analysis of the primary and secondary framing and covering shall be based on linear elastic behavior and shall accurately reflect the final configuration of the structure and all tributary design loadings.

4.2 Basic Design Loads.

4.2.1 Roof Live Load. Shall be applied to the horizontal roof projection. Roof live loads shall be:

0 to 200 square feet tributary area - 20 psf

200 to 600 square feet tributary area - linear variation 20 psf to 12 psf

over 600 square feet tributary area - 12 psf

4.2.2 Wind Pressure. Wind design loads shall be computed in accordance with ANSI A58.1 for exposure "C" and a basic wind speed of 100 miles per hour.

4.2.2.1 Typical Wind Loading. As shown on drawings (Figure 6-18).

4.2.2.2 Wind Loading at Building Corners. As shown on the drawings (Figure 6-18).

4.2.2.3 Wind Loading on Girts. As shown on drawings (Figure 6-18).

4.2.2.4 Wind Loading on Purlins and Roof Tributary Areas. As shown on drawings (Figure 6-18).

4.2.2.5 Wind Loading for Design of Overall Structure. As shown on drawings (Figure 6-18).

4.2.3 Auxiliary and Collateral Design Loads. Auxiliary and collateral design loads are those loads other than the basic design live, dead, and wind loads; which the building shall safely withstand, such as ceilings, insulation, electrical, mechanical, and plumbing systems, and building equipment and supports.

4.3 Application of Design Loads.

4.3.1 Roof Live Load and Dead Load. The roof live load (L), and dead load (D), shall be considered as a uniformly distributed loading acting vertically on the horizontal projection of the roof.

4.3.2 Snow Loads. application of 30 psf due to snow loads.

4.3.3 Wind Loads (W). Application of forces due to wind shall conform to the latest ANSI A58.1

4.3.4 Combination of Loads. The following combinations of loads shall be considered in the design of all members of the structure:

D + L
D + W
.75 (D + L + W)

4.4 Deflection Limitations.

4.4.1 Structural Framing. The primary and secondary framing members shall be so proportioned that their maximum calculated roof live load deflection does not exceed 1/120 of the span;

5. STRUCTURAL FRAMING.

5.1 General

5.1.1 All hot rolled structural shapes and structural tubing shall have a minimum yield point of 36,000 psi in conformance with ASTM A 36 or A 501. All hot rolled steel plate, strip and sheet used in the fabrication of welded assemblies shall conform to the requirements of ASTM A 529, A 572, Grade 42 or A 570 Grade "E" as applicable. All hot rolled sheet and strip used in the fabrication of cold-formed members shall conform to the requirements of ASTM A 570, Grade"E" having a minimum yield strength of 50,000 psi. Design of cold-formed members shall be in accordance with the AISI specifications.

5.1.2 The minimum thickness of framing members shall be:

Cold-formed secondary framing members	- 18 gauge
Pipe or tube columns	- 12 gauge
Webs of welded built-up members	- 1/8 inch
Flanges of welded built-up members	- 1/4 inch
Bracing rods	- 1/4 inch

5.1.3 All framing members shall be fabricated for bolted field assembly. Bolt holes shall be punched or drilled only. No burning-in of holes will be allowed. The faying surfaces of all bolted connections shall be smooth and free from burrs or distortions. Provide washers under head and nut of all bolts. Provide beveled washers to match sloping surfaces as required. Bolts shall be of type specified below. Members shall be straight and dimensionally accurate.

5.1.4 All welded connections shall be in conformance with the STRUCTURAL WELDING CODE D1.1 of the American Welding Society. The flange-to-web welds shall be one side continuous submerged arc fillet welds. Other welds shall be by the shielded arc process.

5.2 Primary Structural Framing.

5.2.1 The primary members shall be constructed of doubly-symmetrical, hot rolled structural steel shapes or doubly-symmetrical built-up members of constant depth, welded from hot rolled steel sheet, strip or plates.

5.2.2 Compression flanges shall be laterally braced to withstand any combination of loading.

5.2.3 Bracing system shall be provided to adequately transmit all lateral forces on the building to the foundation.

5.2.4 All bolt connections of primary structural framing shall be made using high-strength zinc-plated (0.0003 bronze zinc plated) bolts, nuts, and washers conforming to ASTM A 325. Bolted connections shall have not less than two bolts. Bolts shall not be less than 3/4 inch diameter. Shop welds and field bolting are preferred. All field welds will require prior approval of the Owner. Installation of fasteners shall be by the turn-of-nut or load-indicating washer method in accordance with the specifications for structural joints of the Research Council on Riveted and Bolted Structural Joints.

5.3 Secondary members may be constructed of either hot rolled or cold-formed steel. Purlins and girts shall be doubly symmetrical sections of constant depth and they may be built-up, cold-formed or hot rolled structural shapes.

5.3.1 Maximum spacing of roof purlins and wall girts shall not exceed 5 feet.

5.3.2 Compression flanges of purlins and girts shall be laterally braced to withstand any combination of loading.

5.3.3 Supporting lugs shall be used to connect the purlins and girts to the primary framing. The lugs shall be designed to restrain the light gauge sections from tipping or warping at their supports. Each member shall be connected to each lug by a minimum of two fasteners.

5.3.4 Vertical wall members not subjected to axial load, e.g. vertical members at door openings, shall be constant depth sections. They may consist of hot rolled or cold-form steel. They shall be either built-up, cold-formed or hot rolled "C" or "I" shapes.

5.3.5 Fasteners for all secondary framing shall be a minimum of 1/2 inch diameter (0.003 zinc plated) bolts conforming to ASTM A 307. The fasteners shall be tightened to SNUG TIGHT condition. Plain washers shall conform to ANSI standard B18.22.1.

6. ANCHORAGE.

6.1 Anchorage. The building anchor bolts for both primary and secondary columns shall conform to ASTM A 307 steel and shall be designed to resist the column reactions produced by the specified design loading. The quantity, size and location of anchor bolts shall be specified and furnished by the building manufacturer. A minimum of two anchor bolts shall be used with each column.

6.2 Column Base Plates. Base plates for columns shall conform to ASTM A 36 and shall be set on a grout bed of 1 inch minimum thickness.

7. ROOF AND WALL COVERING.

7.1 Roof and wall panels shall conform to zinc-coated steel, ASTM A 446, G 90 coating designation. Minimum depth of each panel corrugation shall be 1-1/2 inches and shall have a minimum material thickness of 22 gauge. The minimum yield strength of panel material shall be 33,000 psi. Wall panels shall be applied with the longitudinal configurations in the vertical position. Roof panels shall be applied with the longitudinal configuration in direction of the roof slope. Side laps of roof and wall panels shall be fastened as shown on drawings. End laps, if required shall occur at structural steel supports and have a minimum length of 12 inches.

7.2 Insulation.

7.2.1 Semi-rigid insulation for the preformed roofing and siding shall be supplied and installed by the preformed roofing and siding manufacturer.

7.2.2 Insulation Retainers. Insulation retainers or sub girts shall be designed to transmit all external loads (wind, snow and live loads) acting on the metal panels to the structural steel framing. The retainers shall be capable of transmitting both the direct and suction loads.

7.3 Wall and Roof Liners. Wall and roof liners shall be a minimum of 24 gauge. All liners shall be formed or patterned to prevent waviness, distortion or failure as a result of the impact by external loads.

7.4 Fasteners. Fasteners for roof and wall panels shall be zinc-coated steel or corrosion-resisting steel. Exposed fasteners shall be gasketed or have gasketed washers of a material compatible with the covering to waterproof the fastener penetration. Gasketed portion of fasteners or washers shall be neoprene or other elastomeric material approximately 1/8 inch thick.

7.4.1 Type of Fasteners. Fasteners for connecting roof or wall panels to structural steel supports shall consist of self-tapping screws, self-drilling and self-tapping screws, bolts, end welded studs, and welds. Fasteners for panels which connect to structural supports shall be located in each valley of the panel and with a minimum of one fastener per valley while at end laps and plain ends, a minimum of two fasteners shall be used per valley. Fasteners shall not be located at panel crowns.

7.4.2 Fasteners which do not provide positive locking such as self-tapping screws or self-drilling and self-tapping screws shall not be used at side laps of panels and for fastening accessories to panels. Fasteners for side laps shall be located in each valley of the overlap and positioned a maximum of 8 inches on center.

7.4.3 Screws shall be not less than No. 14 diameter if self-tapping type and not less than No. 12 diameter if self-drilling and self-tapping type.

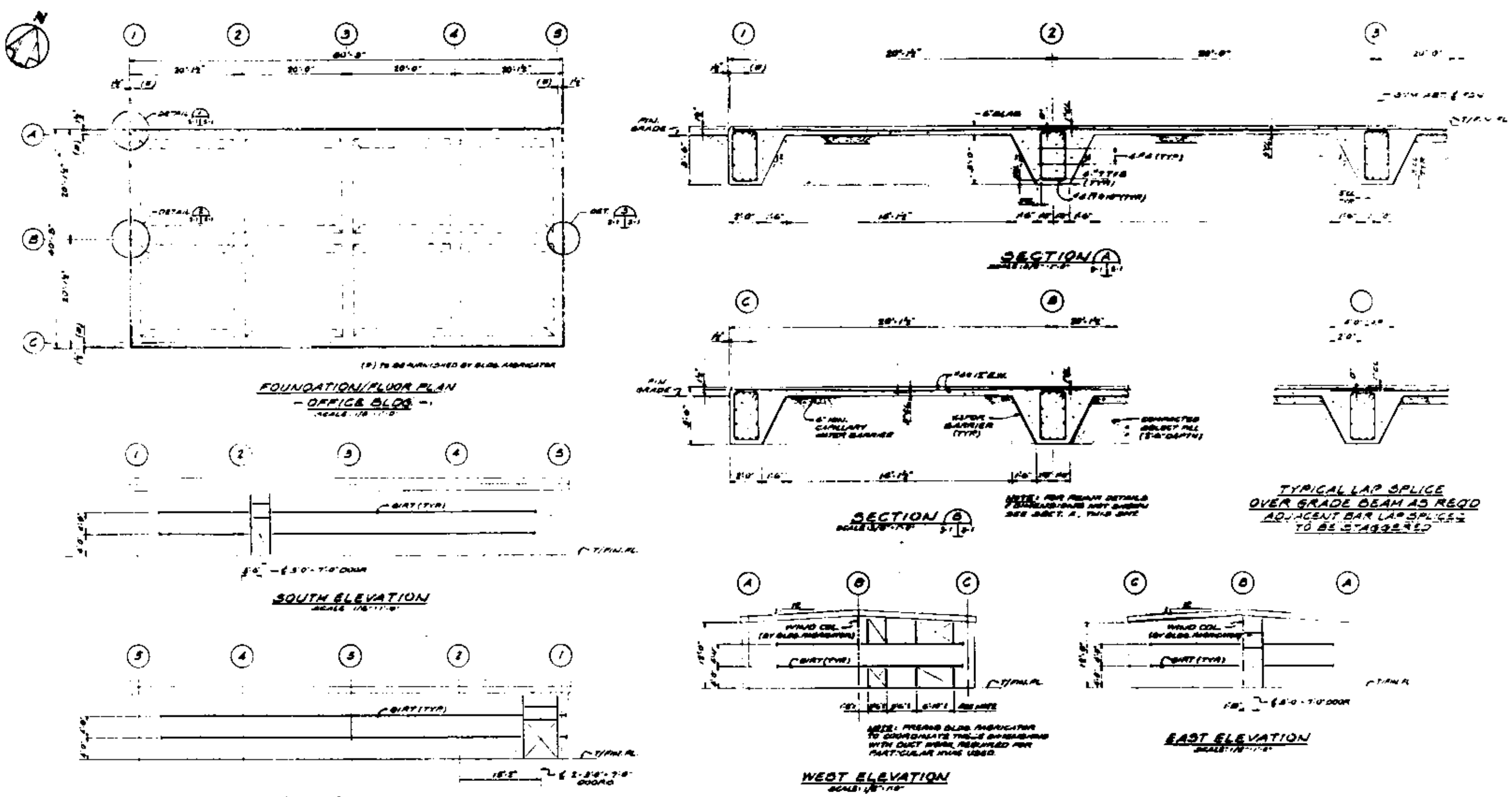
7.4.4 Automatic end-welded studs shall be shouldered type with a shank diameter of not less than 3/16 inch with cap and nut for holding the covering against the shoulder.

7.4.5 Fasteners for use with power actuated tools shall have a shank diameter of not less than 1/2 inch. Fasteners for securing wall panels shall have threaded studs for attaching approved nuts or caps.

7.4.6 Blind rivets shall be stainless steel with 1/8 inch nominal diameter shank. Rivets shall be threaded stem type if used for other than fastening of trim. Rivets with hollow stems shall have closed ends.

7.4.7 Bolts shall not be less than 1/4 inch diameter, shoulders or plain shank as required with proper nuts.

7.4.8 Provide oversize washers with an outside diameter of 1 inch at each fastener or a 22 gauge thick metal strip along each valley of the panel to negate pull-out of the panel around the fasteners.



GENERAL NOTES

1. ALL CONCRETE SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF f'_c = 4,000 psi AT 28 DAYS.

2. ALL REINFORCING BARS SHALL CONFORM TO SPECIFICATIONS FOR DEFORMED BILLET STEEL BARS FOR CONCRETE REINFORCEMENT, ASTM DESIGNATION A-615, GRADE 60, UNLESS NOTED OTHERWISE.

3. CONCRETE AGGREGATE SHALL HAVE A MAXIMUM SIZE OF 3/4 INCH.

4. MINIMUM AND MAXIMUM CONCRETE SLUMP SHALL BE 2 AND 3 INCHES, RESPECTIVELY, UNLESS NOTED OTHERWISE.

5. ALL REINFORCING BARS SHALL BE CONTINUOUS IN ANY ONE DIRECTION, EXCEPT WHERE OTHERWISE SHOWN ON DRAWINGS.

6. NO WELDING OF REINFORCING SHALL BE PERMITTED EXCEPT AS SHOWN ON DRAWINGS AND FOR BRAZING OR SPOT-WELDING AT ENDS OF REINFORCING BARS FOR GROUNDING.

7. SURFACES OF ALL JOINTS SHALL BE ROUGHENED BEFORE THE SET OF CONCRETE. SURFACES OF SET CONCRETE SHALL BE PREPARED BEFORE NEW POUR AS SPECIFIED IN THE ACI MANUAL OF CONCRETE INSPECTION, SP-2.

8. VERTICAL AND HORIZONTAL CONSTRUCTION JOINTS SHALL NOT BE PERMITTED, EXCEPT AS SHOWN ON DRAWINGS, OR WITH THE APPROVAL OF THE CONTRACTING OFFICER.

9. AT LEAST THE TOP 6 INCHES OF SUBGRADE SHALL BE COMPACTED TO 95% OF MAXIMUM DENSITY IN ACCORDANCE WITH ASTM STANDARD D-1557.

10. EXCEPT AS NOTED, ALL CONCRETE CONSTRUCTION, INCLUDING REINFORCEMENT DETAILING AND PLACEMENT, SHALL CONFORM TO THE MANUAL OF STANDARD PRACTICE FOR DETAILING REINFORCED CONCRETE STRUCTURES (ACI 315, LATEST EDITION), AND THE BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE (ACI 318 AND COMMENTARY - LATEST EDITION).

11. THE STRUCTURAL STEEL CONSTRUCTION SHALL CONFORM TO THE AISC SPECIFICATIONS FOR DESIGN, FABRICATION AND ERECTION OF STRUCTURAL STEEL FOR BUILDINGS.

12. ALL PRIMARY STRUCTURAL STEEL SHAPES AND BASE PLATES SHALL CONFORM TO SPECIFICATIONS FOR STRUCTURAL STEEL, ASTM DESIGNATION A-36 (LATEST EDITION), UNLESS NOTED OTHERWISE. HIGH-STRENGTH BOLTING SHALL BE IN ACCORDANCE WITH THE SCRBSJ SPECIFICATION FOR STRUCTURAL JOINTS USING ASTM A-325 BOLTS AND SHALL HAVE A MINIMUM DIAMETER OF 3/4 INCHES.

13. ALL SECONDARY STRUCTURAL STEEL CONSISTING OF HOT-ROLLED SHEET OR STRIP USED FOR THE FABRICATION OF COLD-FORMED MEMBERS SHALL CONFORM TO ASTM A-570 GRADE E. DESIGN OF COLD-FORMED MEMBERS SHALL BE IN ACCORDANCE WITH AISI SPECIFICATIONS. STANDARD CARBON BOLTS CONFORMING TO ASTM A-307 SHALL BE USED FOR ASSEMBLING COLD-FORMED MEMBERS. MINIMUM DIAMETER OF BOLTS IS 1/2 INCH.

14. ALL WELDING OF STRUCTURAL STEEL SHALL CONFORM TO THE STANDARD SPECIFICATIONS FOR STRUCTURAL WELDING CODE AWS D1.1 (LATEST EDITION).

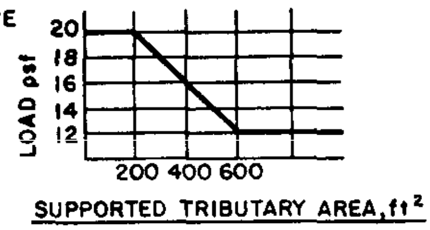
15. THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE DESIGN OF ALL STRUCTURAL STEEL CONNECTIONS AND OTHER STRUCTURAL MEMBERS, NOT SHOWN ON THE DRAWINGS, IN ACCORDANCE WITH THE AISC SPECIFICATIONS. ALL FLEXURAL MEMBER CONNECTIONS SHALL BE DESIGNED FOR THE FULL SHEAR CAPACITY OF THE MEMBER. BOLTED CONNECTIONS SHALL NOT HAVE LESS THAN TWO (2) BOLTS.

16. SHOP DRAWINGS SHALL BE SUBMITTED FOR APPROVAL.

Figure 6-17 General layout of pre-engineered building

1. FLOOR LIVE LOAD: 150 psf

2. ROOF LIVE LOAD



3. ROOF SNOW LOAD: 30 psf

4. DEAD LOAD: AS PER MATERIALS USED

5. WIND LOADS: WIND PARALLEL OR PERPENDICULAR TO ROOF RIDGE (BASED ON 100 mph WIND)

A. WINDWARD/LEEWARD AND ROOF PRESSURES (psf)

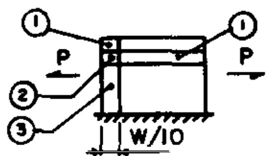
a. WHEN TRIBUTARY SUPPORT AREAS > 200 ft^2

FOR DESIGN OF MAIN FRAMES AND OTHER INDIVIDUAL MEMBERS, USE VALUES SHOWN.

b. WHEN TRIBUTARY SUPPORT AREA ≤ 200 ft^2

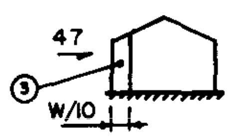
FOR DESIGN OF MAIN FRAMES, USE VALUES SHOWN FOR TRIBUTARY AREA > 200 ft^2. FOR DESIGN OF OTHER INDIVIDUAL MEMBERS, USE VALUES SHOWN

B. SIDEWALL PRESSURES (psf)



FOR DESIGN OF INDIVUAL MEMBERS:
WHEN TRIBUTARY SUPPORTED > 200 ft^2, USE P = 32
WHEN TRIBUTARY SUPPORTED ≤ 200 ft^2, USE P = 43

C. SIDESWAY PRESSURES (psf)



FOR DESIGN OF MAIN FRAMES FOR TRIBUTARY SUPPORTED AREAS, TA, LESS THAN, EQUAL TO, OR GREATER THAN, 200 ft^2.

D. LOCAL PRESSURES - SEE FIGURES IN 'B' AND 'C' FOR REGIONS OF APPLICATION

REGION 1: 50 psf	SUCTION	FOR DESIGN OF DECKING
REGION 2: 105 psf	SUCTION	CONNECTIONS AT REGIONS
REGION 3: 42 psf	SUCTION	CITED

"W" IS LEAST WIDTH OF ENCLOSED AREA

NOTE: FIGURES DEPICT WIND PERPENDICULAR TO RIDGE. FOR WIND PARALLEL TO RIDGE, USE SAME VALUES.

6. LOADING COMBINATIONS:

A. D	WHERE
B. D + L	D = DEAD LOAD
C. D + W	L = LIVE LOAD
D. 0.75 (D + L + W)	W = WIND LOAD

Figure 6-18 Recommended pre-engineered building design loads

SUPPRESSIVE SHIELDING

6-23. General

This manual presents methods for the design and construction of conventional reinforced concrete and steel protective facilities which provide adequate safety for hazardous operations such as munitions loading, maintenance, renovation, or demilitarization. Such safety considerations include the utilization of conventional protective barriers, total containment construction, or the use of separation distances or isolation of the specific operation from other parts of the facility using appropriate quantity distance specifications. However, an alternative available to the designer of these facilities is the use of suppressive shielding as outlined in HNDM 1110-1-2, "Suppressive Shields Structural Design and Analysis Handbook," 18 November 1977.

A suppressive shield is a vented steel enclosure which controls or confines the hazardous blast, fragment, and flame effects of detonations. Suppressive shielding may provide cost or safety effective alternatives to conventional facilities, depending upon the hazardous situation under study. HNDM 1110-1-2 presents procedures for design, analysis, quality control, and economic analysis of suppressive shields. In this section, a brief review of these procedures is presented. The reader should refer to HNDM 1110-1-2 for details necessary for design.

6-24. Application

Facility operations such as munitions loading, maintenance, modification, renovation, or demilitarization must be analyzed to determine which operations involve potentially catastrophic (CAT I or II, MIL STD 882A) hazards in the event of an inadvertent ignition or detonation. Where the hazard analysis shows such a potential, the facility design must provide adequate safety for those operations. The alternatives presented to the designer of the facility are varied and may include the utilization of conventional protective barricades with appropriate separation distances, reinforced concrete or steel structures, suppressive shields, or isolation of a particular operation from the rest of the facility by the appropriate quantity-distance. The decision as to which alternative system to use is based primarily on economic factors, provided all safety considerations are equal. The facility, availability of real estate, and equipment costs to include maintenance, operation, useful life, replacement, and modification or renovation must be analyzed for each alternative method of protection. Costs will be estimated and compared over the facility life to determine the most economical mode of protection.

A major factor which is paramount in the determination of which form of protection to use is the requirement for approval of the facility by the Department of Defense Explosives Safety Board. If the designer can, based on economic factors, adapt suppressive shields in the design and support the adaptation with proven accepted analytical techniques, he should begin development of a facility concept which employs suppressive shields using those shields which have been safety approved.

6-24.1. Safety Approved Suppressive Shields

There are eight suppressive shield design groups that have been developed to various stages of definition. These shield groups are summarized in Table 6-4 and illustrated schematically in Figure 6-19. Of the design groups illustrated, five had been safety approved by the Department of Defense Explosive Safety Board in 1977.

The five suppressive shield group designs approved by the DoD Explosive Safety Board (Groups 3, 4, 5, 6, and 81 mm) have been designed to meet the requirements for most applications to ammunition load, assembly, pack (LAP) in the Munitions Production Base Modernization and Expansion Program. However, specific shield requirements will vary with other applications and, even with LAP applications, design details will vary from plant to plant and between munitions or different operations on the line. It will, therefore, frequently be necessary to modify the approved shields to adapt them to the operation under consideration.

Chapter II and Appendix A of HNDM 1110-1-2 describes the safety approved shield group designs, provides guidance concerning acceptable modifications, recommends procedures for securing safety approval of new shield designs, and provides summary information on overall dimensions of the shield structure, charge capacity, rated overpressure, fragment stopping wall thickness, and type of construction of the five approved basic shield groups.

6-24.2. New Shield Design

In exceptional cases where a safety approved shield cannot be made to fit a desired application, a new shield can be designed. The guidance needed to design a new shield and the procedures for obtaining the safety approval for the new design are outlined in HNDM 1110-1-2.

6-24.2.1. Hazardous Environments

Considering that the hazardous environments normally associated with suppressive shielding involves explosives and/or explosive ordnance, Chapter III of HNDM 1110-1-2 presents information relative to internal and external air blast, fragmentation, and fireball phenomena. This information can be used in support of blast and fragment methods in this manual to determine venting requirements, air blast loads on the structure, and protection required to defeat fragments. Some graphs and prediction methods in Chapter 2 of this manual are taken directly from the suppressive shields manual.

6-24.2.2. Structural Behavior

Suppressive shields can be subjected to large, high pressure loads applied very rapidly. The allowance of inelastic behavior of the shield material structural elements enables much more efficient use of the structural material and does not impair the function of the shield provided, of course, that the inelastic behavior is maintained within acceptable limits.

The structural materials of primary interest in suppressive shielding are steel and reinforced concrete. Chapter IV of HNDM 1110-1-2 discusses the behavior and properties of these structural materials under static and dynamic loading. Additionally, ductility ratios as they apply to suppressive shield application are covered. Some of the information provided will duplicate material in chapters of this manual. In case of a conflict, this manual takes precedence.

6-24.2.3. Structural Design and Analysis

Chapter V of HNDM 1110-1-2 describes techniques which are sufficiently accurate for preliminary designs in all cases, and in most cases, adequate for final designs. These methods deal primarily with the dynamic loadings imposed by internal explosions. The design methods supplement material presented in chapters of this manual. Again, in case of conflict, this manual takes precedence.

6-24.2.4. Structural Details

Each suppressive shield used for ammunition manufacturing and other hazardous operations will have specific requirements for utility penetrations, and doors for personnel, equipment, and products. Guidance on the provision of acceptable structural details such as these is presented in Chapter VI of HNDM 1110-1-2 along with information on structural details which have been successfully proof-tested.

6-24.2.5. Economic Analysis

The design of a facility entails the need to ascertain the most cost effective configuration from among a set of workable design alternatives. All will be designed to provide the desired level of reliability and safety, and the selection of one over another will be based primarily on dollar costs. The economic analysis of alternative facility design is a complex process unique to each facility. Chapter VII of HNDM 1110-1-2 illustrates the many factors that must be considered.

6-24.2.6. Assuring Structural Quality

In the design of suppressive shields, specifications for the quality of the basic material is paramount. The strength of welds and concrete components are also determining factors in the overall strength of the structure. Chapter VIII of HNDM 1110-1-2 provides the guidance which outlines a quality assurance program for suppressive shield design packages.

Included in Appendix A of HNDM 1110-1-2 is a detailed description of the safety approved suppressive shields and guidance concerning acceptable modifications. Copies of the fabrication drawings for each approved shield design are included along with direction for ordering full-size copies. Appendix B of that manual includes response charts for use in preliminary design. The charts are based on the combined short duration shock load and infinite duration quasi-static load, along with an undamped elastic-plastic responding structure.

6-25. Design Criteria

Design criteria for use of suppressive shields, or suppressive shielding panels, are very dependent on specific applications in protective structures. These criteria may include complete suppression of fragmentation effects, both primary and secondary; attenuation of blast overpressures and impulses to specified levels of specific distances from the shield or shield panels; attenuation of fireball radiation; or even essentially complete suppression of all of these effects.

Suppressive shields may or may not present reasonable or cost effective solutions to specific design problems in protective structures. Generally, they have appeared attractive when fragment hazards are severe and when potentially explosive sources are rather concentrated. The safety-approved shields protect against effects as limited as small trays of detonators, and as severe as a large melt kettle in a HE melt-pour operation containing several thousand pounds of explosive. The designer should consider their use, and use the methods presented in HNDM 1110-1-2 to evaluate their efficacy, compared to other types of protective structures discussed in this manual.

No general design criteria can be given here because the criteria for different operations or plants, and available real estate, differ too widely. In each specific protection design contract, the AE should be provided with quite detailed design criteria, in addition to general regulations which fix safety criteria such as AMCR 385-100. Both the specific and more general criteria must be evaluated when deciding whether or not suppressive shields will be useful in the facility design.

6-26. Design Procedures

6-26.1. Space Requirements

Once the operation requiring suppressive shields has been identified, consideration must be given to the size and shape of the equipment needed to perform the operation and the work space required inside the shield. These factors necessarily provide the designer with an estimate of the size and shape of the shield required. Additionally, space available on the line or in the building will place limitations on the overall shield base dimensions and height.

6-26.2. Charge Parameters

A principal factor in the selection of a shield which will govern the shield requirements is the establishment of the charge parameters for any specific application. The charge parameters are: charge weight (W). shape, confinement, and composition; ratio of charge weight to shield internal chapter (W/V); and scaled distance (Z) from the charge to the nearest wall or roof of the shield. ($Z = R/W^{1/3}$), where R is the distance from the center of the charge to the nearest wall or roof in feet and W is the charge weight in pounds. These parameters for approved shield groups are summarized in Table 6-5. New shield designs can be developed for individual needs.

6-26.3. Fragment Parameters

Another key factor in the procedure a designer follows in the selection of an approved design or the design of a new concept is the suppression of primary and secondary fragments generated by the detonation of explosives or munitions. Much of the material in HNDM 1110-1-2 for fragment perforation of spaced plates has been adapted to Chapter 5 of this manual.

6-26.4. Structural Details

Suppressive shields used for ammunition manufacturing and other hazardous operations require provisions for gaining access to the operation being protected. Personnel must be able to enter the shield to accomplish routine and emergency maintenance and clean-up and other essential operations. An opening of sufficient size must be provided to enable the installation or removal of equipment in realistically large subassemblies. Openings for conveyors and chutes must also be provided and properly configured to prevent excessive pressure and fragments from escaping. Provisions must be made to provide all utilities and satisfy all environmental conditioning needs which may be essential to the operations inside the shield.

Utility penetrations, ventilating and air-conditioning ducts, and vacuum lines must not diminish the overall protective capability of the shield. They must not alter the basic mode of structural failure of the suppressive shield and should be small compared to the general size of the shield.

Operations that produce explosive dust may require the use of liners both inside and outside the shield to prevent the accumulation of dust within shield panels. With configurations such as the Group 5 shield, which is primarily designed for use with propellants or pyrotechnic materials, liners must not inhibit the venting characteristics of the shield.

Utility lines passing through suppressive shields are vulnerable to both air blast and fragment hazards. The air blast could push unprotected utility penetrations through the walls of the shield and create secondary fragments. Fragments from an accidental explosion could perforate the thin walls of an unprotected utility pipe and escape from the shield. To eliminate the threat of air blast and fragments, a protective box is used to cover the area where the utility lines pass through the shield wall. The box is configured to rest on the inside surface of the shield and is welded to the shield. The size of the wall penetrations is limited to that required for the utilities. Each pipe is bent at a right angle inside the shield within the protective box. The penetrations of the shield wall are reinforced with a sleeve or box section welded to the shield panel through which the utility line passes. The penetration box is designed to maintain the structural integrity of the shield area penetrated. A typical protective box design is shown in Figure 6-20.

The cover plate thickness is selected to stop the worst case fragment.

Typical penetrations for approved safety design suppressive shields are illustrated in Figures 6-21 and 6-22, and a vacuum line penetration is illustrated in Figure 6-23.

6-26.5. Access Penetrations

In the munitions plant environment, suppressive shields are designed to protect Category I or II hazardous operations as defined in MIL STD 882A. Remote operation may be required so personnel will not be inside the shield during operations. However, personnel access is required to allow for maintenance, repair, and inspection. Further, these doors must provide large openings to enable most equipment to be installed or removed in large subassemblies.

Access is also required for munitions components, explosives, and assembled munitions to pass through the suppressive shield. In the case of conveyor transporting systems, consideration must be given to the proper pass-through of the conveyor. Requirements for this type of access depend on the configuration of the munitions product, transporting pallets, and conveyors, as well as production rates and other factors unique to each operation. For these reasons, definition of specific design requirements is not possible.

6-26.5.1. Personnel Door

Three different types of doors have been developed for use in suppressive shields: sliding, hinged, and double leaf. The hinged door was designed to swing inward. This feature reduces the usable space inside the shield. A sliding door is preferred for personnel access to munitions operations. Figure 6-24 illustrates a typical sliding door. This type door is used with the Group 4, 5, and Milan 81 mm shields. The sliding door consists of an entire shield panel suspended from a monorail system. The panel is inside the shield and is not rigidly attached to the column members. Special consideration was given to the air gap between the door panel and the column to assure that excessive pressure leakage would not occur and that fragments could not pass through the gap.

The cylindrical Group 3 shield contains a two-leaf door, hinged at each side. It swings inward as shown in Figure 6-25. The door is curved to match the shield wall contour and is fabricated from S5 x 10 I-beams. Pressure loading restraint is provided by the door bearing on the external support rings of the shield at the top and bottom of the door. An external latch provides restraint during rebound of the door.

6-26.5.2. Product Door

Only one type of product door has been developed conceptually for use in suppressive shields. It is the rotary, three lobed configuration shown in Figure 6-26. The design procedure for this door is described to illustrate the type of analysis required. It can be used as a guide for analysis of similar alternate design concepts for product doors.

The air blast will most severely load the rotating product door when the munitions opening is coincident with the pocket in the rotary door. A non-overriding clutch prevents the door from counter-rotating. The angular impulsive load is:

$$T_i = i_r A_d r_d \qquad \text{6-38}$$

where

T_i = angular impulsive load
i_r = reflected impulse
A_d = door area
r_d = radius from center of impulse load to the center of door rotation

Assuming the product door to be initially at rest, the rotational velocity imparted to the door is given by:

$$w_i = T_m / I \qquad \text{6-39}$$

where

ω = angular velocity

I_m = mass moment of inertia of the door about shaft axis

The kinetic energy imparted to the door is given by:

$$KE = \frac{I_m \omega^2}{2} = \frac{T_i^{\,2}}{2I_m} \tag{6-40}$$

The strain energy absorbed by a circular shaft is given by:

$$U_s = \frac{\pi L_s}{4G}(r_s \tau_s)^2 \tag{6-41}$$

where

U_s = strain energy
L_s = length of the shaft
G = shear modulus of the shaft material
r_s = radius of the shaft
τ_s = maximum shear stress in the shaft

Equating the kinetic energy of the rotating door to the strain energy in the shaft and solving for the shear stress yields:

$$\tau_s = \frac{T_i}{r_s}\left[\frac{2G}{\pi I_m L_s}\right]^{1/2} \tag{6-42}$$

The computed shear stress in the shaft must be less than the dynamic shear stress of the shaft material, i.e.,:

$$s < 0.55 f_{dy} \tag{6-43}$$

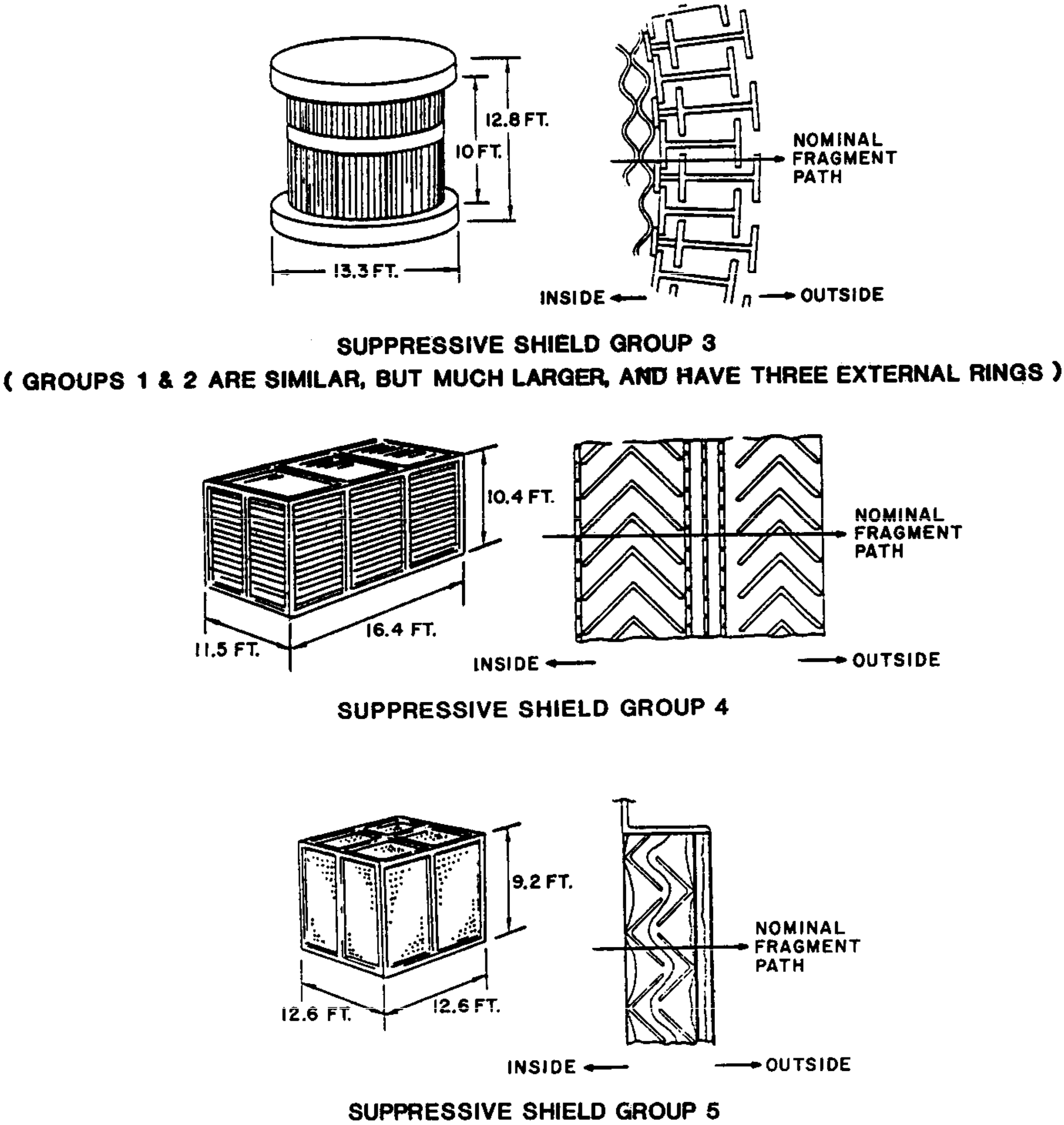
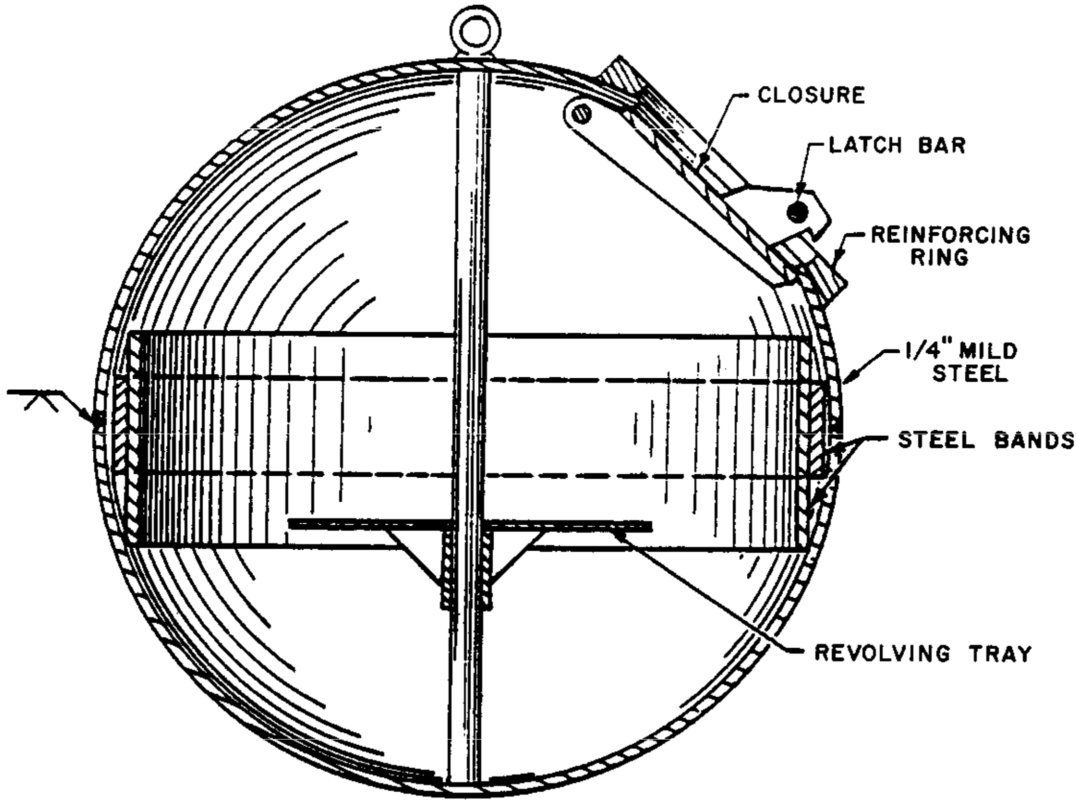
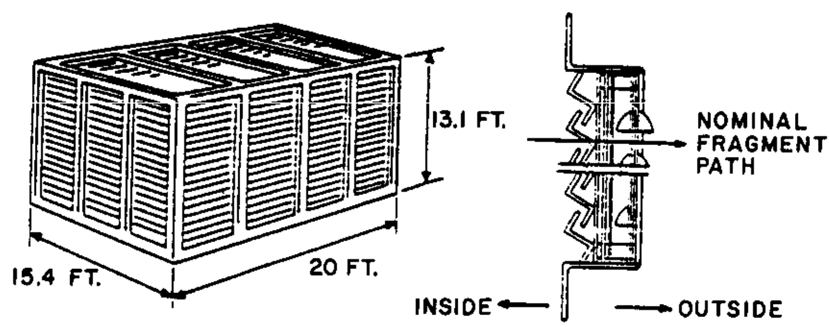


Figure 6-19a General configuration of suppressive shield groups



SUPPRESSIVE SHIELD GROUP 6



SUPPRESSIVE SHIELD GROUP 81mm

Figure 6-19b General configuration of suppressive shield groups (continued)

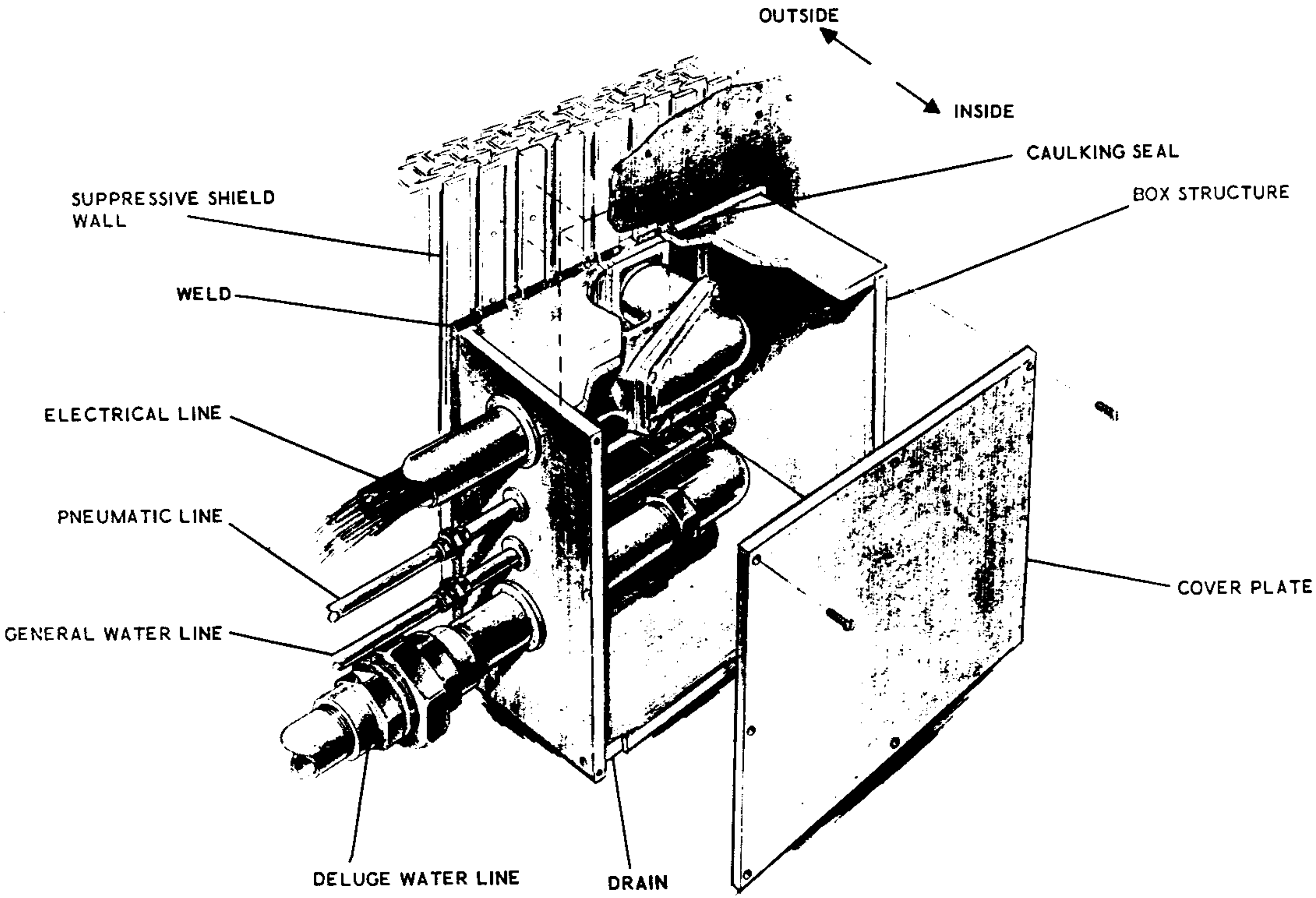


Figure 6-20 Typical utility penetration

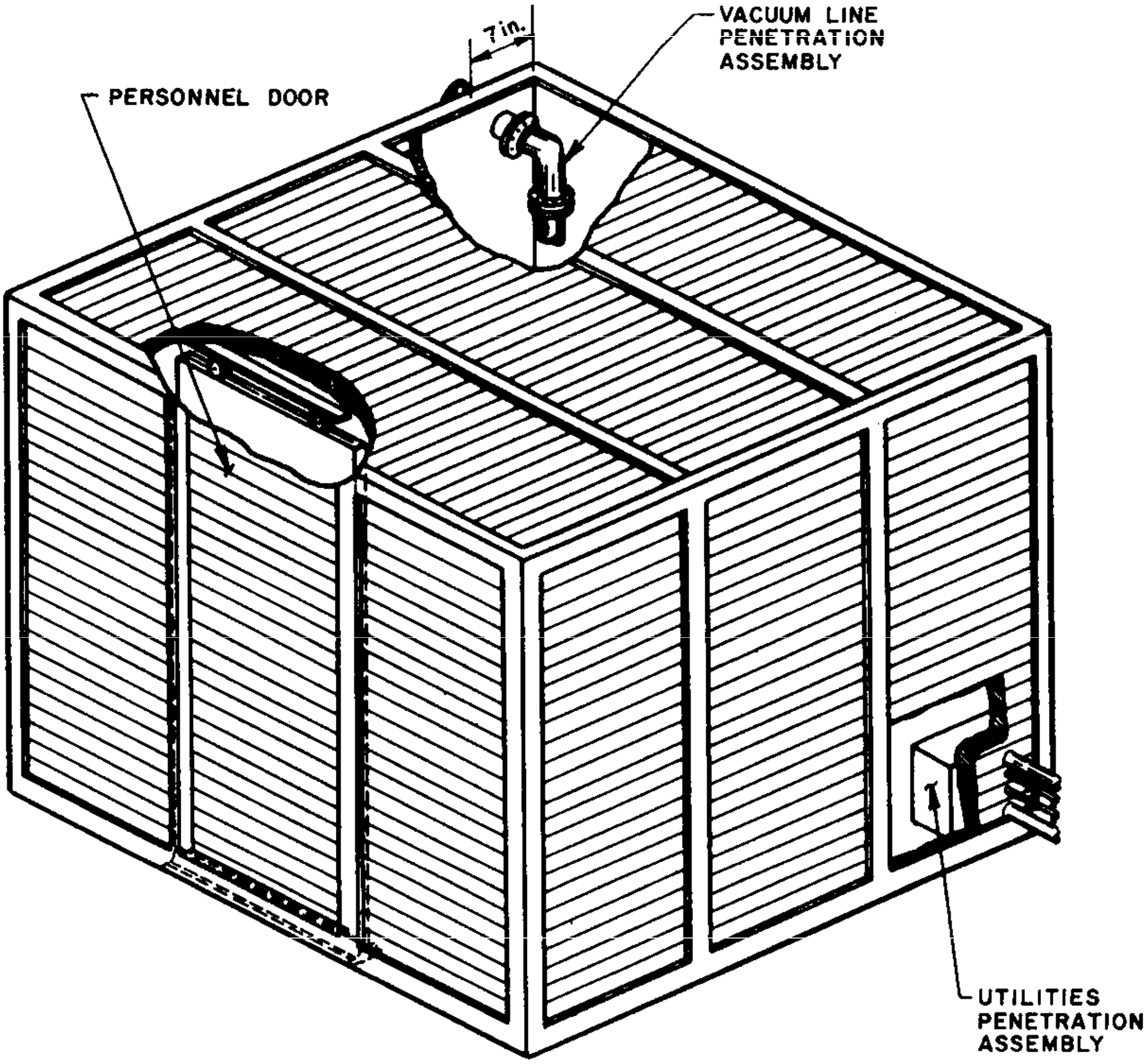


Figure 6-21 Typical location of utility penetration in shield groups 4, 5, and 81 mm

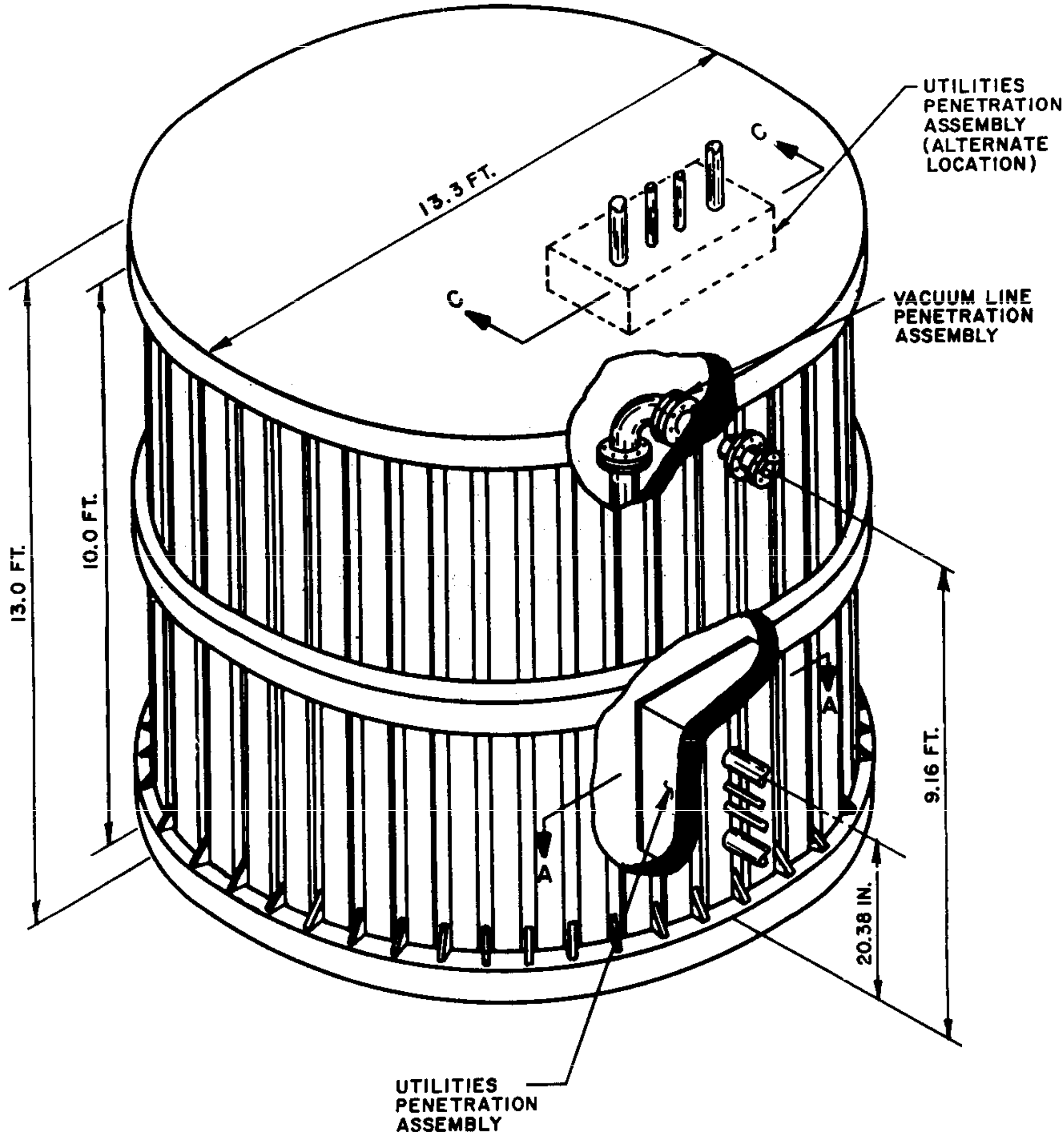


Figure 6-22 Typical location of utility penetrations in shield group 3

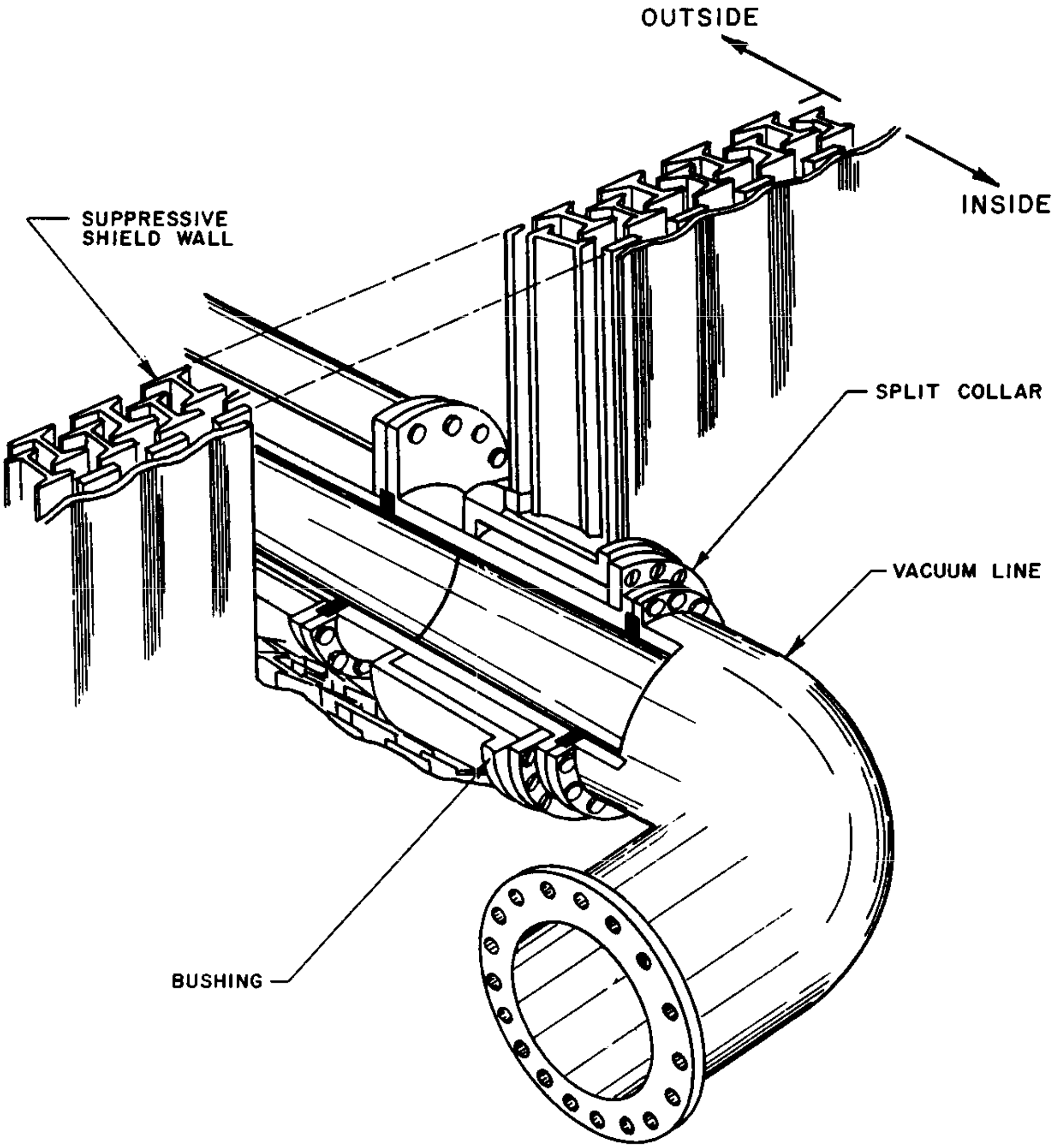


Figure 6-23 Typical vacuum line penetration

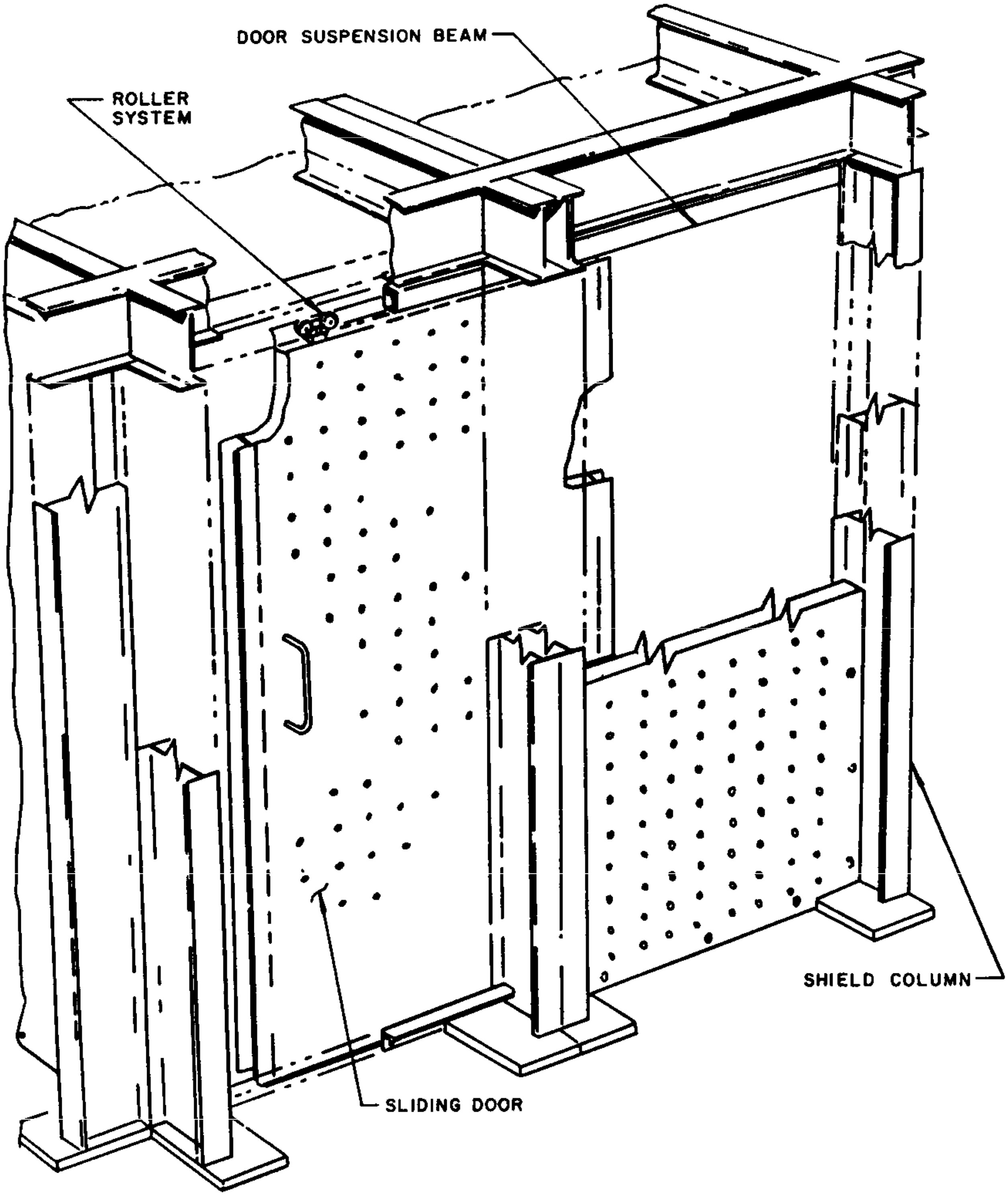


Figure 6-24 Sliding personnel door

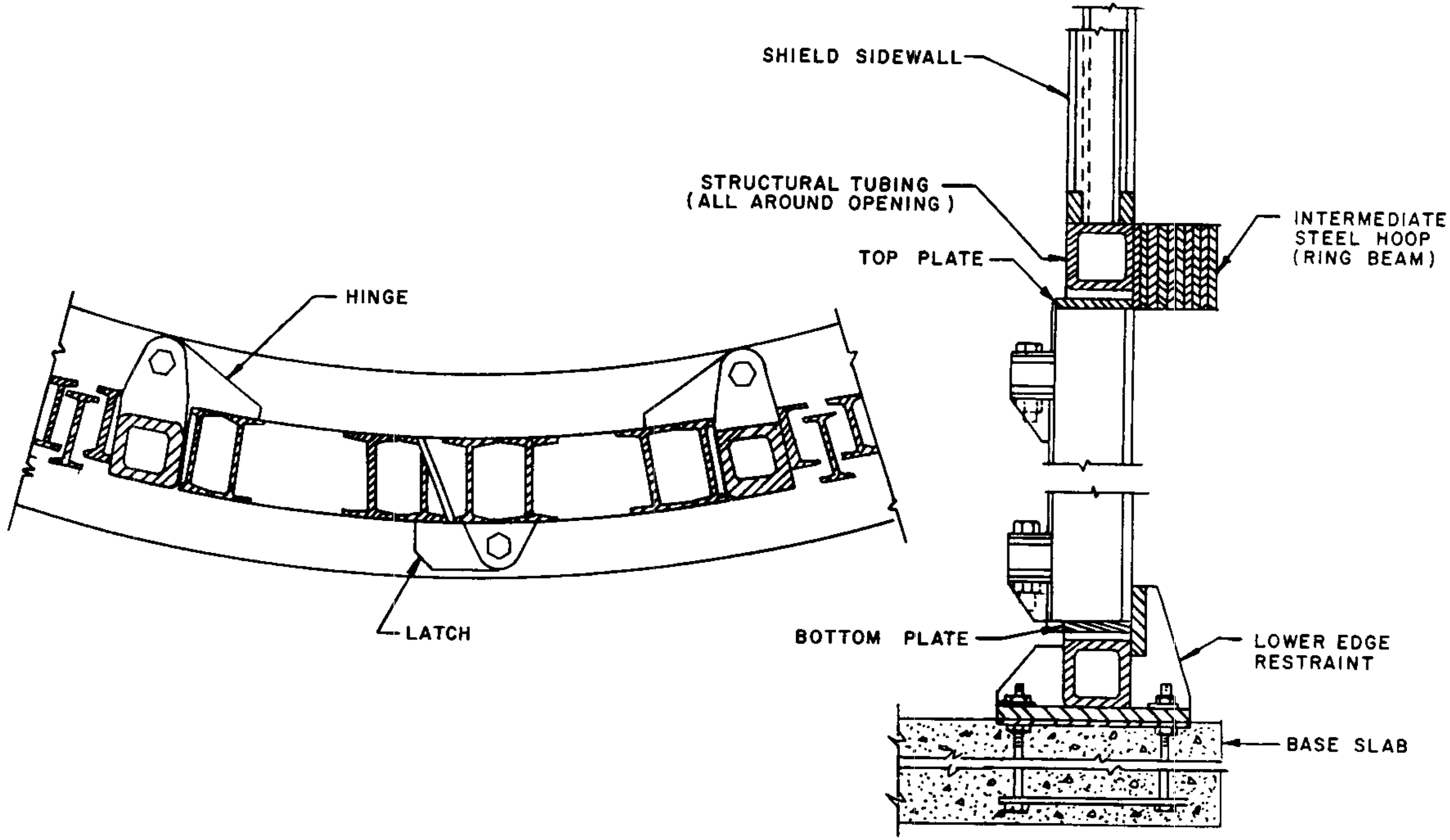


Figure 6-25 Door - group 3 shield

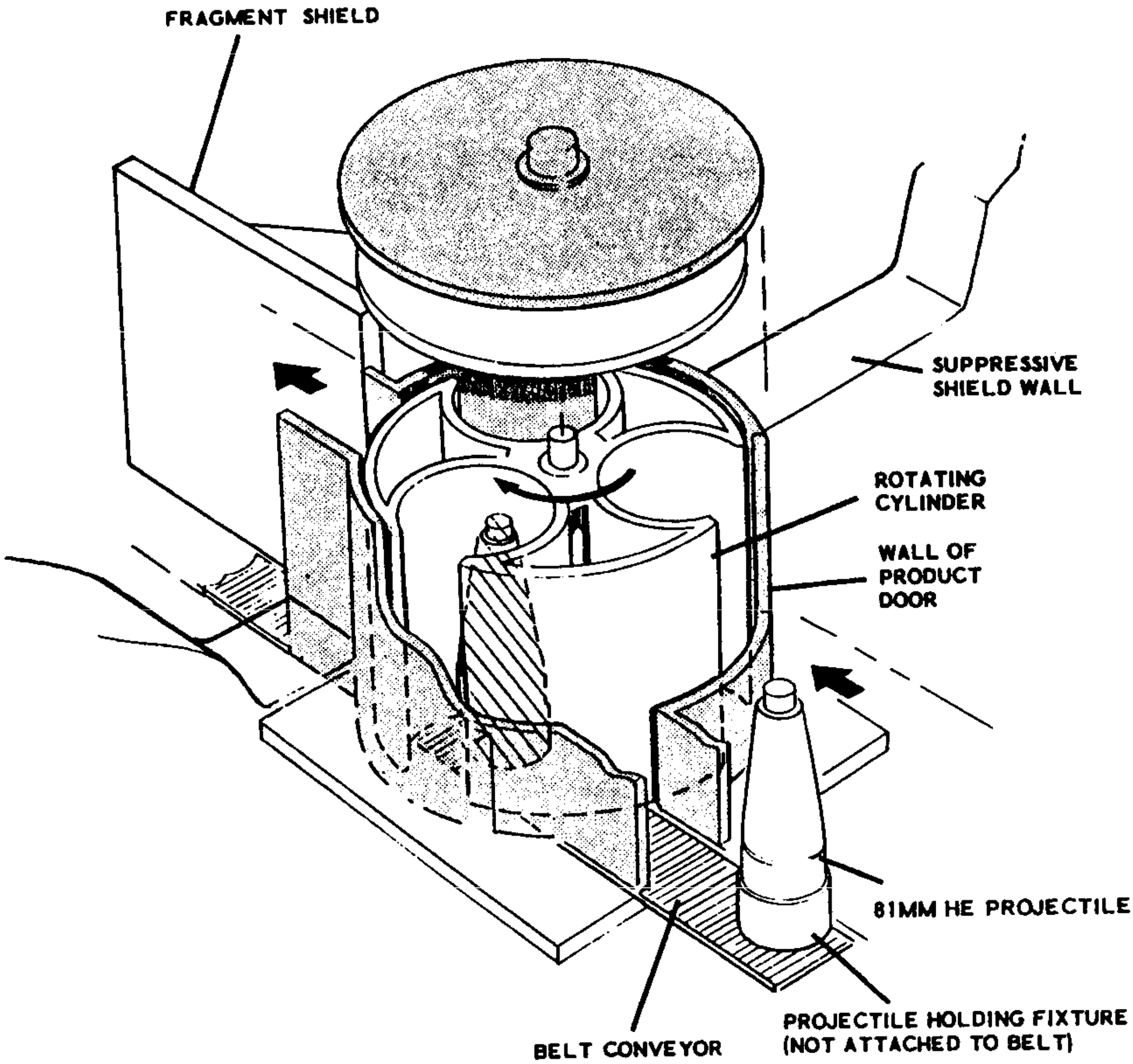


Figure 6-26 Rotating product door

Table 6-4 Summary of Suppressive Shield Groups

SHIELD GROUP	HAZARD PARAMETER		REPRESENTATIVE APPLICATIONS	LEVEL OF PROTECTION *
	BLAST	FRAGMENTATION		
1	High	Severe	Porcupine Melter (2000 lbs) plus two pour units 250 lbs each	Reduce blast pressure at intraline distance by 50 percent
2	High	Severe	HE Bulk (750 lb) Minute melter	Reduce blast pressure at intraline distance by 50 percent
3	High	Moderate	HE Bulk (37 lb) Detonators, fuses	Category I hazard 6.2 feet from shield
4	Medium	Severe	HE Bulk (9lb) Processing rounds	Category I hazard 19 feet from shield
5	Low	Light	30 lb Illuminant Igniter slurry mixing HE processing (1.84 lb)	Category I hazard 3.7 feet from shield
6	Very High	Light	Laboratory, handling, and transportation	Category I hazard at 1 foot from shield
7	Medium	Moderate	Flame/fireball attenuation	Category I hazard at 5 feet from shield
81 mm	High	Moderate	81 mm mortar drill-and-face and/or cast-finishing operation	Category I hazard at 3 feet from shield

* All shield groups contain all fragments

Table 6-5 Charge Parameters for Safety Approved Shields

SHIELD GROUP	MINIMUM Z ($ft/lb^{1/3}$) WALL	ROOF	MAXIMUM W/V (lb/ft^3)
3	1.63	1.45	0.04157
4	2.23	2.19	0.00762
5	4.14	6.79	0.00215
6A	1.01	N/A	0.22970
6B	1.22	N/A	0.13200
Prototype 81 mm *	3.62	3.21	0.00340
Milan 81 mm *	4.23	3.75	0.00280

* See Figure 6-19

BLAST RESISTANT WINDOWS

6-27. Introduction

Historical records of explosion effects demonstrate that blast-propelled glass fragments from failed windows are a major cause of injuries from accidental explosions. Also, failed window glazing often leads to additional injuries as blast pressure can enter interior building spaces and subject personnel to high pressure jetting, incident overpressure, secondary debris impact, and thrown body impact. These risks are heightened in modern facilities, which often have large areas of glass.

Guidelines are presented for both the design, evaluation, and certification of windows to safely survive a prescribed blast environment described by a triangular-shaped pressure-time curve. Window designs using monolithic (unlaminated) thermally tempered glass based on these guidelines can be expected to provide a probability of failure equivalent to that provided by current safety standards for safely resisting wind loads.

Guidelines are presented in the form of load criteria for the design of both the glass panes and framing system for the window. The criteria account for both the bending and membrane stresses and their effect on maximum principle stresses and the nonlinear behavior of glass panes. The criteria cover a broad range of design parameters for rectangular-shaped glass panes. Design charts are presented for monolithic thermally tempered glazing with blast overpressure capacity up to 100 psi, an aspect ratio of $1.00 \leq a/b \leq 4.00$, pane area $1.0 \leq ab \leq 25$ ft^2, and nominal glass thickness $1/4 \leq t \leq 3/4$ inches. An alternate method for blast capacity evaluation by calculation is also presented. This can be used to evaluate the blast capacity of glass when interpolation between charts is unadvisable, when design parameters are outside the limits of the chart, and to calculate rebound loads. Presently, the design criteria are for blast-resistant windows with thermally treated, monolithic tempered glass. Further research is required to extend these design criteria to laminated tempered glass.

6-28 . Background

The design criteria cover monolithic tempered glass meeting the requirements of Federal Specifications DD-G-1403B and DDG-451d. Additionally, thermally tempered glass must meet the minimum fragment weight requirements of ANSI Z97.1-1984, Section 5.1.3(2).

Annealed glass is the most common form of glass available today. Depending on manufacturing techniques, it is also known as plate, float or sheet glass. During manufacture, it is cooled slowly. The process results in very little, if any, residual compressive surface stress. Consequently, annealed glass is of relatively low strength when compared to tempered glass. It has large variations in strength and fractures into dagger-shaped, razor-sharp fragments. For these reasons, annealed glass is not recommended for use in blast resistant windows.

Thermally tempered glass is the most readily available tempered glass on the market. It is manufactured from annealed glass by heating to a high uniform temperature and then applying controlled rapid cooling. As the internal temperature profile relaxes towards uniformity, internal stresses are created.

The outer layers, which cool and contract first, are set in compression, while internal layers are set in tension. As it is rare for flaws, which act as stress magnifiers, to exist in the interior of tempered glass sheets, the internal tensile stress is of relatively minimal consequence. As failure originates from tensile stresses exciting surface flaws in the glass, precompression permits a larger load to be carried before the net tensile strength of the tempered glass pane is exceeded. Tempered glass is typically four to five times stronger than annealed glass.

The fracture characteristics of tempered glass are superior to those of annealed glass. Due to the high strain energy stored by the prestress, tempered glass will eventually fracture into small cubical-shaped fragments instead of the razor-sharp, dagger-shaped fragments associated with the fracture of annealed glass. Breakage patterns of side and rear windows in American automobiles are a good example of the failure mode of thermally or heat-treated tempered glass.

Semi-tempered glass is often marketed as safety or heat-treated glass. However, it exhibits neither the dicing characteristics upon breakage nor the higher tensile strength associated with fully tempered glass, and, therefore, it is not recommended for blast resistant windows.

Another common glazing material is wire-reinforced glass, annealed glass with an embedded layer of wire mesh. Its only use is as a fire-resistant barrier. Wire glass has the fracture and low strength characteristics of annealed glass and, although the wire binds fragments, it contributes metal fragments as an additional hazard. Wire glass is never recommended for blast resistant windows.

6-29. Design Criteria for Glazing

6-29.1. Specified Glazing

The design of blast-resistant windows is currently restricted to heat-treated, fully-tempered glass in fixed or non-openable frames meeting both Federal Specification DD-G1403B and ANSI Z97.1-1984. Tempered glass meeting only DD-G-1403B may possess a surface precompression of only 10,000 psi. At this level of precompression, the fracture pattern is similar to annealed and semi-tempered glass. Tempered glass meeting the minimum fragment specifications of ANSI Z97.1-1984 (Section 5.1.3(2)) has a higher surface precompression level and tensile strength, which improves the capacity of blast-resistant windows and results in smaller, cubical-shaped fragments on failure.

Although thermally tempered glass exhibits the safest failure mode of any glass, failure under blast loading still presents a significant health hazard. Results from blast tests reveal that on failure, tempered glass fragments may be propelled in cohesive clumps that only fragment on impact into smaller rock-salt-shaped fragments. Even if the tempered glass initially breaks into small fragments, sufficient velocities may still be imparted by the blast loading to cause severe hazards to personnel. Because the expected geometry of glass fragments involve multiple, potentially sharp corners, a high probability of injury would result from application of the 58 ft-lb criterion for acceptable kinetic energy. Because of these hazards to personnel, blast-resistant glazing should be designed to survive expected loads.

6-29.2. Design Stresses

The design stress, the maximum principal tensile stress allowed for the glazing, f_{un}, is set at 16000 psi. This correlates with a probability of failure equal to or less than 0.001. The design stress for blast-resistant glazing is slightly higher than that commonly used in the design of one-minute wind loads, but, it is justified because of the expected, significantly shorter, duration of loading.

6-29.3. Dynamic Response to Blast Load

An analytical model was used to predict the blast load capacity of annealed and tempered glazings. Characteristic parameters of the model are illustrated in Figure 6-27.

The glazing is a rectangular, fully thermally tempered glass plate having a long dimension, a; a short dimension, b; a thickness, t; a poisson ratio, $\nu = 0.22$; and an elastic modulus, $E = 1 \times 10^6$ psi. The plate is simply supported along all four edges, with no inplane and rotational restraints at the edges. The bending stiffness of the support elements is assumed to be infinite relative to the pane. Recent static and blast load tests indicate that the allowable frame member deflections of 1/264-th of the span will not significantly reduce pane resistance from that predicted for an infinitely stiff frame.

The blast pressure loading is described by a peak triangular-shaped pressure-time curve as shown in Figure 6-27. The blast pressure rises instantaneously to a peak blast pressure, B, and then decays linearly with a blast pressure duration, T. The pressure is uniformly distributed over the surface of the pane and applied normal to the pane.

The resistance function, r(X) (static uniform load, r as a function of center deflection, X) for the plate accounts for both bending and membrane stresses. The effects of membrane stresses produce a nonlinear stiffness of the resistance-deflection function (Figure 6-27). The design deflection, X_u is defined as the center deflection where the maximum principle tensile stress at any point in the glass first reaches the design stress, f_{un}, of 16,000 psi. Typically, as the deflection of the plate exceeds a third of its thickness, the points of maximum stress will migrate from the center and toward the corners of the plate.

The model, illustrated in Figure 6-27, uses a single-degree-of-freedom system to simulate the dynamic response of the plate. To be conservative, no damping of the window pane is assumed. The model calculates the peak blast pressures required to exceed the prescribed probability of failure. The model assumes that failure occurs when the maximum deflection exceeds ten times the glazing thickness. This restricts solutions to the valid range of the Von Karmen plate equations while preventing edge disengagement of the plate.

6-29.4. Design Charts

Charts are presented in Figures 6-28 to 6-48 and Tables 6-6 to 6-12 for the design and evaluation of glazing to survive a prescribed blast loading with a probability of failure no greater than 0.001. The charts relate the peak blast overpressure capacity of thermally tempered glazing to all combinations of the following design parameters: a/b = 1.00, 1.25, 1.50, 1.75 2.00, 3.00, and 4.00; $1.00 \leq ab \leq 25$ ft^2; $12 \leq b \leq 60$ inches; $2 \leq T \leq 1{,}000$ msec; and t = 1/4, 5/16, 3/8, 1/2, 5/8, and 3/4 inches (nominal). Thermally tempered glass up to 3/4 inch thick can easily be purchased in the United States. Thickness greater than 3/4 inch can only be obtained by lamination, but, research and blast load testing are required to develop approved designs for laminated glass.

The charts are based on the minimum thickness of fabricated glass allowed by Federal Specification DD-G-451d (see Table 6-7). They are created by numerically integrating the equations of motion of a single-degree-of-freedom system as modeled in Figure 6-27. A Wilson-Theta technique was employed with a time step corresponding limited to 1/25th of each of the five increasing linear resistances used to model the resistance function.

6-29.5. Alternate Design Procedure

Design procedures in this section can be used to evaluate the blast capacity of monolithic tempered glass when interpolation between charts is unadvisable, when design parameters are outside the limits of the chart, or to calculate rebound loads. It is recommended that the design charts be used for initial guesses of required glass thickness.

Procedures to calculate resistance, r_u, (Table 6-6) deflection, X_u, effective static resistance, r_{eff}, effective pane stiffness, K_e, and the period of vibration, T_N follow. The response charts can be used with these parameters to determine dynamic response. Table 6-11 reports the fundamental period of vibration and Table 6-12 reports the effective elastic, static resistance for most dimensions of glass panes. In many cases these values and the single equation in Step 10 below can be used to directly compute blast overpressure capacity.

<u>Step 1.</u> Determine if the glass pane will behave as a linearly responding plate under the design load. If the ratio of the short side of the plate, b, to its actual (not nominal) thickness is less than the maximum in the second column of Table 6-8, then simple formulas can be used for parameters needed to use the response charts. Only glass panes above and to the left of the steeped line in Table 6-6 will qualify. If the glass pane has a b/t-ratio less than that specified in Table 6-8, the glass will behave in a nonlinear manner with the membrane stresses induced by straining of the neutral plane or axis of the plate. Proceed to steps 2-9 to determine key parameters for this nonlinear plate behavior.

For glass plates that respond linearly, the design static resistance and the effective elastic resistance are:

$$r_{eff} = r_u = C_r(t/b)^2 \text{ psi} \qquad \text{6-44a}$$

The center deflection is:

$$X_u = C_D\ (b^2/t) \qquad \text{6-44b}$$

Coefficients for computing the effective resistance, C_r, and the center deflection, C_D, are listed in Table 6-8.

The fundamental period of vibration is :

$$T_N = C_T \ (b^2/t) \tag{6-45}$$

Coefficient C_T is reported in the last column of Table 6-8. Proceed to Step 10 to determine blast capacity.

Step 2 For nonlinear behavior with the b/t-ratio greater than specified in column 2 of Table 6-8, determine the nondimensional design stress, S_{SD}, as:

$$S_{ND} = 0.0183 \ (b/t)^2 \tag{6-46}$$

where: b = short span of glass measured between center lines of the gaskets (inches), and

t = actual thickness of glass, from Table 6-7 (inches)

For values of a/b greater than 4, use a/b = 4.

Step 3 Enter Figure 6-49 with the values of S_{ND} and a/b to determine the nondimensional design load, L_{ND}.

Step 4 Compute static design resistance as:

$$r_u = 876{,}000 \ (L_{ND})(t/b)^4 \text{ psi} \tag{6-47}$$

Use this value for frame design calculations other than those for rebound. Use r_{eff} defined in Step 7 for the rebound phase.

Step 5 Use a/b and L_{ND} in Figure 6-50 to obtain the nondimensional deflection, X/t. If X/t exceeds 10, use the value of L_{ND} corresponding to an X/t of 10 and recalculate r_u using Figure 6-49.

Step 6 Determine center deflection of the glass pane as:

$$X_u = (X/t)t, \text{ inches} \tag{6-48}$$

Step 7 Determine the effective elastic, static design resistance as:

$$r_{eff} = 0.4 \ (r_1 + r_2 + r_3 + r_4 + 0.5r_u) \text{ psi} \tag{6-49}$$

where: r_1 = resistance at 0.2 X_u
r_2 = resistance at 0.4 X_u
r_3 = resistance at 0.6 X_u
r_4 = resistance at 0.8 X_u
r_u obtained in Step 4
X_u obtained in Step 6

Figures 6-49 and 6-50 should be used for r_1 through r_4. The equivalent static design load is the resistance that a linearly responding plate would exhibit at the same strain energy as the nonlinearly responding plate at the design center deflection X_u. It is always less than r_u. By this technique, the linear response charts can be used with reasonable accuracy.

<u>Step 8</u> **Determine effective stiffness as:**

$$K_E = r_{eff}/X_u, \text{ psi/in} \tag{6-50}$$

<u>Step 9</u> **Determine the fundamental period of vibration as:**

$$T_N = 2\pi\,(K_{LM}\, m \,/\, K_E)^{1/2}, \text{ msec} \tag{6-51}$$

where: $K_{LM} = 0.63 + 0.16(a/b - 1)$ $\quad 1 \le a/b \le 2$
$K_{LM} = 0.79$ $\quad a/b \ge 2$

The unit mass, m, of the glass is:

$$m = 233t, \text{ lb-ms}^2/\text{in}^3$$

<u>Step 10</u> Use Figure 3-49 of Chapter 3 with the ratio of load duration to period of vibration, T/T_N, to obtain the dynamic load factor, D_{LF}. The blast overpressure capacity of the glass pane is:

$$B = r_{eff}/D_{LF} \tag{6-52}$$

For T/T_N-ratios greater than 10, set D_{LF} equal to 2. For ratios less than 0.05, set D_{LF} equal to 0.3.

6-30. Design Criteria for Frames

6-30.1. Sealants, Gaskets, and Beads

All gaskets or beads must be at least 3/8 inch wide with a Shore "A" durometer hardness of 50 and conform to ASTM Specification C509-84 (Cellular Elastomeric Preformed Gasket Sealing Material). The bead and sealant must form a weatherproof seal.

6-30.2. Glazing Setting

Minimum frame edge clearances, face clearance, and bite (Figure 6-51) are specified in Table 6-7.

6-30.3. Frame Loads

The window frame must develop the static design strength, r_u of the glass pane (Table 6-6). Otherwise, failure will occur at less than the predicted blast pressure capacity of the window pane. This results from the frame deflections which induce higher principal tensile stresses in the pane, thus reducing the strain energy capacity available to resist the blast loading.

In addition to the load transferred to the frame by the glass, frame members must also resist the static design load, r_u, applied to all exposed members. Maximum allowable limits for frame design are:

1. <u>Deflection:</u> Relative displacements of frame members shall be the smallest of 1/264th of its span or 1/8 inch.

2. Stress: The maximum stress in any member shall not exceed $f_m/1.65$, where f_m is the yield stress of the members material.

3. Fasteners: The maximum stress in any fastener shall not exceed $f_m/2.00$.

The design loads for the glazing are based on large deflection theory, but the resulting design loads transferred to the frame are based on small deflection theory for normally loaded plates. Analysis indicates this approach to be considerably simpler and more conservative than using the frame loading based exclusively on large deflection plate behavior, characteristic of window panes. The effect of the static design load, r_u, applied directly to the exposed frame members of width, w, should also be considered.

The design load, r_u, produces a line shear, V_x, applied by the long side, a, equal to:

$$V_x = C_x\, r_u\, b \sin(\pi x/a) + r_u\, w, \text{ lb/in} \qquad 6\text{-}53$$

The design load, r_u, produces a line shear, V_y, applied by the short side, b, equal to:

$$V_y = C_y\, r_u\, b \sin(\pi y/a) + r_u\, w, \text{ lb/in} \qquad 6\text{-}54$$

The design load also produces a corner concentrated load, R, tending to uplift the corners of the window pane equal to:

$$R = C_R\, r_u\, b^2, \text{ lb} \qquad 6\text{-}55$$

Distribution of these forces are illustrated in Figure 6-52. Table 6-9 presents design coefficients, C_x, C_y, and C_R for practical aspect ratios. Linear interpolation can be used for aspect ratios not shown.

Although frames with mullions are included in the design criteria, it is recommended that single pane frames be used. Experience indicates that mullions complicate the design and reduce the reliability of blast-resistant frames. If mullions are used, the certification test must be conducted as the complexity of the mullion cross sections may cause some of the assumptions to be unconservative for local shear and stress concentrations. Also, economic analysis indicates that generally thicker glass will be more cost effective than the more complex mullion frame. If mullions are used, the loads from Equations 6-55 to 6-57 should be used to check the frame mullions and fasteners for compliance with deflection and stress criteria. Note that the design load for mullions is twice the load given by Equations 6-53 to 6-55 to account for the effects of two panes supported by a common mullion.

Special design considerations should be taken to assure that deflection of the building wall will not impose deflections on the frame greater than 1/264th of the length of the panes edge. When insufficient wall rigidity is provided, it is recommended that the frames be pinned at the corners to provide isolation from the walls rotation.

6-30.4. Rebound

Response to the dynamic loading will cause the window to rebound (outward deflection) after its initial positive (inward) deflection. The outward pane displacement and the stresses produced by the negative deflection must be safely resisted by the window while positive pressures act on the window. Otherwise, the window which safely resists stresses caused by inward deflections may fail in rebound while the positive pressure still acts. This can propel glass fragments into the structure. However, if the window fails in rebound during the negative phase of blast loading, glass fragments will be drawn away from the structure. Rebound will occur during the negative loading phase if the effective blast duration is no greater than one half of the natural period of vibration, T_N, of the glass pane. For $T \geq 10\ T_N$, significant rebound does not occur during the positive blast loading, so, for this situation, design for rebound is not required. For $0.5 \leq T/T_N \leq 10$, the frame must be designed for the peak negative pressure acting during the positive overpressure phase. Table 6-11 reports T_N for practical glass pane dimensions.

As the rebound chart, Figure 3-268 of Chapter 3, can be unconservative for predicting maximum rebound (for glass panes), dynamic analysis using numerical integration or a more conservative simplified analysis is required. In lieu of a numerical analysis, it is conservative to set the maximum rebound load, r^-, to the static design load, r_u. The resistance function for this analysis can be generated by the Alternate Design Procedure. If the pane has a b/t ratio less than specified in Table 6-8, the pane will behave as a linear plate and Equations 6-44 can be used to determine r_u, X_u, and the resistance function. If the pane has a b/t ratio larger than specified in Table 6-8, use Steps 2 through 7 to define the resistance function. The negative resistance function is a mirror image of the positive resistance function.

The portion of the frame outboard of the glass must resist the rebound load, r^-. Use Equations 6-53 to 6-55 to apply the rebound load to the frame members. In some design situations the resistance built into the member outboard of the glass to resist the corner concentrated force, R (Equation 6-55), during deflections of the pane inward will provide enough strength to resist rebound.

6-31. Acceptance Test Specification

Certification tests of the entire window assembly are required unless analysis demonstrates that the window design is consistent with the design criteria. All window assembly designs using mullions must be tested. The certification tests consist of applying static uniform loads on at least two sample window assemblies until failure occurs in either the glass or frame. Although at least two static uniform load failure tests are required, the acceptance criteria presented below encourages a larger number of test samples. All testing should be performed by an independent testing laboratory certified by the contracting officer.

6-31.1. Test Procedure - Window Assembly Test

The test windows (glass panes plus support frames) shall be identical in type, size, sealant, and construction to those furnished. The test frame assembly shall be secured to simulate the adjoining walls. Using either a vacuum or a liquid-filled bladder, an increasing uniform load shall be applied to the entire window assembly (glass and frame) until failure occurs in either the glass or frame. Failure shall be defined as either breaking of glass or loss of frame resistance. The failure load shall be recorded to three significant figures. The load should be applied at a rate of 0.5 r_u per minute which corresponds to approximately one minute of significant stress duration until failure. Table 6-6 presents the static ultimate resistance of old tempered glass, correlated with a probability of failure, equal to 0.001 and a load duration of 1 second, whereas this criteria is for a load duration of 1 minute. This longer duration will weaken the glass by ceramic fatigue, but new glass should tend to be stronger than old glass. To account for these effects the certified static load capacity, r_s, of a glass pane is to be rated as:

$$r_s = 0.876\ r_u \qquad 6\text{-}56$$

6-31.2. **Acceptance Criteria**

The window assembly (frame and glazing) are considered acceptable when the arithmetic mean of all the samples tested, r, is such that:

$$\bar{r} \geq r_u + s\ \alpha \qquad 6\text{-}57$$

where

r_u = static ultimate resistance of the glass pane
s = sample standard deviation
α = acceptance coefficient

For n test samples, $\bar{r}$ is defined as:

$$\bar{r} = \frac{\sum_{i=1}^{n} r_i}{n} \qquad 6\text{-}58$$

where r_i is the recorded failure load of the i^{th} test sample. The standard sample deviation, s, is defined as:

$$s = \left[\frac{\sum_{i=1}^{n} \left[r_i - \bar{r} \right]^2}{(n-1)} \right]^{1/2} \qquad 6\text{-}59$$

The minimum value of the sample standard deviation, s, permitted to be employed in equation 6-47 is:

$$s_{min} = 0.145\ r_u \qquad 6\text{-}60$$

This assures a sample standard deviation which is no better than the ideal tempered glass in ideal frames.

The acceptance coefficient, α, is tabulated in Table 6-10 in terms of the number of samples tested.

The following equation is presented to aid in determining if additional test samples are justified. If:

$$\bar{r} \leq r_u + s\,\beta \qquad \text{6-61}$$

then with 90% confidence, the design will not prove to be adequate with additional testing. The rejection coefficient, β, is from Table 6-10.

6-31.3. Certification for Rebound

Acceptance testing shall be performed for rebound unless analysis demonstrates that the frame meets rebound criteria. All frames with mullions must be tested. Testing is performed with the load applied to the inboard surface of the window assembly. The equivalent static rebound load, r^-, is substituted for the design load, r_u.

6-32. Installation Inspection

A survey of glazing failures due to wind load indicates that improper installation of setting blocks, gaskets, or lateral shims, or poor edge bite is a significant cause of the failures experienced.

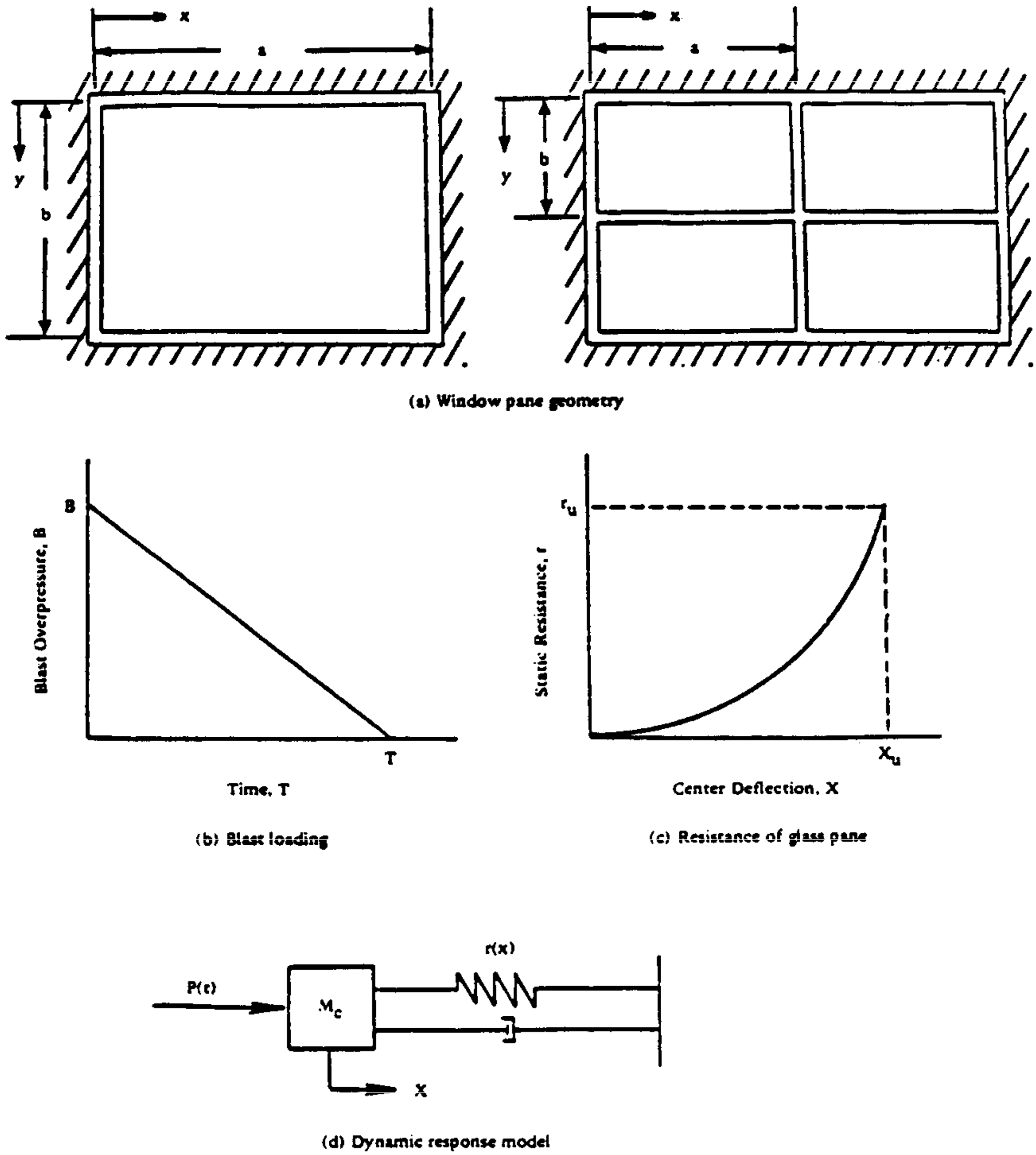


Figure 6-27 Characteristic parameters for glass pane, blast loading, resistance function and response model.

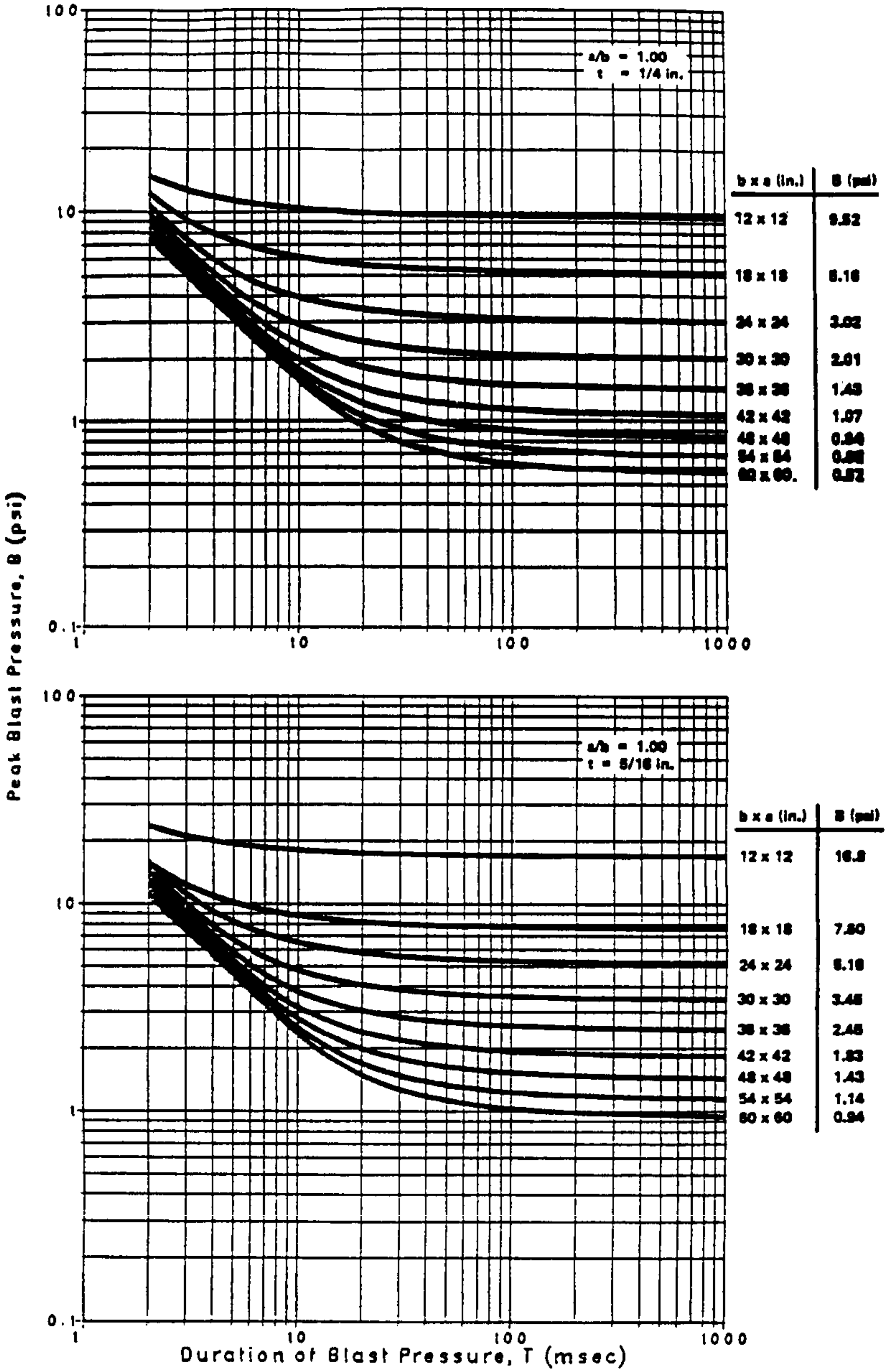


Figure 6-28 Peak blast pressure capacity for tempered glass panes: a/b = 1.00, t = 1/4 and 5/16 in.

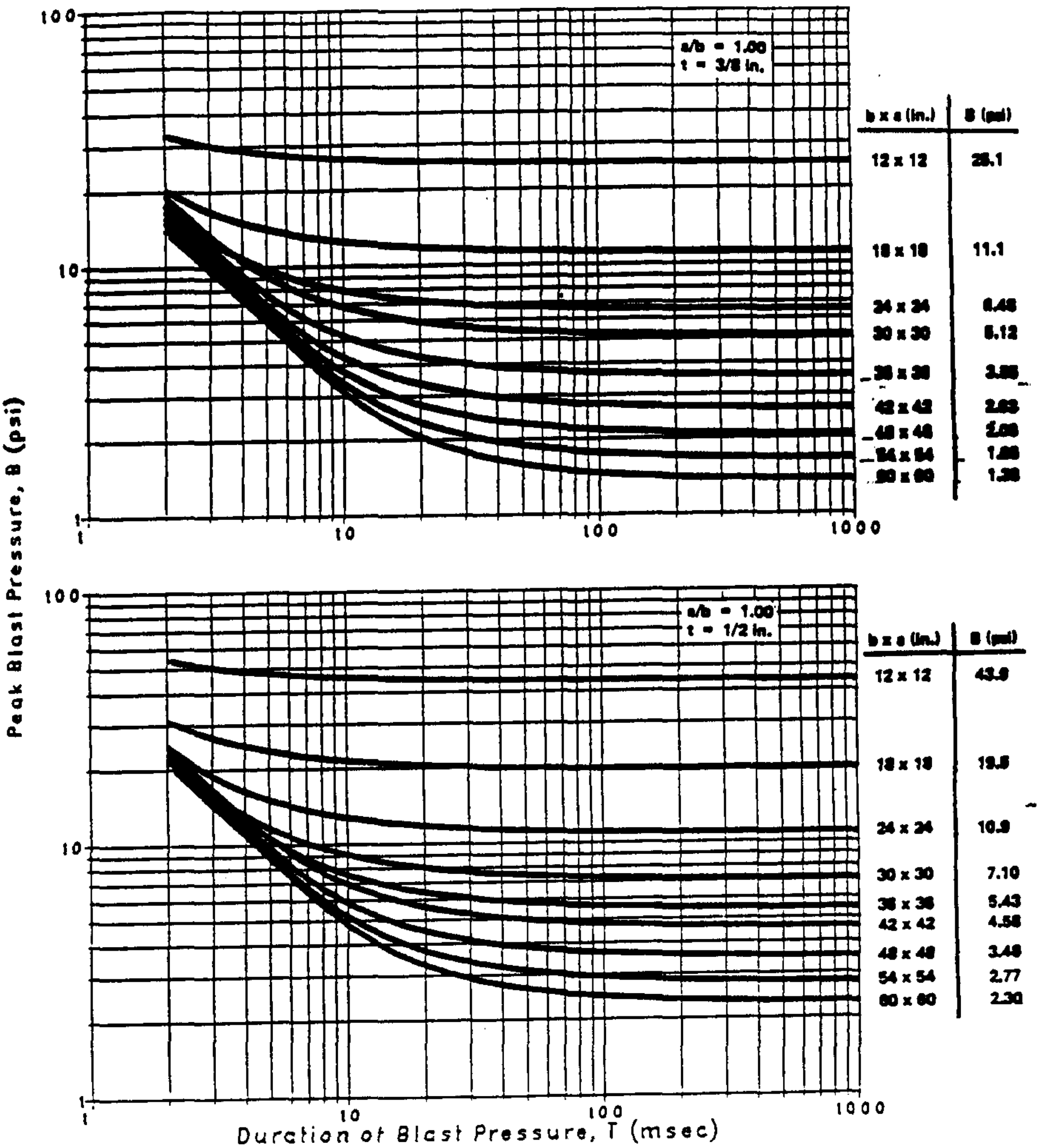


Figure 6-29 Peak blast pressure capacity for tempered glass panes: a/b = 1.00, t = 3/8 and 1/2 in.

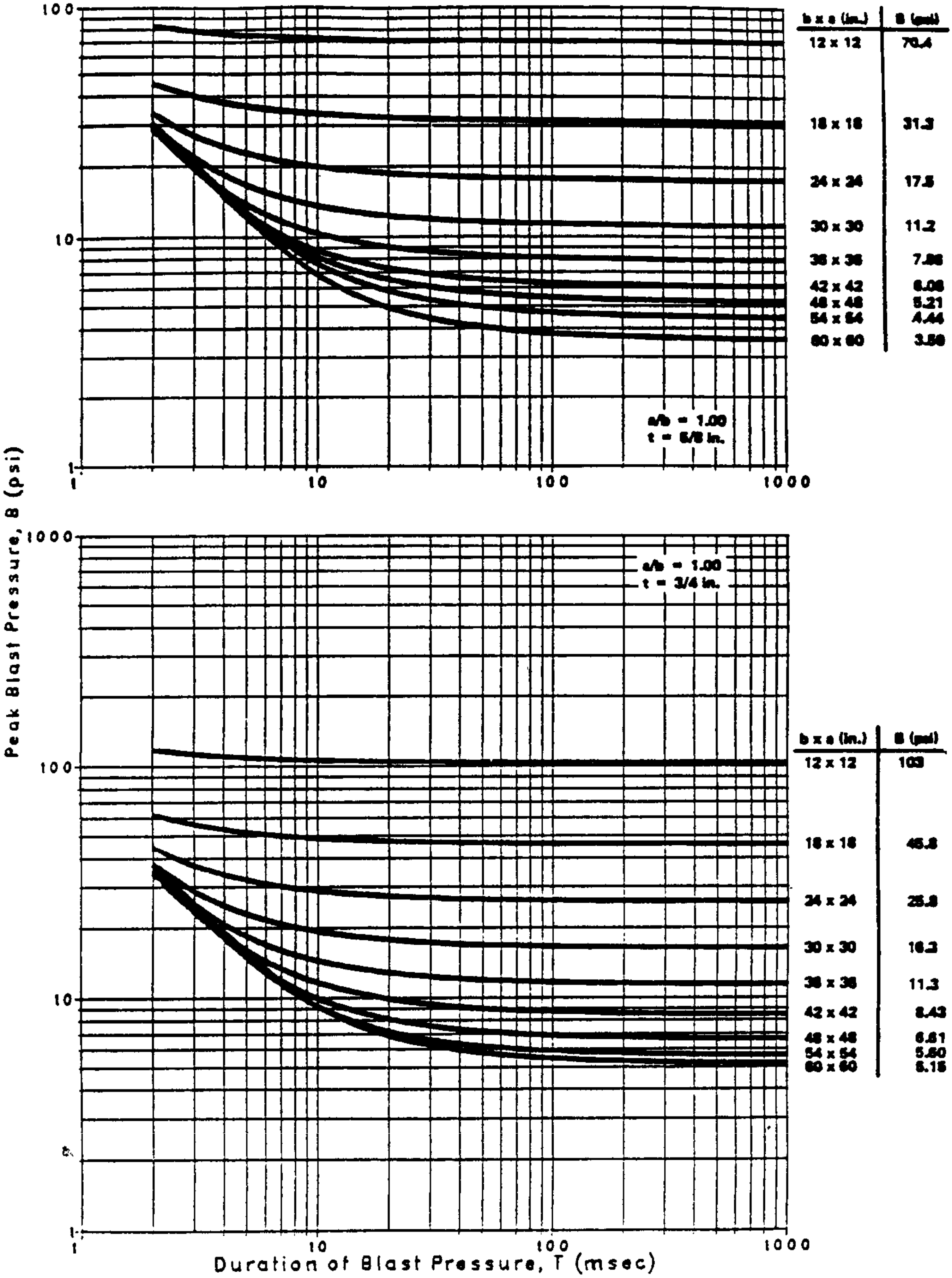


Figure 6-30 Peak blast pressure capacity for tempered glass panes: a/b = 1.00, t = 5/8 and 3/4 in.

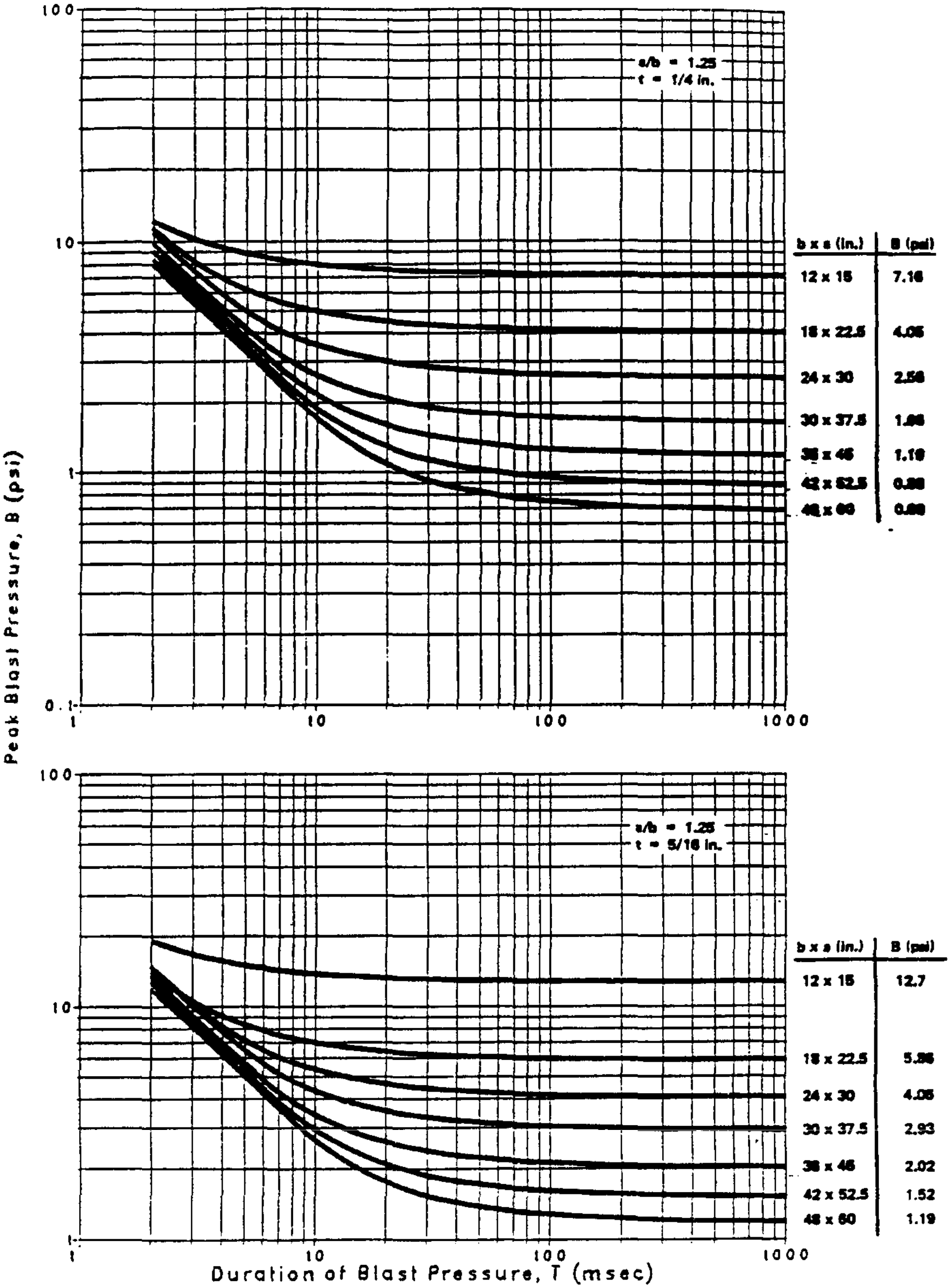


Figure 6-31 Peak blast pressure capacity for tempered glass panes: a/b = 1.25, t = 1/4 and 5/16 in.

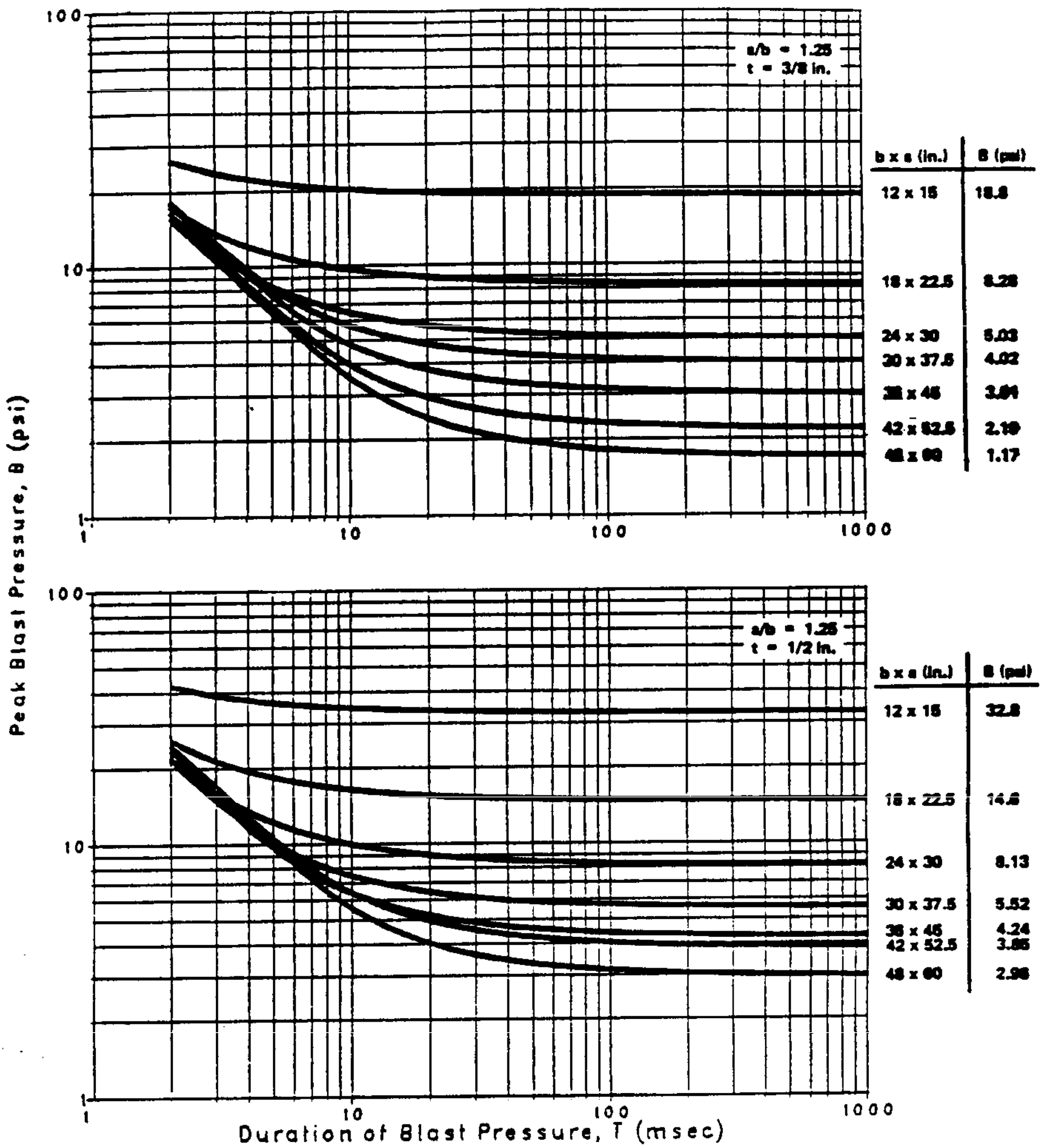


Figure 6-32 Peak blast pressure capacity for tempered glass panes: a/b = 1.25, t = 3/8 and 1/2 in.

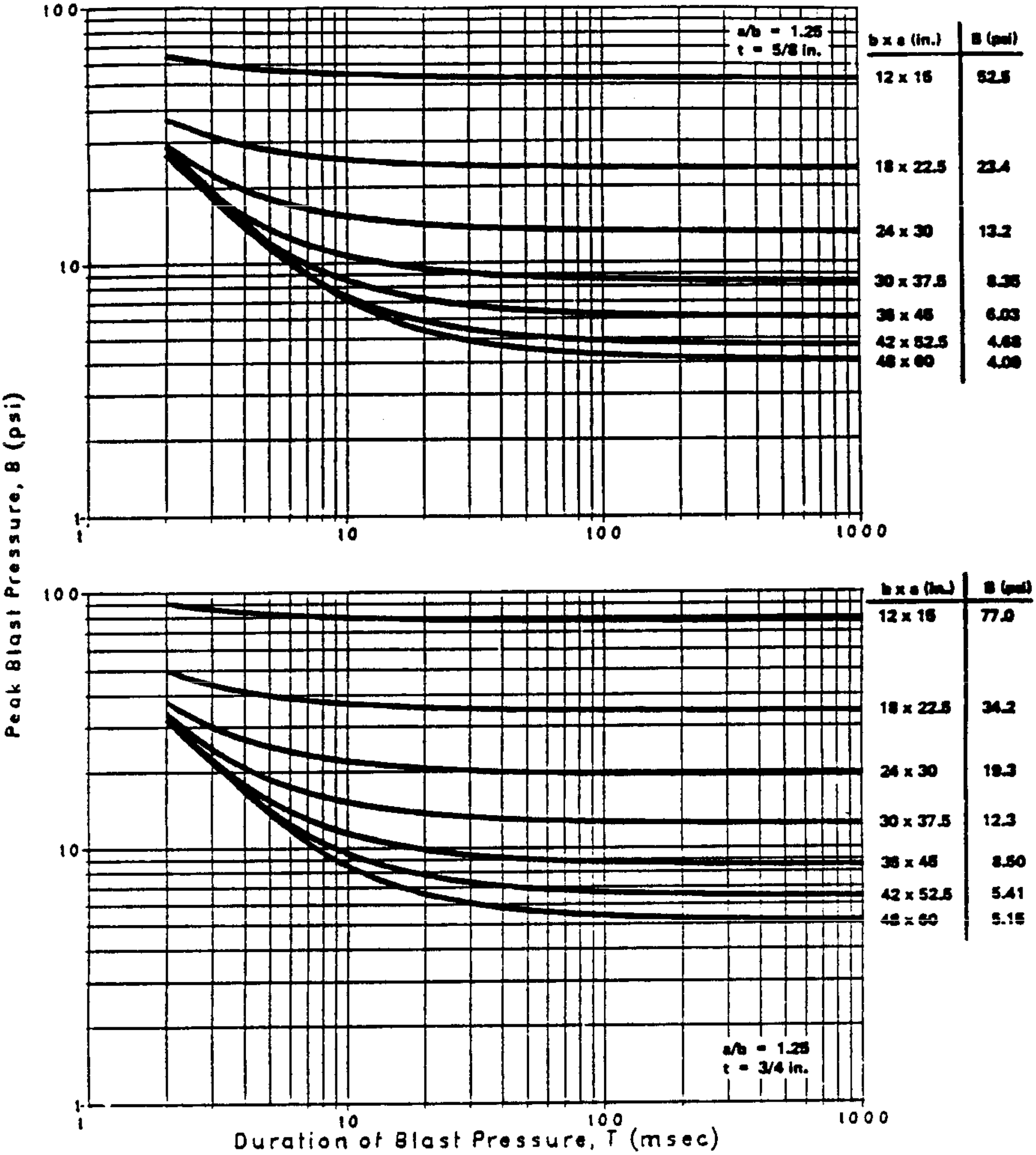


Figure 6-33 Peak blast pressure capacity for tempered glass panes: a/b = 1.25, t = 5/8 and 3/4 in.

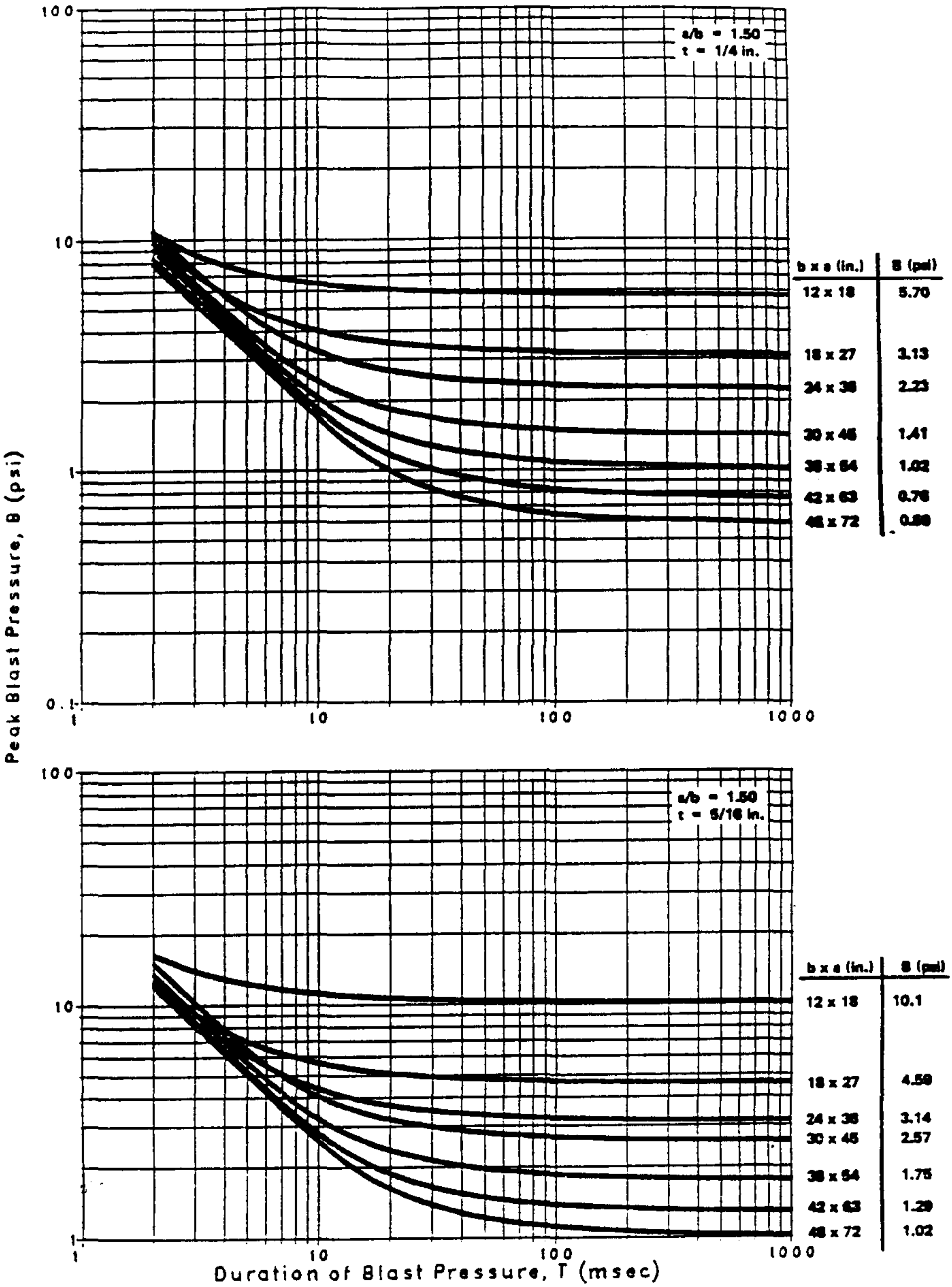


Figure 6-34 Peak blast pressure capacity for tempered glass panes: a/b = 1.50, t = 1/4 and 5/16 in

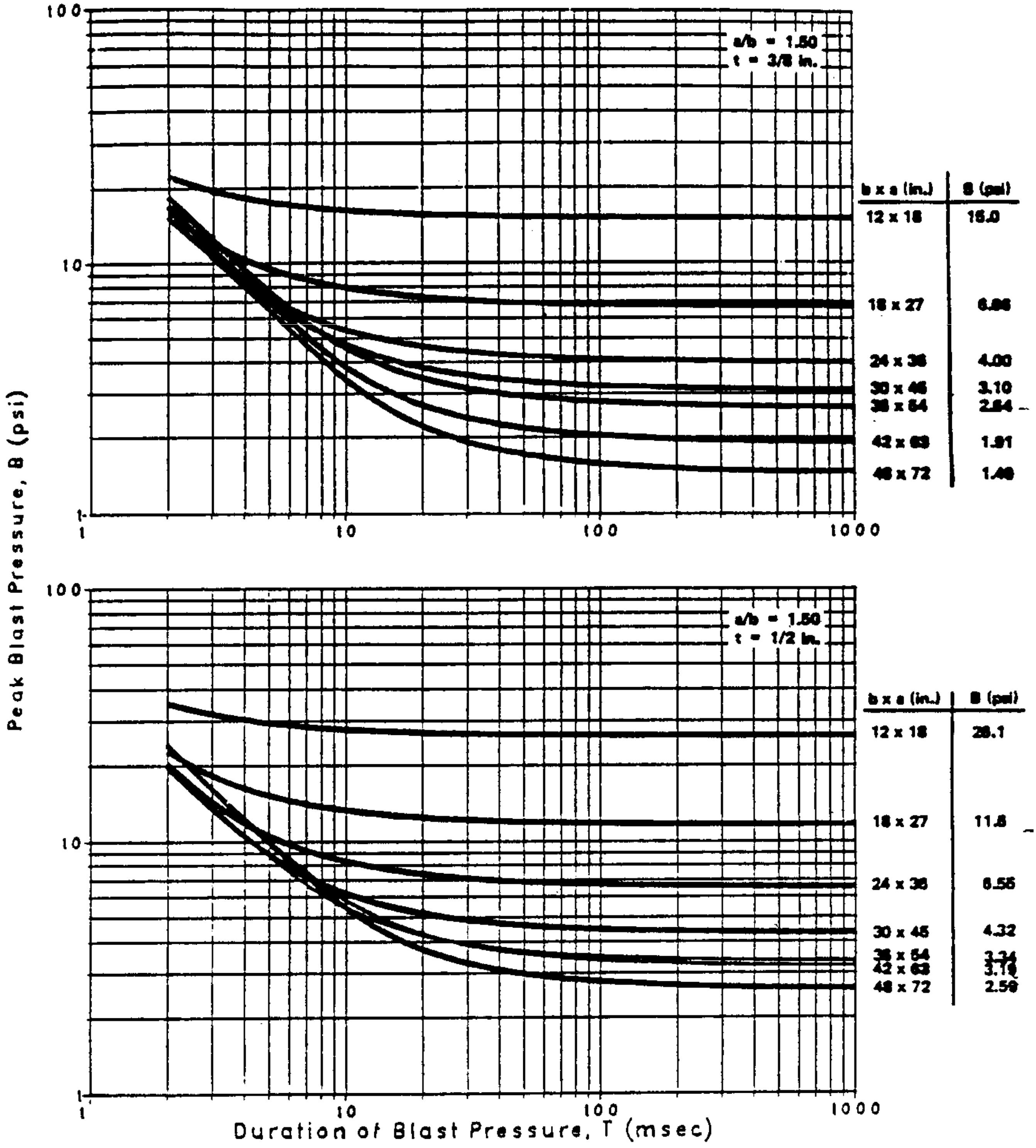


Figure 6-35 Peak blast pressure capacity for tempered glass panes: a/b = 1.50, t = 3/8 and 1/2 in.

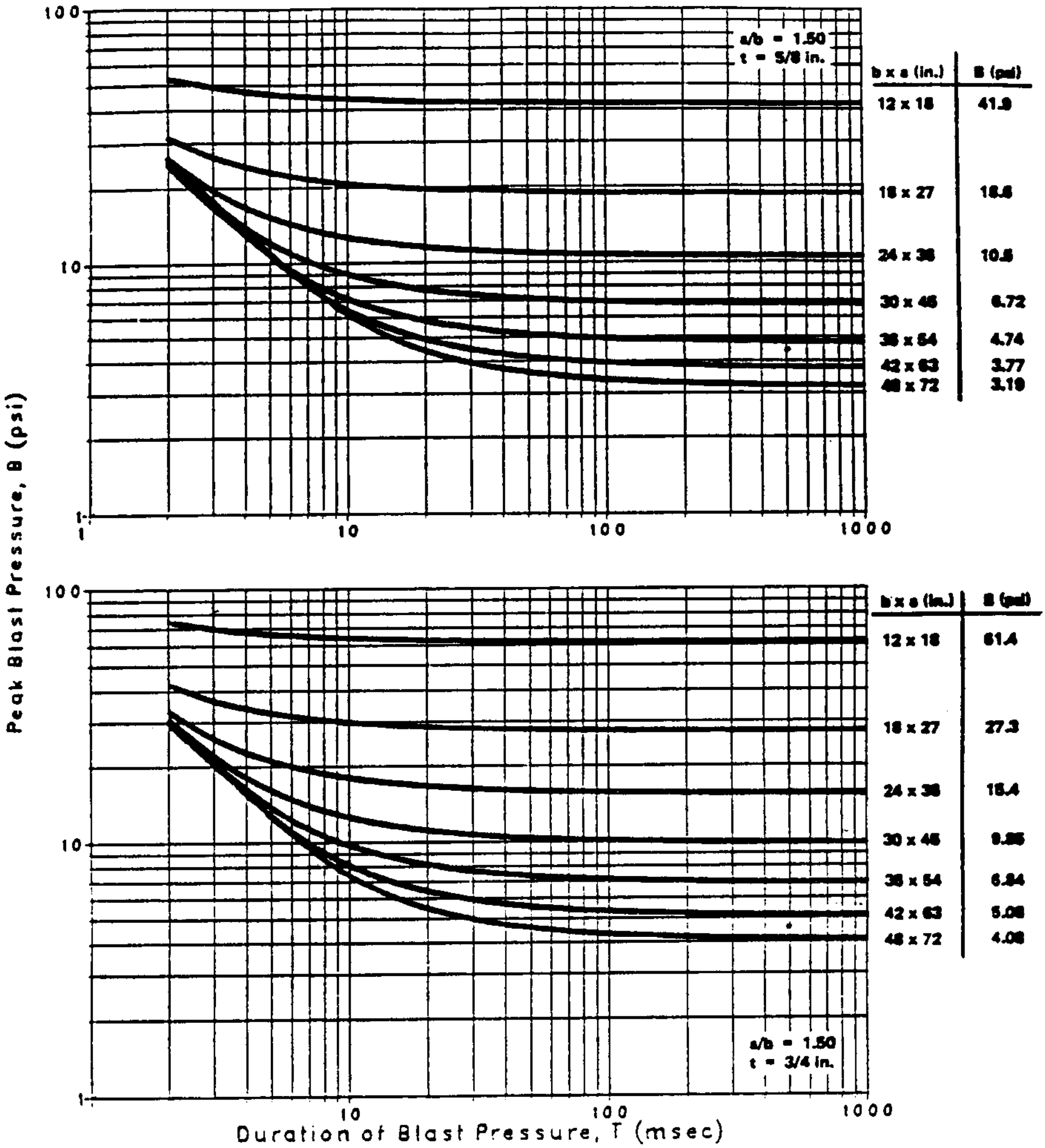


Figure 6-36 Peak blast pressure capacity for tempered glass panes: a/b = 1.50, t = 5/8 and 3/4 in.

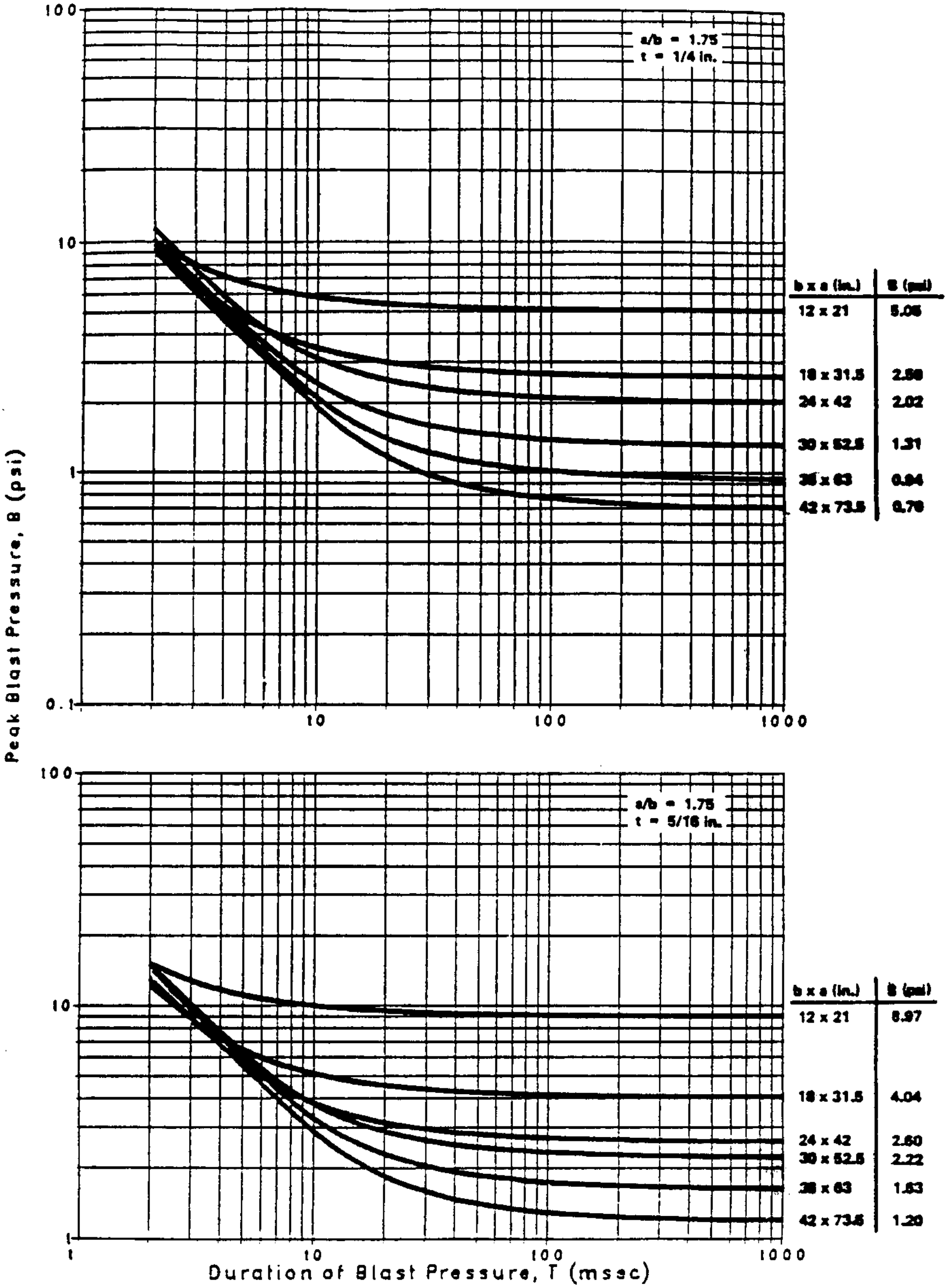


Figure 6-37 Peak blast pressure capacity for tempered glass panes: a/b = 1.75, t = 1/4 and 5/16 in.

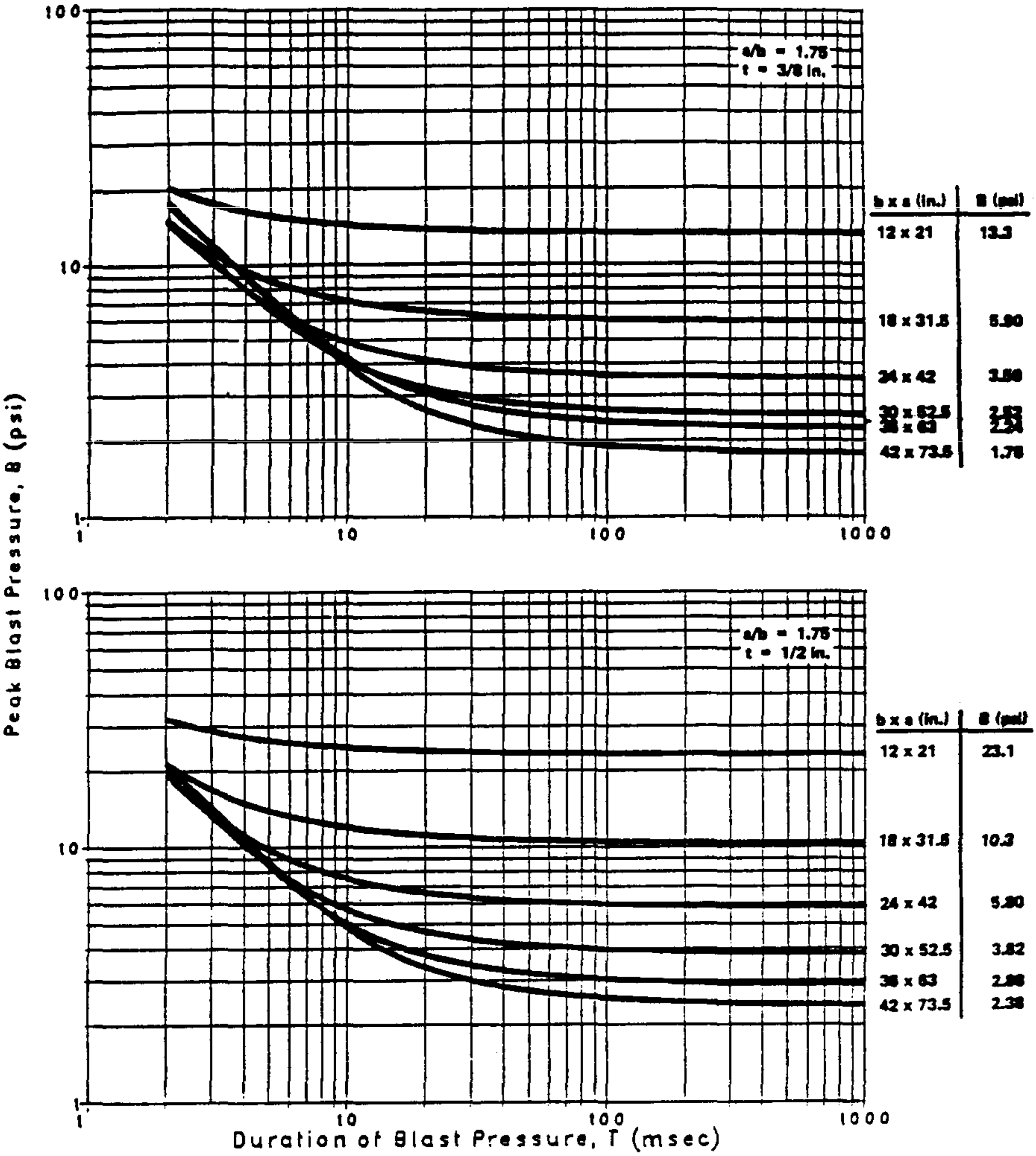


Figure 6-38 Peak blast pressure capacity for tempered glass panes: a/b = 1.75, t = 3/8 and 1/2 in.

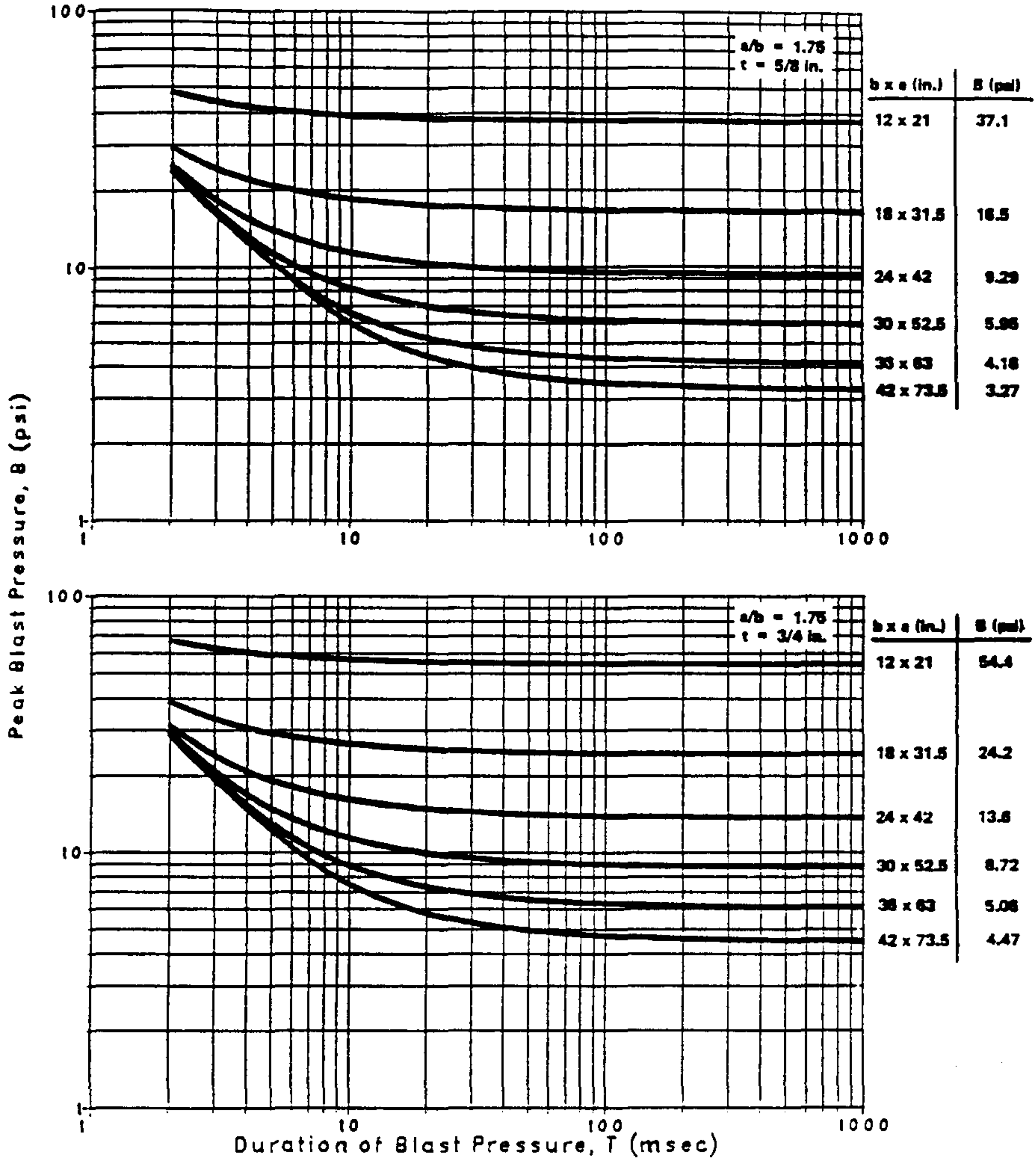


Figure 6-39 Peak blast pressure capacity for tempered glass panes: a/b = 1.75, t = 5/8 and 3/4 in.

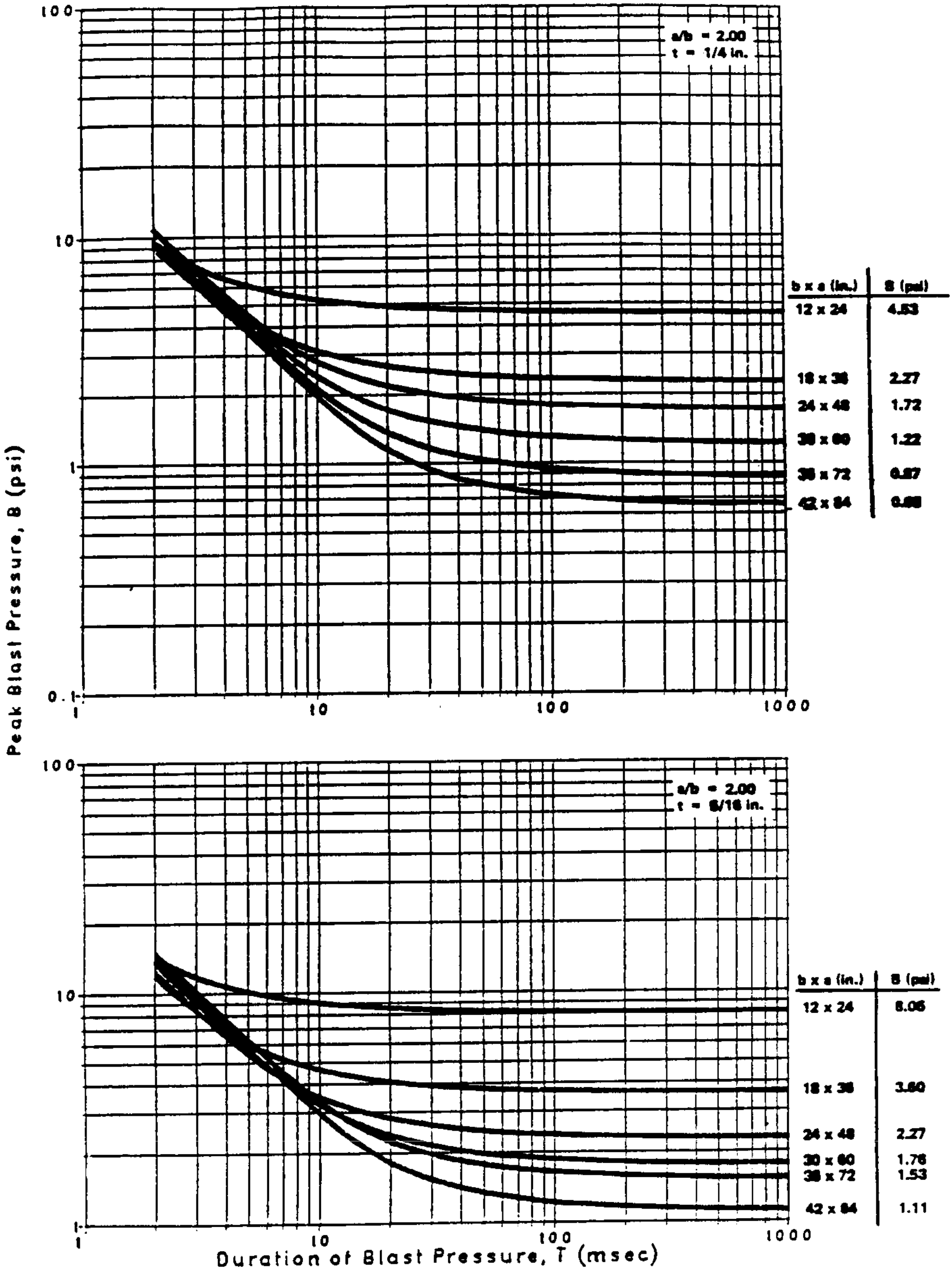


Figure 6–40 Peak blast pressure capacity for tempered glass panes: a/b = 2.00, t = 1/4 and 5/16 in.

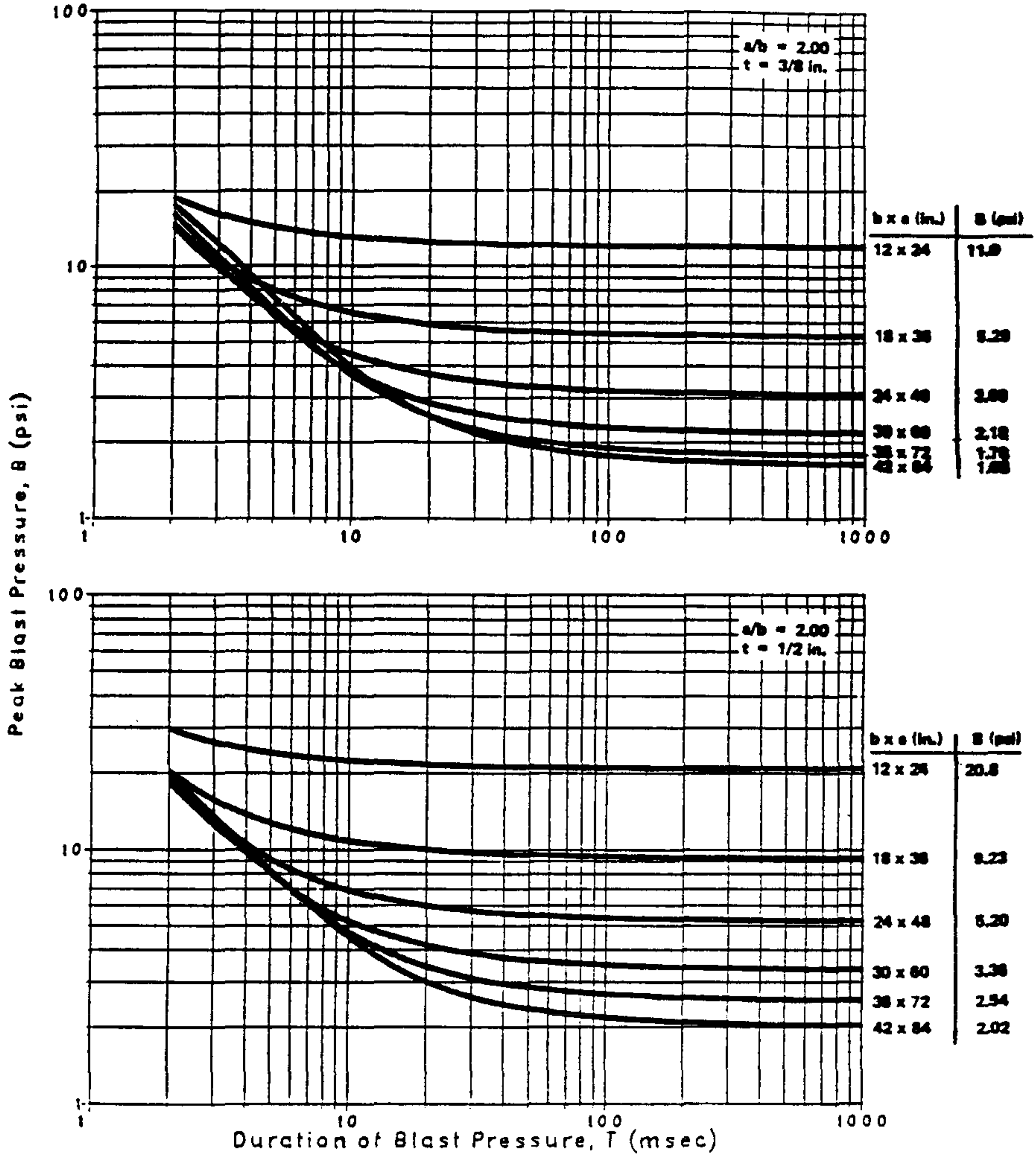


Figure 6-41 Peak blast pressure capacity for tempered glass panes: a/b = 2.00, t = 3/8 and 1/2 in.

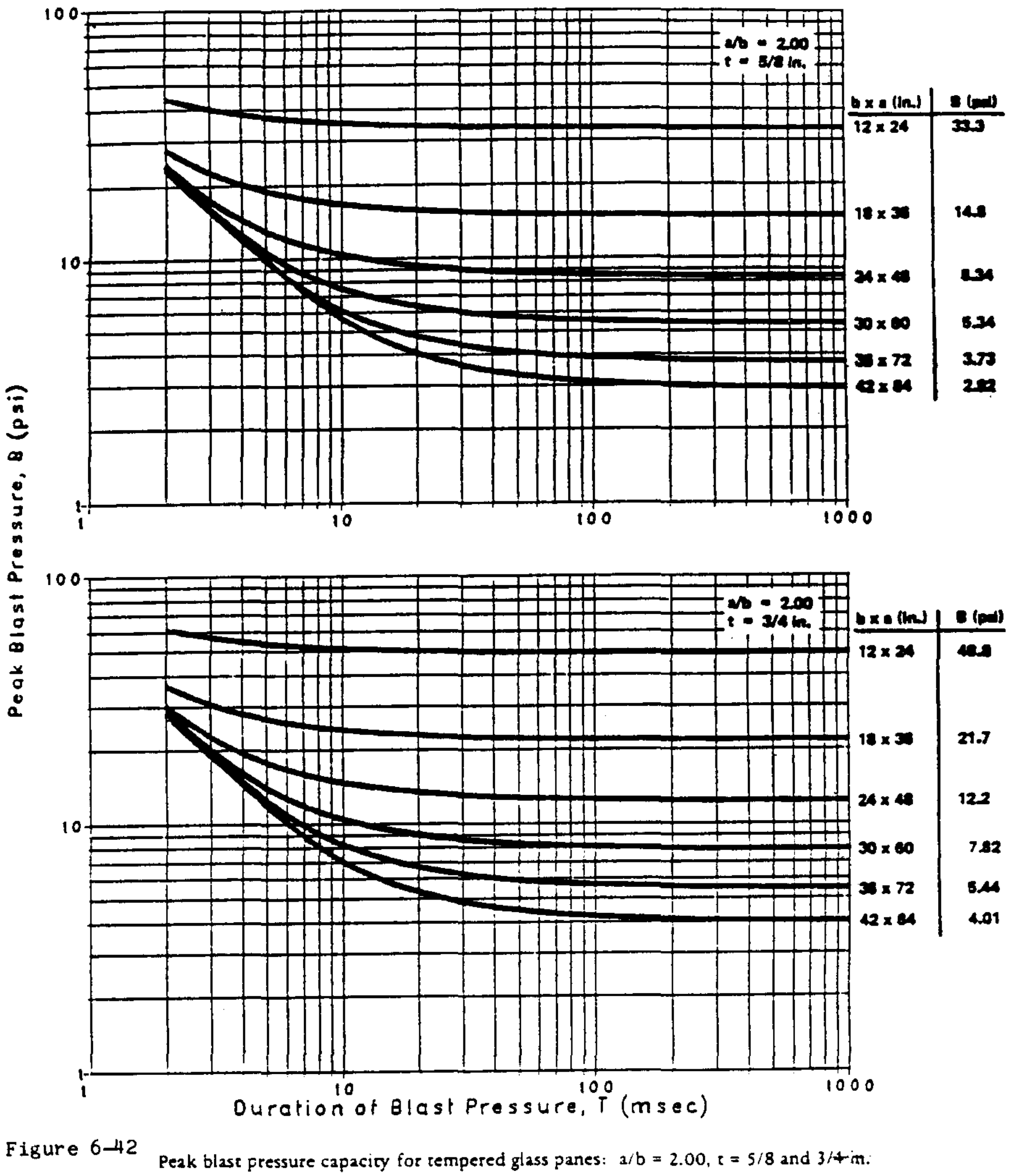


Figure 6-42 Peak blast pressure capacity for tempered glass panes: a/b = 2.00, t = 5/8 and 3/4 in.

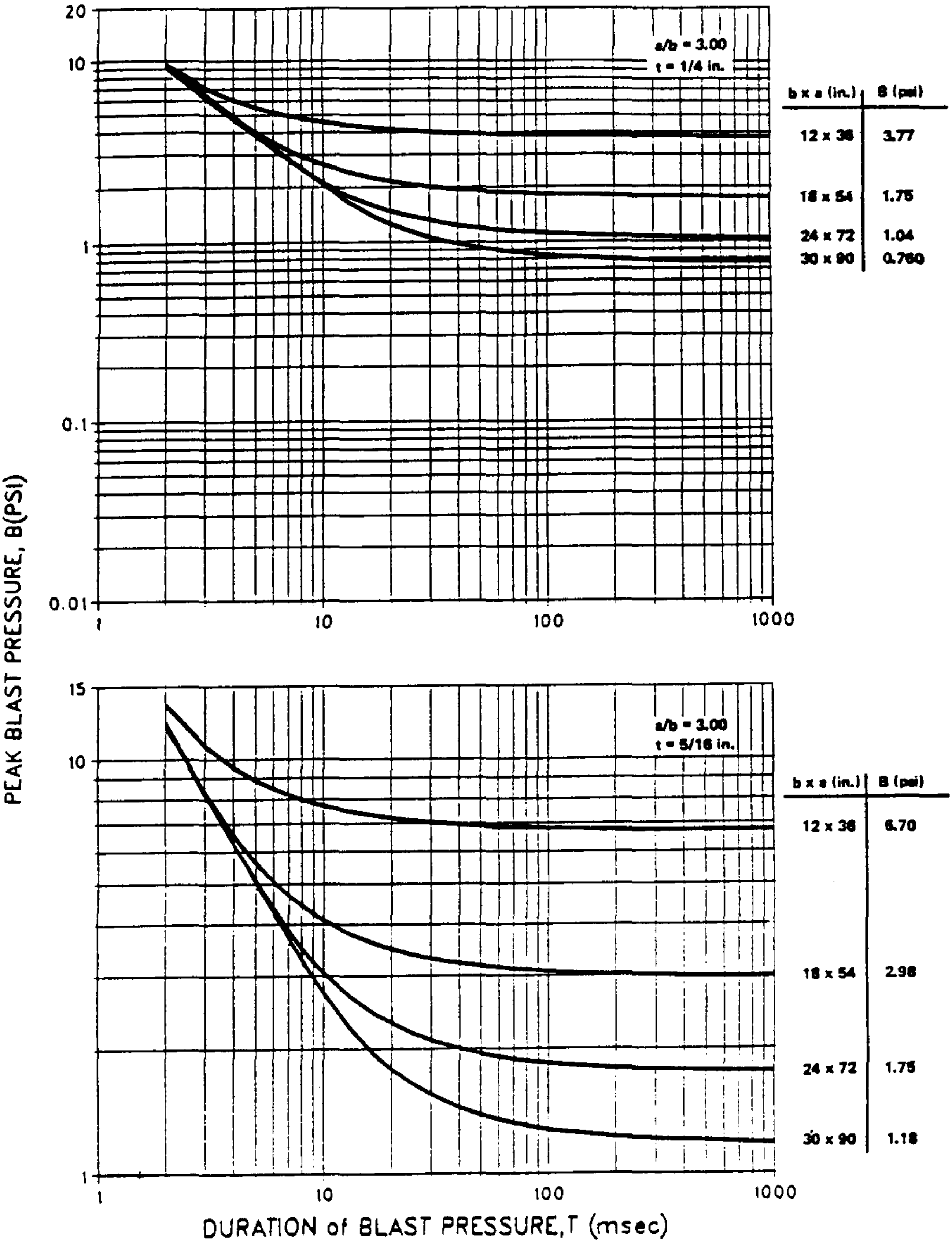


Figure 6-43 Peak blast pressure capacity for tempered glass panes: a/b = 3.00, t = 1/4 and 5/16 in.

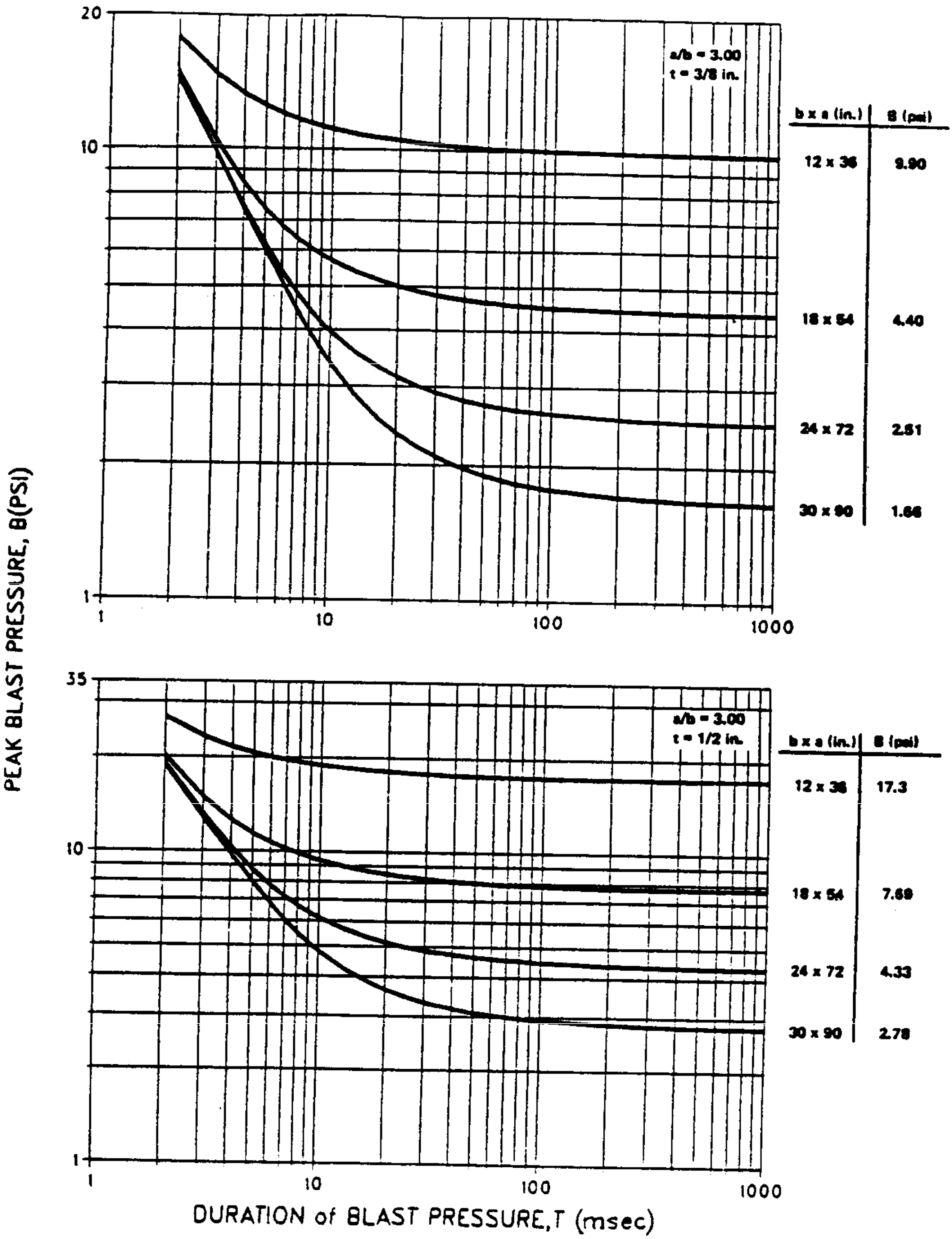


Figure 6-44 Peak blast pressure capacity for tempered glass panes: a/b = 3.00, t = 3/8 and 1/2 in.

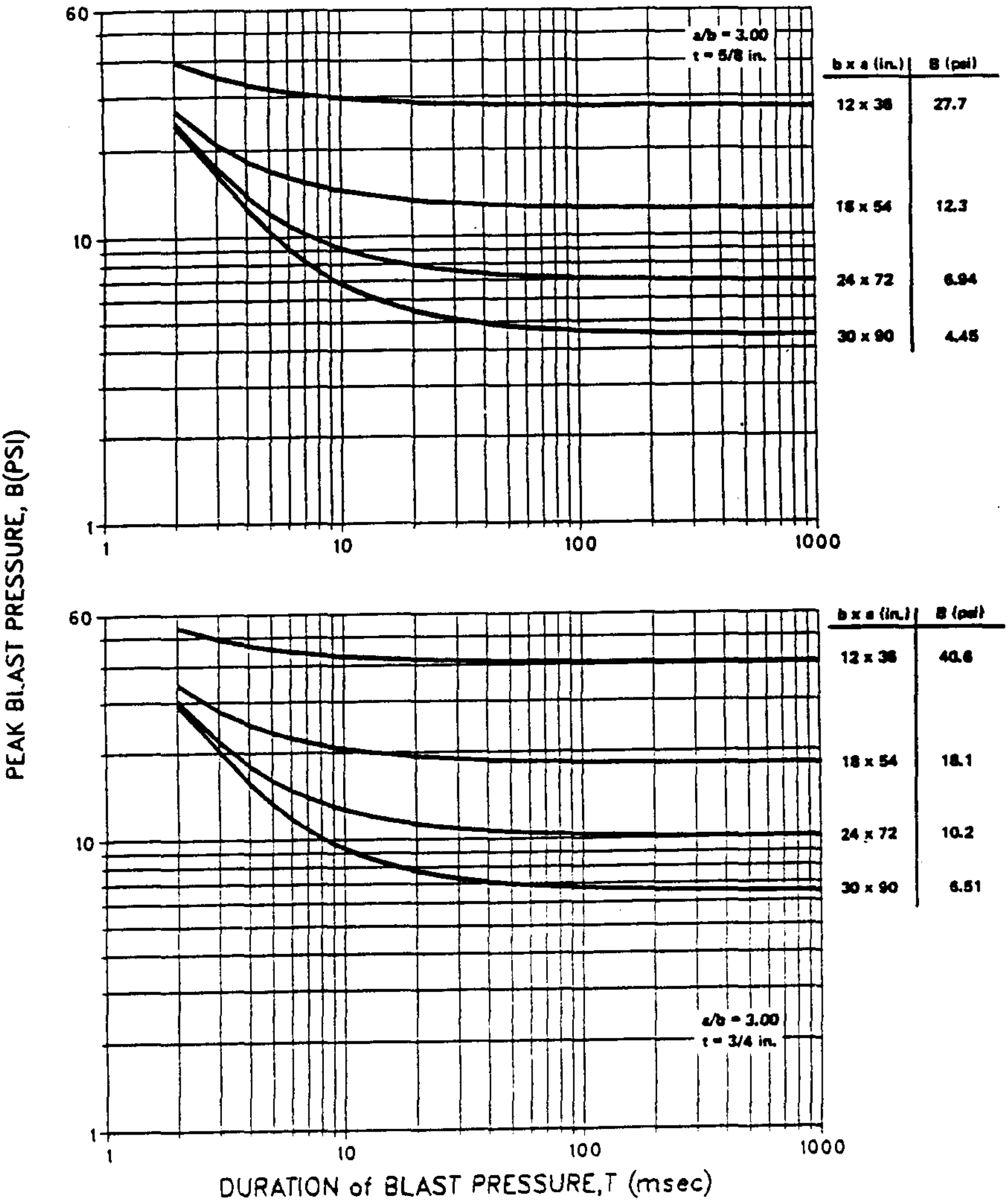


Figure 6-45 Peak blast pressure capacity for tempered glass panes: a/b = 3.00, t = 5/8 and 3/4 in.

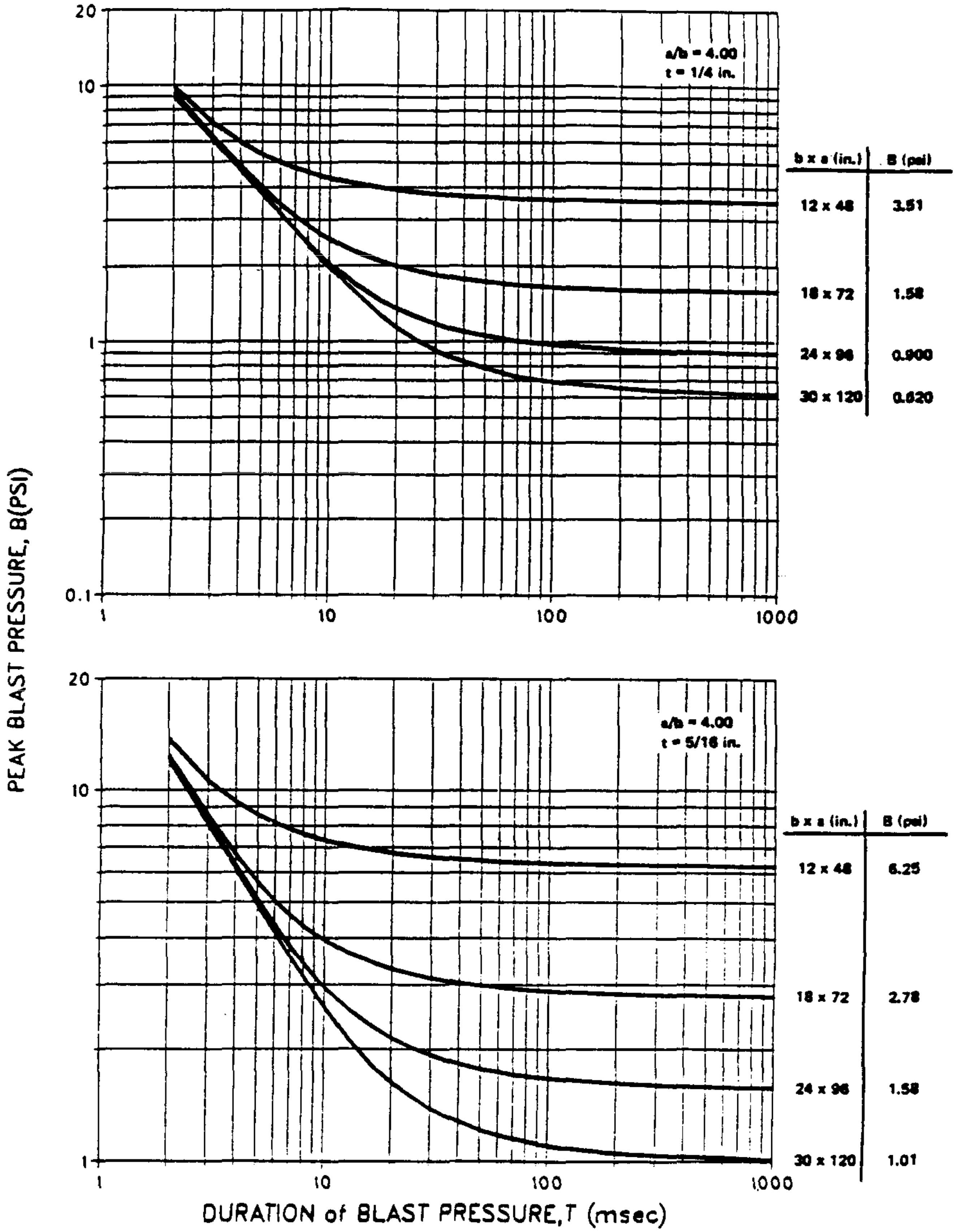


Figure 6-46 Peak blast pressure capacity for tempered glass panes: a/b = 4.00, t = 1/4 and 5/16 in.

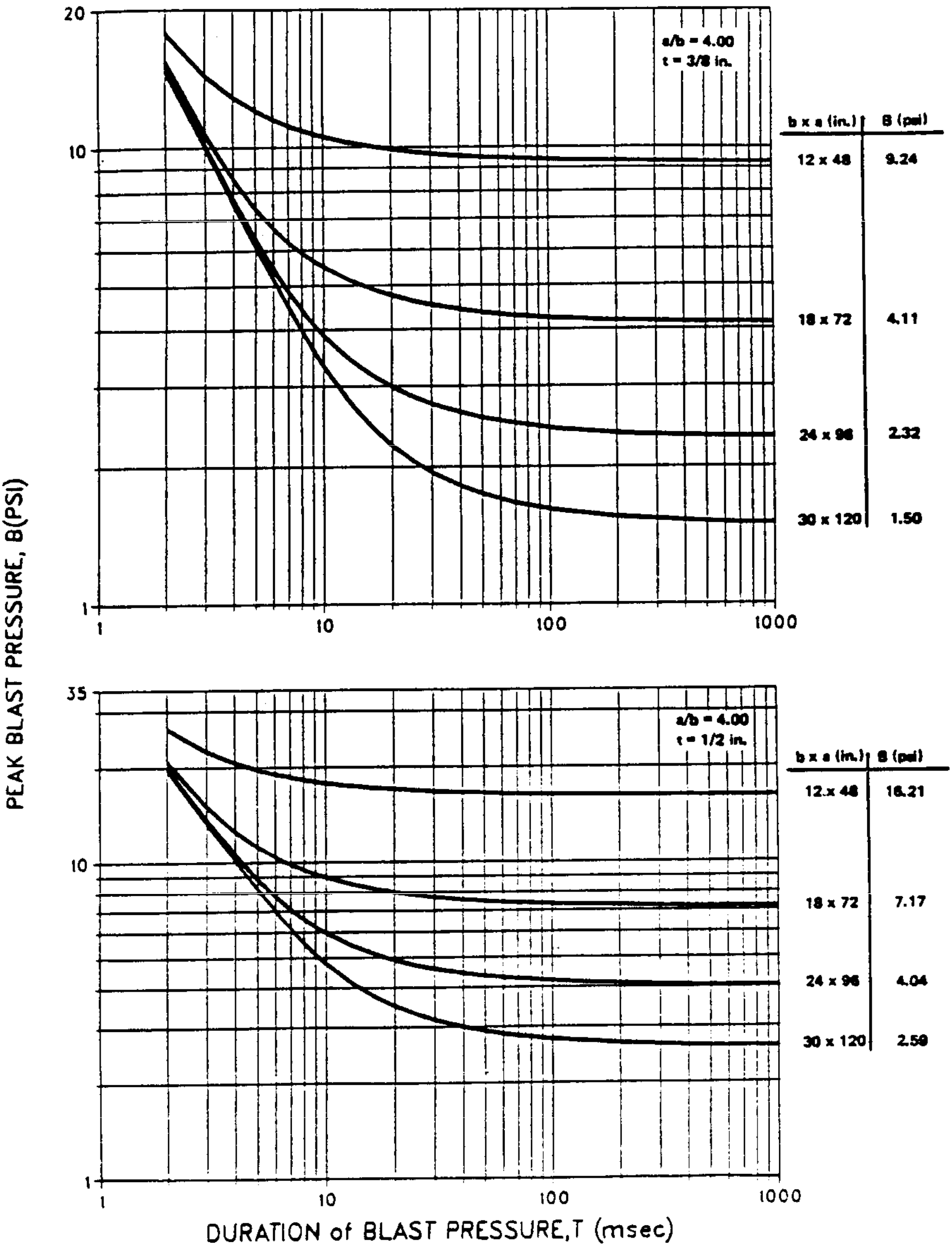


Figure 6-47 Peak blast pressure capacity for tempered glass panes: a/b = 4.00, t = 3/8 and 1/2 in.

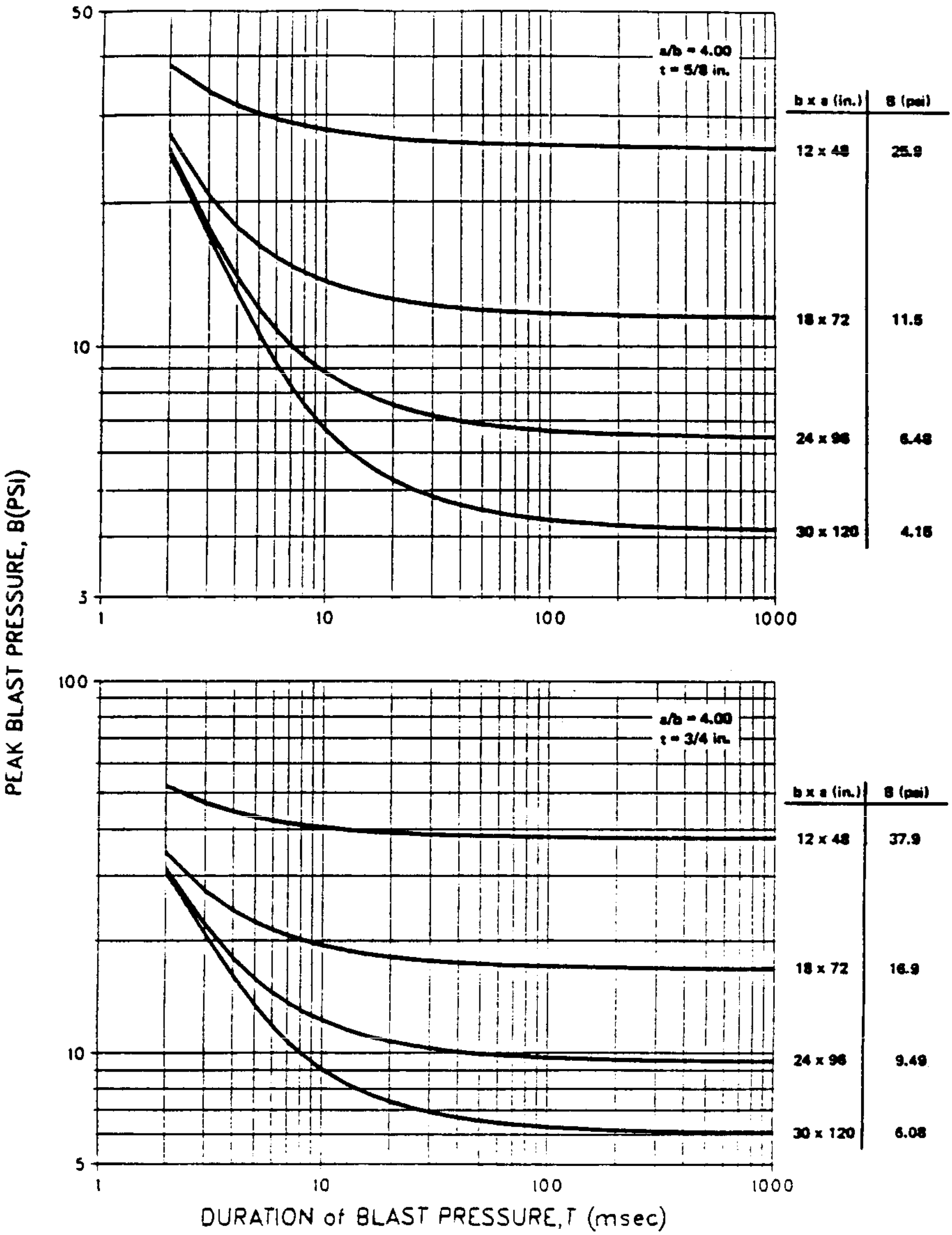


Figure 6-48 Peak blast pressure capacity for tempered glass panes: a/b = 4.00, t = 5/8 and 3/4 in.

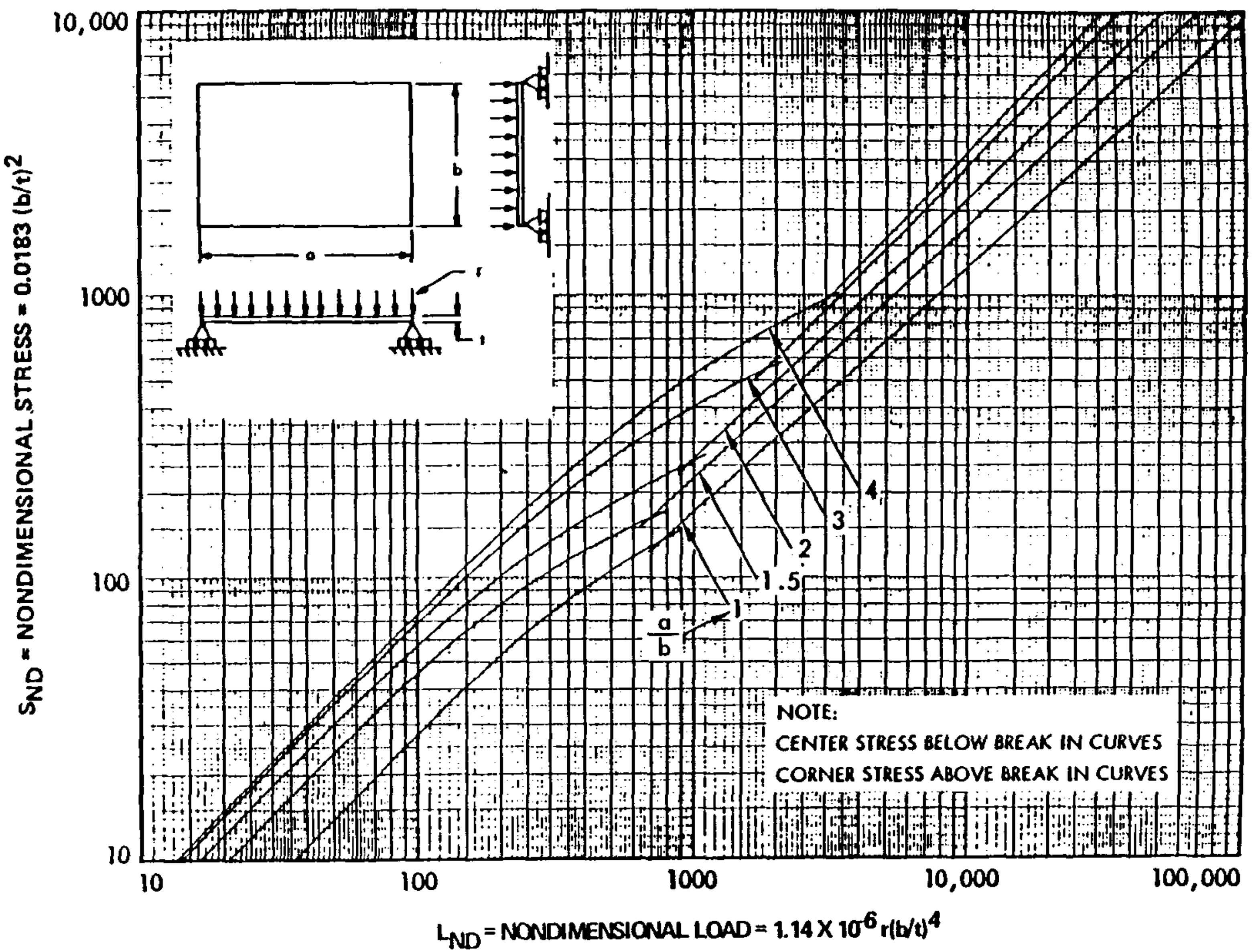


Figure 6-49 *Nondimensional static load-stress relationships for simply supported tempered glass (after Moore).*

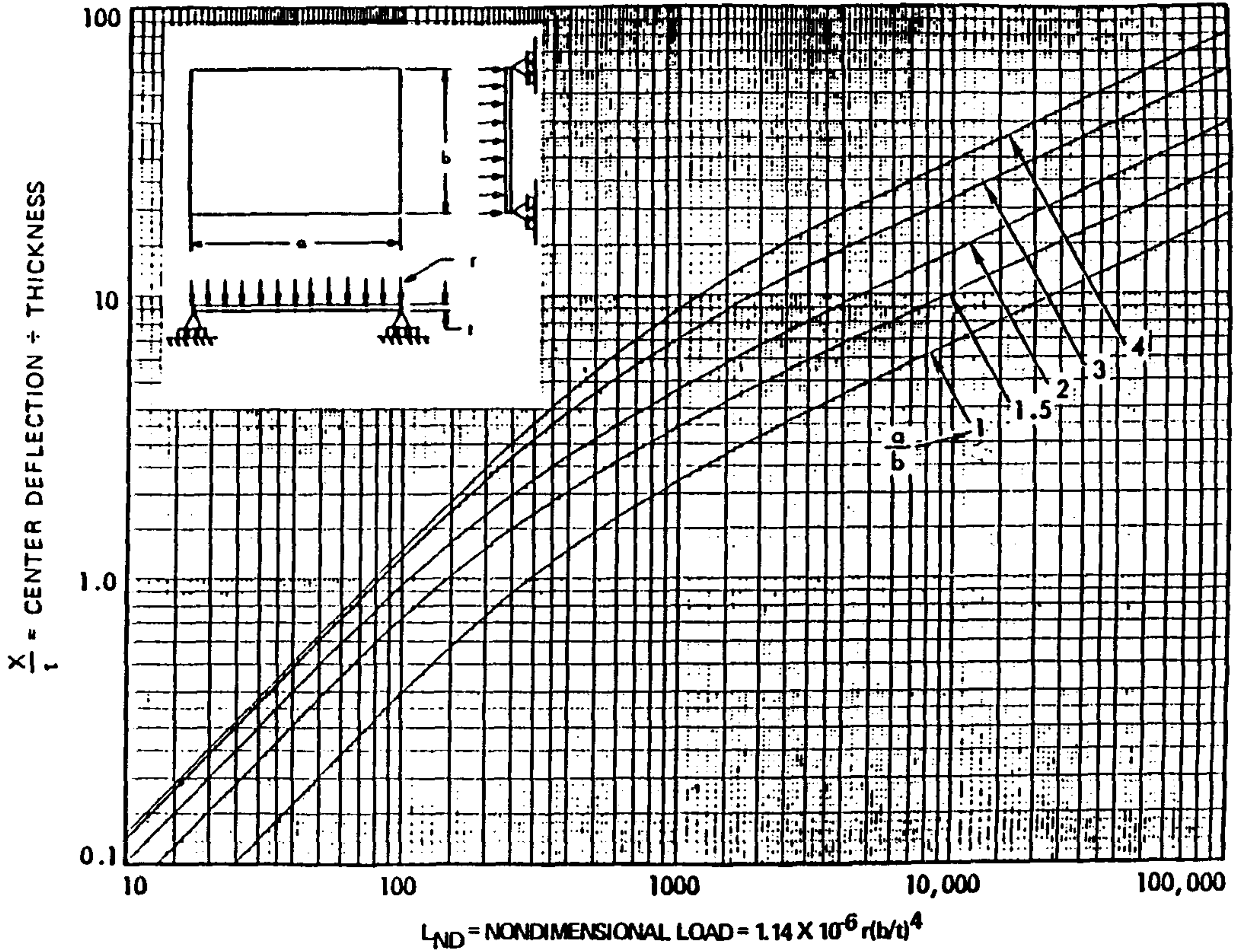


Figure 6-50 Nondimensional static load-crater deflection relationships for simply supported tempered glass (after Moore).

A - edge clearance
B - bite
C - face clearance

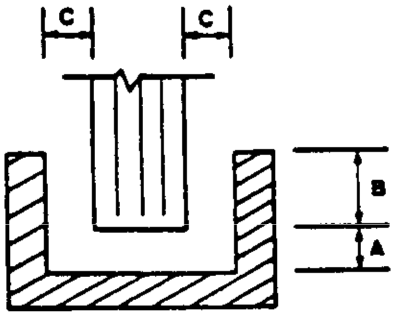


Figure 6-51 Edge, face, and bite requirements.

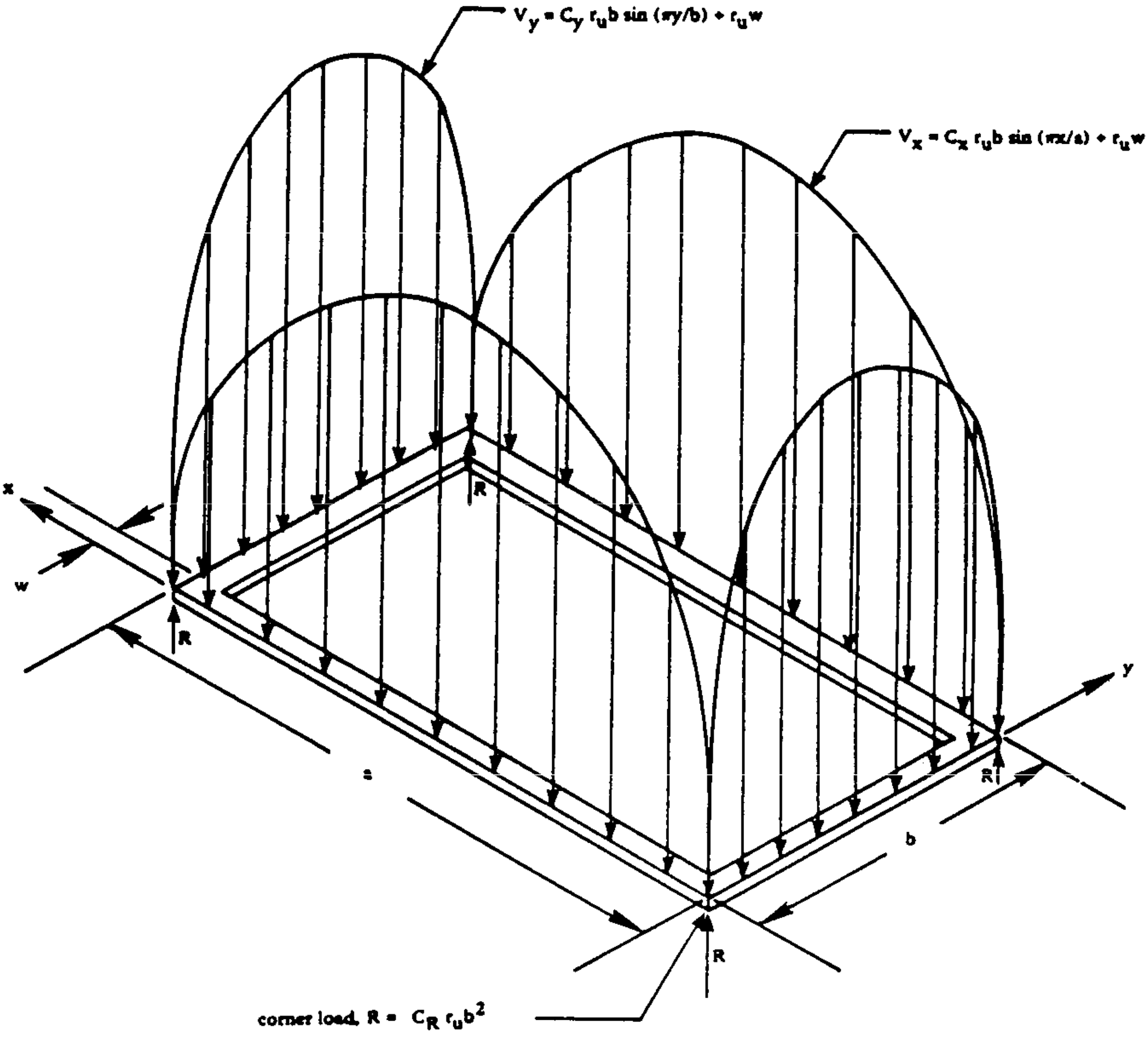


Figure 6-52 Distribution of lateral load transmitted by glass pane to the window frame.

Table 6-6 **Static Design Strength r_u (psi), for Tempered Glass* [a = long dimension of glass pane (in.); b = short dimension of glass pane (in.)]**

ASPECT RATIO = 1.00

Glass Size, b x a (in.)	Static Design Strength (psi) for a Window Thickness, t, of -- 3/4 in.	5/8 in.	1/2 in.	3/8 in.	5/16 in.	1/4 in.
12x12	206	141	87.7	40.3	27.5	20.2
13x13	176	120	74.7	42.8	23.9	17.6
14x14	151	103	64.5	36.9	21.1	15.5
15x15	132	90.1	56.1	32.2	18.7	14.2
16x16	116	79.2	49.3	28.3	16.7	13.4
17x17	103	70.1	43.7	25.5	15.1	12.7
18x18	91.6	62.5	39.0	23.1	14.1	12.6
19x19	82.2	56.1	35.0	21.0	13.5	12.1
20x20	74.2	50.7	31.6	19.2	12.9	11.0
21x21	67.3	46.0	28.6	17.7	12.7	10.0
22x22	61.3	41.9	26.4	16.3	12.6	9.20
23x23	56.1	38.3	24.4	15.1	11.8	8.52
24x24	51.5	35.2	22.7	14.3	10.9	7.91
25x25	47.5	32.4	21.2	13.8	10.1	7.43
26x26	43.9	30.0	19.7	13.4	9.39	7.00
27x27	40.7	27.9	18.5	12.9	8.80	6.62
28x28	37.9	26.2	17.4	12.8	8.26	6.22
29x29	35.3	24.6	16.4	12.6	7.78	5.86
30x30	33.0	23.2	15.4	12.6	7.39	5.53
31x31	30.9	21.9	14.6	12.0	7.04	5.22
32x32	29.0	20.8	14.2	11.3	6.71	4.94
33x33	27.4	19.7	13.8	10.6	6.39	4.69
34x34	26.0	18.7	13.5	10.0	6.07	4.45
35x35	24.8	17.8	13.2	9.50	5.77	4.23
36x36	23.6	17.0	12.8	9.05	5.50	4.04
37x37	22.5	16.2	12.7	8.63	5.24	3.86
38x38	21.5	15.4	12.7	8.24	5.01	3.69
39x39	20.5	14.8	12.6	7.88	4.79	3.53
40x40	19.7	14.4	12.5	7.57	4.58	3.39
41x41	18.8	14.1	11.9	7.30	4.39	3.25
42x42	18.1	13.8	11.4	7.04	4.21	3.12
43x43	17.3	13.5	10.9	6.80	4.05	3.00
44x44	16.7	13.2	10.4	6.56	3.90	2.89
45x45	16.0	13.0	9.99	6.32	3.75	2.78
46x46	15.4	12.9	9.59	6.08	3.62	2.68
47x47	14.9	12.8	9.24	5.86	3.49	2.58
48x48	14.5	12.7	8.91	5.65	3.37	2.49
49x49	14.2	12.6	8.59	5.45	3.25	2.41
50x50	14.0	12.6	8.30	5.27	3.15	2.33
51x51	13.7	12.4	8.02	5.09	3.04	2.25
52x52	13.5	11.9	7.76	4.92	2.95	2.18
53x53	13.3	11.5	7.54	4.76	2.85	2.11
54x54	13.1	11.1	7.33	4.61	2.77	2.05
55x55	12.9	10.7	7.13	4.47	2.68	1.99
56x56	12.8	10.3	6.94	4.33	2.60	1.93
57x57	12.7	9.99	6.76	4.20	2.53	1.87
58x58	12.7	9.66	6.59	4.08	2.45	1.82
59x59	12.6	9.38	6.40	3.97	2.38	1.77
60x60	12.6	9.11	6.22	3.85	2.32	1.72

* Panes to the right and below the stepped dividing line behave according to large deflection plate theory.

(continued)

Table 6-6 Static Design Strength r_u (psi), for Tempered Glass* [a = long dimension of glass pane (in.); b = short dimension of glass pane (in.)] (Continued)

ASPECT RATIO = 1.25

Glass Size, b x a (in.)	Static Design Strength (psi) for a Window thickness, t, of -- 3/4 in.	5/8 in.	1/2 in.	3/8 in.	5/16 in.	1/4 in.
12x15	154	105	65.5	37.5	20.5	15.4
13x16.25	131	89.5	55.8	32.0	17.9	13.5
14x17.5	113	77.2	48.1	27.6	15.8	12.3
15x18.75	98.5	67.2	41.9	24.0	14.2	11.1
16x20	86.6	59.1	36.8	21.1	13.0	10.2
17x21.25	76.7	52.4	32.6	19.0	12.0	9.86
18x22.5	68.4	46.7	29.1	17.2	11.1	9.78
19x23.75	61.4	41.9	26.1	15.8	10.3	9.72
20x25	55.4	37.8	23.6	14.6	9.96	9.54
21x26.25	50.3	34.3	21.4	13.6	9.79	8.69
22x27.5	45.8	31.3	19.7	12.8	9.78	7.95
23x28.75	41.9	28.6	18.2	12.0	9.70	7.30
24x30	38.5	26.3	17.0	11.3	9.49	6.75
25x31.25	35.5	24.2	15.9	10.7	8.77	6.27
26x32.5	32.8	22.4	14.9	10.2	8.14	5.83
27x33.75	30.4	20.8	14.1	9.98	7.57	5.45
28x35	28.3	19.5	13.4	9.80	7.07	5.12
29x36.25	26.4	18.4	12.8	9.79	6.63	4.81
30x37.5	24.6	17.4	12.3	9.77	6.23	4.54
31x38.75	23.1	16.4	11.7	9.71	5.87	4.31
32x40	21.7	15.6	11.2	9.65	5.54	4.09
33x41.25	20.4	14.9	10.7	9.22	5.25	3.90
34x42.5	19.4	14.2	10.3	8.71	4.98	3.71
35x43.75	18.5	13.6	10.1	8.24	4.74	3.53
36x45	17.6	13.1	9.93	7.81	4.52	3.36
37x46.25	16.8	12.7	9.80	7.41	4.32	3.20
38x47.5	16.1	12.3	9.79	7.05	4.14	3.06
39x48.75	15.5	11.8	9.78	6.72	3.97	2.92
40x50	14.8	11.4	9.75	6.42	3.82	2.80
41x51.25	14.3	11.0	9.71	6.13	3.66	2.68
42x52.5	13.8	10.6	9.66	5.87	3.51	2.57
43x53.75	13.4	10.3	9.47	5.63	3.37	2.47
44x55	13.0	10.2	9.06	5.40	3.24	2.37
45x56.25	12.6	10.0	8.68	5.19	3.11	2.28
46x57.5	12.3	9.87	8.32	5.00	3.00	2.19
47x58.75	11.9	9.80	7.99	4.81	2.89	2.11
48x60	11.5	9.79	7.67	4.64	2.78	2.03
49x61.25	11.2	9.78	7.37	4.49	2.68	1.96
50x62.5	10.9	9.77	7.11	4.34	2.59	1.89
51x63.75	10.6	9.74	6.85	4.21	2.50	1.83
52x65	10.3	9.70	6.61	4.08	2.42	1.77
53x66.25	10.2	9.67	6.38	3.96	2.34	1.71

* Panes to the right and below the stepped dividing line behave according to large deflection plate theory.

(continued)

Table 6-6 Static Design Strength r_u (psi), for Tempered Glass* [a = long dimension of glass pane (in.); b = short dimension of glass pane (in.)] (Continued)

ASPECT RATIO = 1.50

Glass Size, b x a (in.)	3/4 in.	5/8 in.	1/2 in.	3/8 in.	5/16 in.	1/4 in.
	Static Design Strength (psi) for a Window thickness, t, of --					
12x18	123	83.8	52.3	29.9	16.3	11.9
13x19.5	105	71.4	44.5	25.5	13.9	10.5
14x21	90.2	61.6	38.4	22.0	12.3	9.43
15x22.5	78.6	53.6	33.4	19.2	11.1	8.88
16x24	69.1	47.2	29.4	16.8	10.0	8.26
17x25.5	61.2	41.8	26.0	14.9	9.31	8.14
18x27	54.6	37.3	23.2	13.3	8.85	8.02
19x28.5	49.0	33.4	20.8	12.3	8.84	7.90
20x30	44.2	30.2	18.8	11.4	7.83	7.78
21x31.5	40.1	27.4	17.1	10.6	7.81	7.62
22x33	36.5	24.9	15.6	9.86	7.80	7.03
23x34.5	33.4	22.8	14.2	9.32	7.77	6.45
24x36	30.7	21.0	13.1	8.98	7.77	5.95
25x37.5	28.3	19.3	12.4	8.64	7.63	5.50
26x39	26.2	17.9	11.7	8.24	7.19	5.10
27x40.5	24.3	16.6	11.0	7.86	6.69	4.74
28x42	22.6	15.4	10.4	7.85	6.24	4.42
29x43.5	21.0	14.4	9.89	7.85	5.83	4.14
30x45	19.7	13.4	9.42	7.84	5.47	3.88
31x46.5	18.4	12.8	9.16	7.83	5.13	3.64
32x48	17.3	12.2	8.91	7.82	4.83	3.43
33x49.5	16.2	11.6	8.65	7.72	4.55	3.27
34x51	15.3	11.1	8.34	7.62	4.30	3.13
35x52.5	14.4	10.6	8.05	7.28	4.07	3.00
36x54	13.6	10.2	8.02	6.90	3.85	2.87
37x55.5	13.0	9.78	7.99	6.55	3.66	2.74
38x57	12.5	9.42	7.96	6.22	3.47	2.61
39x58.5	12.0	9.21	7.93	5.92	3.33	2.50
40x60	11.6	9.01	7.91	5.64	3.21	2.39
41x61.5	11.2	8.82	7.88	5.38	3.09	2.29
42x63	10.8	8.60	7.85	5.13	2.98	2.19
43x64.5	10.4	8.35	7.77	4.91	2.88	2.10
44x66	10.1	8.12	7.69	4.70	2.77	2.02
45x67.5	9.71	7.90	7.62	4.50	2.66	1.94
46x69	9.42	7.69	7.35	4.31	2.56	1.86
47x70.5	9.25	7.62	7.06	4.14	2.47	1.79
48x72	9.08	7.55	6.78	3.97	2.38	1.73

* Panes to the right and below the stepped dividing line behave according to large deflection plate theory.

(continued)

Table 6-6 Static Design Strength r_u (psi), for Tempered Glass* [a = long dimension of glass pane (in.); b = short dimension of glass pane (in.)] (Continued)

ASPECT RATIO = 1.75

Glass Size, b x a (in.)	Static Design Strength (psi) for a Window thickness, t, of -- 3/4 in.	5/8 in.	1/2 in.	3/8 in.	5/16 in.	1/4 in.
12x21	109	74.2	46.3	26.5	14.4	10.2
13x22.75	92.6	63.2	39.4	22.6	12.3	8.91
14x24.5	79.9	54.5	34.0	19.5	10.6	8.01
15x26.25	69.6	47.5	29.6	17.0	9.46	7.32
16x28	61.2	41.7	26.0	14.9	8.52	6.83
17x29.75	54.2	37.0	23.0	13.2	7.85	6.36
18x31.5	48.3	33.0	20.6	11.8	7.30	5.93
19x33.25	43.4	29.6	18.5	10.6	6.88	5.76
20x35	39.1	26.7	16.7	9.71	6.49	5.73
21x36.75	35.5	24.2	15.1	8.96	6.12	5.70
22x38.5	32.4	22.1	13.8	8.36	5.84	5.68
23x40.25	29.6	20.2	12.6	7.87	5.73	5.57
24x42	27.2	18.6	11.6	7.43	5.71	5.27
25x43.75	25.1	17.1	10.7	7.11	5.70	5.00
26x45.5	23.2	15.8	9.97	6.81	5.69	4.67
27x47.25	21.5	14.7	9.37	6.52	5.66	4.35
28x49	20.0	13.6	8.83	6.24	5.45	4.05
29x50.75	18.6	12.7	8.38	5.98	5.21	3.79
30x52.5	17.4	11.9	8.01	5.82	4.98	3.56
31x54.25	16.3	11.1	7.66	5.74	4.70	3.36
32x56	15.3	10.4	7.35	5.73	4.43	3.17
33x57.75	14.4	9.93	7.11	5.71	4.17	3.00
34x59.5	13.5	9.46	6.89	5.70	3.94	2.85
35x61.25	12.8	9.02	6.67	5.69	3.73	2.72
36x63	12.1	8.62	6.45	5.67	3.54	2.60
37x64.75	11.4	8.30	6.24	5.63	3.37	2.49
38x66.5	10.8	8.00	6.04	5.44	3.21	2.37
39x68.25	10.3	7.73	5.87	5.26	3.06	2.26
40x70	9.91	7.47	5.80	5.09	2.93	2.16
41x71.75	9.52	7.26	5.74	4.93	2.82	2.06
42x73.5	9.15	7.07	5.72	4.71	2.71	1.97
43x75.25	8.81	6.90	5.70	4.50	2.61	1.89
44x77	8.50	6.73	5.69	4.30	2.51	1.81
45x78.75	8.24	6.55	5.70	4.12	2.42	1.74

* Panes to the right and below the stepped dividing line behave according to large deflection plate theory.

(continued)

Table 6-6 Static Design Strength r_u (psi), for Tempered Glass* [a = long dimension of glass pane (in.); b = short dimension of glass pane (in.)] (Continued)

ASPECT RATIO = 2.00

Glass Size, b x a (in.)	Static Design Strength (psi) for a Window thickness, t, of -- 3/4 in.	5/8 in.	1/2 in.	3/8 in.	5/16 in.	1/4 in.
12x24	97.6	66.6	41.5	23.8	13.0	9.05
13x26	83.1	56.7	35.4	20.3	11.0	7.81
14x28	71.7	48.9	30.5	17.5	9.52	6.87
15x30	62.4	42.6	26.6	15.2	8.31	6.29
16x32	54.9	37.5	23.4	13.3	7.43	5.81
17x34	48.6	33.2	20.7	11.9	6.72	5.40
18x36	43.4	29.6	18.5	10.6	6.26	5.03
19x38	38.9	26.6	16.6	9.49	5.86	4.71
20x40	35.1	24.0	14.9	8.56	5.51	4.56
21x42	31.9	21.7	13.6	7.85	5.19	4.46
22x44	29.0	19.8	12.4	7.25	4.90	4.42
23x46	26.6	18.1	11.3	6.73	4.64	4.39
24x48	24.4	16.6	10.4	6.39	4.55	4.37
25x50	22.5	15.3	9.56	6.08	4.47	4.32
26x52	20.8	14.2	8.84	5.79	4.40	4.24
27x54	19.3	13.2	8.23	5.53	4.39	4.01
28x56	17.9	12.2	7.73	5.29	4.38	3.74
29x58	16.7	11.4	7.27	5.07	4.37	3.50
30x60	15.6	10.7	6.86	4.86	4.31	3.28
31x62	14.6	9.98	6.57	4.67	4.25	3.09
32x64	13.7	9.36	6.32	4.58	4.08	2.93
33x66	12.9	8.80	6.08	4.52	3.85	2.78
34x68	12.2	8.31	5.87	4.47	3.64	2.64
35x70	11.5	7.91	5.66	4.41	3.44	2.51
36x72	10.8	7.53	5.47	4.40	3.26	2.39
37x74	10.3	7.18	5.29	4.39	3.11	2.28
38x76	9.73	6.86	5.12	4.38	2.97	2.18
39x78	9.24	6.62	4.96	4.37	2.84	2.08
40x80	8.78	6.42	4.81	4.34	2.72	1.98
41x82	8.37	6.23	4.67	4.30	2.60	1.89
42x84	8.03	6.05	4.60	4.25	2.50	1.80

* Panes to the right and below the stepped dividing line behave according to large deflection plate theory.

(continued)

Table 6-6 Static Design Strength r_u (psi), for Tempered Glass* [a = long dimension of glass pane (in.); b = short dimension of glass pane (in.)] (Continued)

ASPECT RATIO = 3.00

Glass Size, b x a (in.)	Static Design Strength (psi) for a Window thickness, t, of -- 3/4 in.	5/8 in.	1/2 in.	3/8 in.	5/16 in.	1/4 in.
12x36	81.1	55.4	34.5	19.8	10.8	7.53
13x39	69.1	47.2	29.4	16.9	9.18	6.41
14x42	59.6	40.7	25.4	14.5	7.92	5.57
15x45	51.9	35.4	22.1	12.7	6.90	4.94
16x48	45.6	31.2	19.4	11.1	6.06	4.42
17x51	40.4	27.6	17.2	9.86	5.43	3.98
18x54	36.1	24.6	15.3	8.79	4.92	3.61
19x57	32.4	22.1	13.8	7.89	4.48	3.29
20x60	29.2	19.9	12.4	7.12	4.10	3.01
21x63	26.5	18.1	11.3	6.46	3.77	2.80
22x66	24.1	16.5	10.3	5.88	3.48	2.61
23x69	22.1	15.1	9.40	5.44	3.22	2.44
24x72	20.3	13.8	8.63	5.06	3.00	2.29
25x75	18.7	12.8	7.95	4.71	2.82	2.15
26x78	17.3	11.8	7.35	4.40	2.66	2.08
27x81	16.0	10.9	6.82	4.13	2.51	2.01
28x84	14.9	10.2	6.34	3.88	2.38	1.95
29x87	13.9	9.48	5.91	3.65	2.26	1.89
30x90	13.0	8.86	5.56	3.44	2.14	1.83
31x93	12.2	8.30	5.26	3.25	2.08	1.80
32x96	11.4	7.79	4.98	3.08	2.03	1.78
33x99	10.7	7.32	4.72	2.93	1.97	1.76
34x102	10.1	6.90	4.48	2.80	1.92	1.77

* Panes to the right and below the stepped dividing line behave according to large deflection plate theory.

Table 6-6 Static Design Strength r_u (psi), for Tempered Glass* [a = long dimension of glass pane (in.); b = short dimension of glass pane (in.)] (Continued)

ASPECT RATIO = 4.00

Glass Size, b x a (in.)	Static Design Strength (psi) for a Window thickness, t, of -- 3/4 in.	5/8 in.	1/2 in.	3/8 in.	5/16 in.	1/4 in.
12x48	75.7	51.7	32.2	18.5	10.1	7.02
13x52	64.5	44.0	27.5	15.7	8.57	5.99
14x56	55.6	38.0	23.7	13.6	7.39	5.16
15x60	48.5	33.1	20.6	11.8	6.43	4.52
16x64	42.6	29.1	18.1	10.4	5.66	3.99
17x68	37.7	25.8	16.1	9.20	5.01	3.56
18x72	33.7	23.0	14.3	8.20	4.49	3.19
19x76	30.2	20.6	12.9	7.36	4.05	2.87
20x80	27.3	18.6	11.6	6.65	3.67	2.60
21x84	24.7	16.9	10.5	6.03	3.34	2.37
22x88	22.5	15.4	9.59	5.49	3.06	2.18
23x92	20.6	14.1	8.77	5.03	2.81	2.02
24x96	18.9	12.9	8.05	4.63	2.59	1.88
25x100	17.5	11.9	7.42	4.28	2.39	1.76
26x104	16.1	11.0	6.86	3.97	2.23	1.66
27x108	15.0	10.2	6.36	3.70	2.09	1.57
28x112	13.9	9.49	5.92	3.45	1.96	1.49
29x116	13.0	8.85	5.52	3.22	1.84	1.41
30x120	12.1	8.27	5.15	3.02	1.75	1.34

* Panes to the right and below the stepped dividing line behave according to large deflection plate theory.

Table 6-7 Minimum Design Thickness, Clearances, and Bite Requirements

Glass Thickness (Nominal) in	mm	Actual Glass Thickness For Design, t (in)	"A" Minimum Edge Clearance (in)	"B" Nominal Bite (in)	"C" Minimum Face Clearance (in)
5/32	4.0	0.149	3/16	1/2	1/8
3/16	5.0	0.180	3/16	1/2	1/8
1/4	6.0	0.219	1/4	1/2	1/8
3/8	10.0	0.355	5/16	1/2	3/16
1/2	12.0	0.469	3/8	1/2	1/4
5/8	16.0	0.594	3/8	1/2	1/4
3/4	19.0	0.719	3/8	1/2	5/16

Table 6-8 **Maximum (b/t) Ratio for Linear Plate Behavior Under Blast Load and Coefficients for Resistance and Deflection and Fundamental Period of Simply Supported Glass Plates Based on Small Deflection Theory (No Tensile Membrane Behavior)**

Aspect Ratio, a/b	Maximum (b/t) Ratio	Design Coefficients		
		Design Resistance, C_r	Design Deflection, C_D	Fundamental Period of Vibration, C_T
1.0	53.6	5.79×10^{4}	2.58×10^{-4}	5.21×10^{-3}
1.2	59.0	4.42×10^{4}	2.72×10^{-4}	6.30×10^{-3}
1.4	63.9	3.68×10^{4}	2.83×10^{-4}	7.21×10^{-3}
1.5	66.2	3.36×10^{4}	2.88×10^{-4}	7.60×10^{-3}
1.6	67.9	3.22×10^{4}	2.91×10^{-4}	7.99×10^{-3}
1.8	71.3	2.91×10^{4}	2.98×10^{-4}	8.65×10^{-3}
2.0	74.7	2.72×10^{4}	3.02×10^{-4}	9.23×10^{-3}
3.0	84.3	2.32×10^{4}	3.12×10^{-4}	10.12×10^{-3}
4.0	89.4	2.24×10^{4}	3.15×10^{-4}	10.36×10^{-3}
∞	89.4	2.24×10^{4}	3.15×10^{-4}	10.44×10^{-3}

Table 6-9 Coefficients for Frame Loading

a/b	C_R	C_x	C_y
1.00	0.065	0.495	0.495
1.10	0.070	0.516	0.516
1.20	0.074	0.535	0.533
1.30	0.079	0.554	0.551
1.40	0.083	0.570	0.562
1.50	0.085	0.581	0.574
1.60	0.086	0.590	0.583
1.70	0.088	0.600	0.591
1.80	0.090	0.609	0.600
1.90	0.091	0.616	0.607
2.00	0.092	0.623	0.614
3.00	0.093	0.644	0.655
4.00	0.094	0.687	0.685

Table 6-10 Statistical Acceptance and Rejection Coefficients

Number of Window Assemblies n	Acceptance Coefficient α	Rejection Coefficient β
2	4.14	.546
3	3.05	.871
4	2.78	1.14
5	2.65	1.27
6	2.56	1.36
7	2.50	1.42
8	2.46	1.48
9	2.42	1.49
10	2.39	1.52
11	2.37	1.54
12	2.35	1.57
13	2.33	1.58
14	2.32	1.60
15	2.31	1.61
16	2.30	1.62
17	2.28	1.64
18	2.27	1.65
19	2.27	1.65
20	2.26	1.66
21	2.25	1.67
22	2.24	1.68
23	2.24	1.68
24	2.23	1.69
25	2.22	1.70
30	2.19	1.72
40	2.17	1.75
50	2.14	1.77

Table 6-11 **Fundamental Period of Vibration, T_n, for Monolithic Tempered Glass**

Aspect Ratio = 1.00

Glass Dimensions (in.)		Fundamental Period of Vibration (msec) for Window Thickness, t, of --					
b	a	3/4 in.	5/8 in.	1/2 in.	3/8 in.	5/16 in.	1/4 in.
12	12	1.05	1.27	1.61	2.13	2.83	3.38
13	13	1.23	1.49	1.89	2.50	3.29	3.93
14	14	1.43	1.73	2.19	2.89	3.76	4.50
15	15	1.64	1.99	2.52	3.30	4.39	5.09
16	16	1.87	2.26	2.87	3.72	4.95	5.68
17	17	2.11	2.55	3.23	4.17	5.53	6.27
18	18	2.37	2.86	3.63	4.63	6.12	6.84
19	19	2.64	3.19	4.02	5.10	6.71	7.42
20	20	2.92	3.54	4.43	5.77	7.30	8.08
21	21	3.22	3.90	4.86	6.32	7.89	8.76
22	22	3.53	4.28	5.30	6.89	8.44	9.45
23	23	3.86	4.68	5.75	7.48	9.06	10.2
24	24	4.21	5.07	6.23	8.07	9.72	10.9
25	25	4.56	5.47	6.70	8.65	10.4	11.6
26	26	4.94	5.89	7.40	9.25	11.1	12.3
27	27	5.32	6.33	7.94	9.84	11.8	13.1
28	28	5.72	6.77	8.50	10.4	12.5	13.8
29	29	6.12	7.23	9.07	11.0	13.2	14.6
30	30	6.52	7.70	9.65	11.5	14.0	15.3
31	31	6.93	8.17	10.3	12.2	14.7	16.1
32	32	7.36	8.64	10.8	12.8	15.4	16.9
33	33	7.80	9.41	11.4	13.5	16.1	17.8
34	34	8.24	9.95	12.0	14.2	16.9	18.6
35	35	8.70	10.5	12.6	14.9	17.7	19.4
36	36	9.17	11.1	13.2	15.6	18.4	20.2
37	37	9.64	11.6	13.8	16.3	19.2	21.0
38	38	10.1	12.2	14.3	17.0	20.0	21.9
39	39	11.0	12.8	14.9	17.7	20.9	22.8
40	40	11.4	13.4	15.5	18.5	21.7	23.6
41	41	12.0	14.0	16.1	19.2	22.5	24.5
42	42	12.5	14.6	16.8	19.9	23.3	25.4
43	43	13.1	15.2	17.4	20.6	24.1	26.3
44	44	13.6	15.8	18.1	21.3	25.0	27.2
45	45	14.2	16.4	18.8	22.1	25.8	28.1
46	46	14.8	16.9	19.5	22.8	26.7	29.0
47	47	15.4	17.5	20.2	23.6	27.5	30.8
48	48	16.0	18.1	20.9	24.4	28.4	31.7
49	49	16.6	18.7	21.6	25.2	29.3	32.6
50	50	17.2	19.2	22.3	26.0	30.2	33.5
51	51	17.7	19.8	23.0	26.8	31.1	34.4
52	52	18.3	20.4	23.8	27.6	32.0	35.4
53	53	19.0	21.1	24.5	28.4	32.9	35.4
54	54	19.5	21.8	25.2	29.2	33.7	36.3
55	55	20.1	22.4	26.0	30.0	34.6	37.3
56	56	20.7	23.1	26.7	30.8	35.5	38.2
57	57	21.3	23.8	27.4	31.6	36.4	39.2
58	58	21.8	24.5	28.1	32.5	37.3	40.1
59	59	22.4	25.2	28.8	33.3	38.2	41.1
60	60	23.0	25.9	29.6	34.1	39.2	42.1

Table 6-11 Fundamental Period of Vibration, T_n, for Monolithic Tempered Glass (Continued)

Aspect Ratio = 1.25

Glass Dimensions (in.)		Fundamental Period of Vibration (msec) for Window Thickness, t, of --					
b	a	3/4 in.	5/8 in.	1/2 in.	3/8 in.	5/16 in.	1/4 in.
12	15	1.40	1.69	2.14	2.83	3.82	4.39
13	16.25	1.64	1.99	2.52	3.32	4.41	5.04
14	17.5	1.90	2.30	2.92	3.85	5.03	5.99
15	18.75	2.18	2.64	3.35	4.42	5.68	6.77
16	20	2.49	3.01	3.81	5.03	6.61	7.57
17	21.25	2.81	3.40	4.30	5.61	7.36	8.34
18	22.5	3.15	3.81	4.82	6.21	8.14	9.08
19	23.75	3.50	4.24	5.37	6.83	8.94	9.82
20	25	3.88	4.70	5.95	7.48	9.72	10.6
21	26.25	4.28	5.18	6.56	8.12	10.5	11.5
22	27.5	4.70	5.69	7.14	9.19	11.2	12.4
23	28.75	5.14	6.22	7.73	9.95	12.0	13.3
24	30	5.59	6.77	8.34	10.7	12.7	14.3
25	31.25	6.07	7.34	8.96	11.5	13.6	15.2
26	32.5	6.56	7.94	9.60	12.3	14.5	16.2
27	33.75	7.08	8.56	10.3	13.1	15.5	17.2
28	35	7.61	9.13	10.9	13.9	16.4	18.2
29	36.25	8.16	9.71	12.1	14.6	17.3	19.2
30	37.5	8.74	10.3	12.9	15.3	18.3	20.2
31	38.75	9.33	11.0	13.6	16.1	19.3	21.2
32	40	9.94	11.6	14.4	16.8	20.3	22.3
33	41.25	10.5	12.2	15.2	17.7	21.3	23.3
34	42.5	11.1	12.9	16.0	18.6	22.3	24.4
35	43.75	11.7	13.5	16.8	19.5	23.3	25.5
36	45	12.3	14.8	17.6	20.4	24.3	26.6
37	46.25	12.9	15.5	18.3	21.3	25.3	27.7
38	47.5	13.5	16.3	19.1	22.3	26.3	28.8
39	48.75	14.2	17.0	19.8	23.2	27.4	29.9
40	50	14.8	17.8	20.5	24.2	28.4	31.0
41	51.25	15.5	18.6	21.3	25.1	29.5	32.2
42	52.5	16.1	19.4	22.0	26.1	30.6	33.4
43	53.75	16.8	20.2	22.8	27.1	31.7	34.6
44	55	18.2	21.0	23.7	28.1	32.8	35.7
45	56.25	18.9	21.8	24.6	29.1	33.9	37.0
46	57.5	19.7	22.5	25.5	30.1	35.0	38.2
47	58.75	20.5	23.3	26.4	31.1	36.2	39.4
48	60	21.2	24.0	27.4	32.1	37.3	40.6
49	61.25	22.0	24.8	28.3	33.1	38.5	41.8
50	62.5	22.8	25.5	29.2	34.1	39.6	43.1
51	63.75	23.6	26.2	30.2	35.2	40.8	44.3
52	65	24.4	27.0	31.1	36.2	42.0	45.6
53	66.25	25.2	27.8	32.1	37.2	43.2	46.8

Table 6-11 Fundamental Period of Vibration, T_n, for Monolithic Tempered Glass (Continued)

Aspect Ratio = 1.50

Glass Dimensions (in.)		Fundamental Period of Vibration (msec) for Window Thickness, t, of --					
b	a	3/4 in.	5/8 in.	1/2 in.	3/8 in.	5/16 in.	1/4 in.
12	18	1.59	1.92	2.43	3.22	4.36	5.09
13	19.5	1.86	2.26	2.86	3.78	5.12	5.83
14	21	2.16	2.62	3.31	4.38	5.84	6.61
15	22.5	2.48	3.00	3.80	5.03	6.56	7.79
16	24	2.82	3.42	4.33	5.72	7.32	8.70
17	25.5	3.19	3.86	4.89	6.46	8.09	9.63
18	27	3.57	4.33	5.48	7.23	9.37	10.5
19	28.5	3.98	4.82	6.10	7.93	10.3	11.3
20	30	4.41	5.34	6.76	8.65	11.2	12.1
21	31.5	4.86	5.89	7.46	9.40	12.1	13.0
22	33	5.34	6.46	8.18	10.2	13.0	14.1
23	34.5	5.83	7.06	8.94	10.9	13.8	15.1
24	36	6.35	7.69	9.71	12.4	14.6	16.2
25	37.5	6.89	8.34	10.4	13.3	15.5	17.3
26	39	7.46	9.02	11.1	14.2	16.5	18.4
27	40.5	8.04	9.73	11.9	15.1	17.6	19.6
28	42	8.65	10.5	12.6	16.0	18.6	20.7
29	43.5	9.28	11.2	13.4	16.9	19.7	21.9
30	45	9.93	12.0	14.2	17.8	20.8	23.1
31	46.5	10.6	12.7	14.9	18.5	21.9	24.3
32	48	11.3	13.4	16.6	19.3	23.1	25.5
33	49.5	12.0	14.1	17.5	20.2	24.2	26.7
34	51	12.8	14.9	18.4	21.1	25.4	27.8
35	52.5	13.5	15.6	19.3	22.1	26.6	29.0
36	54	14.3	16.4	20.2	23.2	27.8	30.3
37	55.5	15.0	17.2	21.2	24.3	28.9	31.5
38	57	15.7	18.0	22.1	25.3	30.2	32.8
39	58.5	16.4	18.7	22.9	26.4	31.3	34.1
40	60	17.2	20.5	23.8	27.5	32.5	35.4
41	61.5	17.9	21.4	24.5	28.6	33.7	36.7
42	63	18.6	22.3	25.3	29.7	34.9	38.1
43	64.5	19.4	23.2	26.2	30.8	36.1	39.4
44	66	20.2	24.2	27.1	32.0	37.4	40.8
45	67.5	21.0	25.1	28.0	33.1	38.7	42.2
46	69	21.7	26.0	29.0	34.3	40.0	43.6
47	70.5	22.4	26.9	30.0	35.5	41.3	45.0
48	72	24.5	27.8	31.1	36.7	42.6	46.4

Table 6-11 Fundamental Period of Vibration, T_n, for Monolithic Tempered Glass (Continued)

Aspect Ratio = 1.75

Glass Dimensions (in.)		Fundamental Period of Vibration (msec) for Window Thickness, t, of --					
b	a	3/4 in.	5/8 in.	1/2 in.	3/8 in.	5/16 in.	1/4 in.
12	21	1.69	2.04	2.59	3.42	4.63	5.52
13	22.75	1.98	2.40	3.04	4.01	5.44	6.37
14	24.5	2.30	2.78	3.52	4.65	6.30	7.27
15	26.25	2.64	3.19	4.04	5.34	7.15	8.20
16	28	3.00	3.63	4.60	6.08	8.02	9.52
17	29.75	3.39	4.10	5.19	6.86	8.93	10.6
18	31.5	3.80	4.60	5.82	7.69	9.85	11.7
19	33.25	4.23	5.12	6.49	8.57	11.2	12.8
20	35	4.69	5.68	7.19	9.41	12.3	13.8
21	36.75	5.17	6.26	7.92	10.3	13.4	14.8
22	38.5	5.67	6.87	8.70	11.2	14.5	15.8
23	40.25	6.20	7.51	9.51	12.1	15.6	16.8
24	42	6.75	8.17	10.4	13.0	16.6	18.0
25	43.75	7.33	8.87	11.2	13.9	17.6	19.1
26	45.5	7.92	9.59	12.1	15.5	18.6	20.3
27	47.25	8.54	10.3	12.9	16.6	19.6	21.6
28	49	9.19	11.1	13.8	17.7	20.7	22.9
29	50.75	9.86	11.9	14.7	18.8	21.8	24.2
30	52.5	10.6	12.8	15.6	19.8	23.0	25.5
31	54.25	11.3	13.6	16.5	20.9	24.2	26.9
32	56	12.0	14.5	17.5	22.0	25.4	28.2
33	57.75	12.8	15.4	18.3	22.9	26.7	29.6
34	59.5	13.6	16.2	20.1	23.9	28.0	30.9
35	61.25	14.4	17.1	21.2	24.9	29.4	32.3
36	63	15.2	17.9	22.3	25.9	30.7	33.6
37	64.75	16.1	18.8	23.3	27.0	32.0	35.0
38	66.5	16.9	19.8	24.4	28.1	33.4	36.5
39	68.25	17.8	20.7	25.5	29.3	34.7	38.0
40	70	18.6	21.6	26.6	30.4	36.1	39.5
41	71.75	19.5	22.5	27.7	31.5	37.4	40.9
42	73.5	20.3	24.4	28.7	32.8	38.8	42.5
43	75.25	21.2	25.4	29.7	34.0	40.2	44.0
44	77	22.1	26.5	30.7	35.3	41.6	45.5
45	78.75	23.0	27.6	31.7	36.6	43.0	47.1

Table 6-11 Fundamental Period of Vibration, T_n, for Monolithic Tempered Glass (Continued)

Aspect Ratio = 2.00

Glass Dimensions (in.)		Fundamental Period of Vibration (msec) for Window Thickness, t, of --					
b	a	3/4 in.	5/8 in.	1/2 in.	3/8 in.	5/16 in.	1/4 in.
12	24	1.78	2.16	2.73	3.61	4.89	5.85
13	26	2.09	2.53	3.21	4.24	5.74	6.82
14	28	2.43	2.94	3.72	4.91	6.66	7.84
15	30	2.78	3.37	4.27	5.64	7.63	8.89
16	32	3.17	3.83	4.86	6.42	8.61	9.96
17	34	3.58	4.33	5.48	7.24	9.64	11.4
18	36	4.01	4.85	6.15	8.12	10.7	12.7
19	38	4.47	5.41	6.85	9.05	11.8	13.9
20	40	4.95	5.99	7.59	10.0	13.3	15.2
21	42	5.46	6.61	8.37	11.09	14.5	16.4
22	44	5.99	7.25	9.18	12.0	15.7	17.6
23	46	6.55	7.92	10.0	13.0	17.0	18.8
24	48	7.13	8.63	10.9	14.1	18.3	19.9
25	50	7.73	9.36	11.9	15.2	19.5	21.1
26	52	8.37	10.1	12.8	16.2	20.7	22.2
27	54	9.02	10.9	13.8	17.9	21.9	23.5
28	56	9.70	11.7	14.8	19.1	23.0	24.9
29	58	10.4	12.6	15.8	20.3	24.1	26.4
30	60	11.1	13.5	16.8	21.6	25.3	27.9
31	62	11.9	14.4	17.8	22.9	26.4	29.3
32	64	12.7	15.3	18.9	24.1	27.7	30.8
33	66	13.5	16.3	20.0	25.3	29.1	32.3
34	68	14.3	17.3	21.17	26.6	30.5	33.8
35	70	15.2	18.3	22.1	27.8	32.0	35.3
36	72	16.0	19.2	24.0	28.9	33.5	36.8
37	74	16.9	20.3	25.2	30.1	35.0	38.4
38	76	17.9	21.3	26.5	31.2	36.5	40.0
39	78	18.8	22.3	27.7	32.4	37.9	41.6
40	80	19.8	23.4	29.0	33.5	39.4	43.2
41	82	20.8	24.5	30.3	34.7	40.9	44.9
42	84	21.7	25.6	31.5	35.8	42.4	46.6

Table 6-11 Fundamental Period of Vibration, T_n, for Monolithic Tempered Glass (Continued)

Aspect Ratio = 3.00

Glass Dimensions (in.)		Fundamental Period of Vibration (msec) for Window Thickness, t, of --					
b	a	3/4 in.	5/8 in.	1/2 in.	3/8 in.	5/16 in.	1/4 in.
12	36	2.02	2.45	3.10	4.10	5.55	6.64
13	39	2.37	2.87	3.64	4.81	6.51	7.79
14	42	2.75	3.33	4.22	5.58	7.55	9.02
15	45	3.16	3.83	4.84	6.40	8.67	10.3
16	48	3.60	4.35	5.51	7.28	9.87	11.6
17	51	4.06	4.91	6.22	8.22	11.1	13.1
18	54	4.55	5.51	6.98	9.22	12.4	14.5
19	57	5.07	6.14	7.77	10.3	13.7	16.4
20	60	5.62	6.80	8.61	11.4	15.1	18.1
21	63	6.19	7.50	9.50	12.6	16.6	19.8
22	66	6.80	8.23	10.4	13.8	18.1	21.6
23	69	7.43	8.99	11.4	15.0	20.1	23.4
24	72	8.09	9.79	12.4	16.3	21.7	25.3
25	75	8.78	10.6	13.5	17.6	23.5	27.2
26	78	9.49	11.5	14.6	19.0	25.3	29.1
27	81	10.2	12.4	15.7	20.4	27.1	31.0
28	84	11.0	13.3	16.9	21.8	28.9	32.9
29	87	11.8	14.3	18.1	23.3	30.8	34.9
30	90	12.6	15.3	19.3	24.8	32.8	36.8
31	93	13.5	16.3	20.6	26.9	34.7	38.7
32	96	14.4	17.4	21.9	28.6	36.5	40.5
33	99	15.3	18.5	23.2	30.3	38.4	42.3
34	102	16.2	19.7	24.6	32.0	40.4	44.1

Table 6-11 Fundamental Period of Vibration, T_n, for Monolithic Tempered Glass (Continued)

Aspect Ratio = 4.00

Glass Dimensions (in.)		Fundamental Period of Vibration (msec) for Window Thickness, t, of --					
b	a	3/4 in.	5/8 in.	1/2 in.	3/8 in.	5/16 in.	1/4 in.
12	48	2.09	2.53	3.21	4.24	5.75	6.87
13	52	2.46	2.97	3.77	4.98	6.74	8.07
14	56	2.85	3.45	4.37	5.77	7.82	9.36
15	60	3.27	3.96	5.02	6.63	8.98	10.7
16	64	3.72	4.51	5.71	7.54	10.2	12.2
17	68	4.20	5.09	6.44	8.51	11.5	13.7
18	72	4.71	5.70	7.22	9.54	12.9	15.3
19	76	5.25	6.35	8.05	10.6	14.3	17.0
20	80	5.82	7.04	8.92	11.8	15.9	18.9
21	84	6.41	7.76	9.83	13.1	17.4	20.8
22	88	7.04	8.52	10.8	14.3	19.1	22.8
23	92	7.69	9.31	11.8	15.6	20.8	24.8
24	96	8.37	10.1	12.8	16.9	22.8	26.9
25	100	9.09	11.0	13.9	18.3	24.7	29.1
26	104	9.83	11.9	15.1	19.8	26.6	31.3
27	108	10.6	12.8	16.3	21.3	28.6	33.5
28	112	11.4	13.8	17.5	22.9	30.7	35.8
29	116	12.2	14.8	18.8	24.5	32.8	38.2
30	120	13.1	15.8	20.1	26.2	35.0	40.6

Table 6-12 Effective Elastic Static Resistance, r_{eff}

Aspect Ratio = 1

Glass Dimensions (in.)		Effective Elastic Static Resistance (psi) for Glass Thickness, t, of --					
b	a	3/4 in.	5/8 in.	1/2 in.	3/8 in.	5/16 in.	1/4 in.
12	12	206.	140.	87.7	50.3	27.5	19.1
13	13	175.	119.	74.7	42.8	23.9	16.3
14	14	151.	103.	64.5	36.9	21.1	14.2
15	15	131.	90.1	56.1	32.2	17.5	12.7
16	16	115.	79.2	49.3	28.3	15.5	11.6
17	17	102.	70.1	43.7	25.5	13.8	10.7
18	18	91.6	62.5	39.0	23.1	12.6	10.3
19	19	82.2	56.1	35.0	21.0	11.7	9.75
20	20	74.2	50.7	31.6	18.0	11.0	8.76
21	21	67.3	46.0	28.6	16.4	10.4	7.93
22	22	61.3	41.9	26.4	15.1	10.2	7.20
23	23	56.1	38.3	24.4	13.9	9.51	6.57
24	24	51.5	35.2	22.7	12.9	8.71	6.01
25	25	47.5	32.4	21.2	12.2	8.01	5.56
26	26	43.9	23.0	18.6	11.5	7.39	5.19
27	27	40.7	27.9	17.3	11.0	6.83	4.87
28	28	37.9	26.2	16.2	10.5	6.33	4.55
29	29	35.3	24.6	15.1	10.3	5.89	4.25
30	30	33.0	23.2	14.2	10.2	5.53	3.98
31	31	30.9	21.9	13.4	9.61	5.22	3.74
32	32	29.0	20.8	12.8	9.00	4.95	3.51
33	33	27.4	18.5	12.2	8.45	4.68	3.31
34	34	26.0	17.5	11.7	7.95	4.42	3.14
35	35	24.8	16.6	11.3	7.49	4.18	2.98
36	36	23.6	15.7	10.9	7.06	3.96	2.82
37	37	22.5	14.9	10.5	6.67	3.75	2.67
38	38	21.5	14.2	10.4	6.31	3.57	2.54
39	39	19.4	13.5	10.3	5.98	3.39	2.42
40	40	18.5	13.0	10.1	5.70	3.24	2.30
41	41	17.6	12.6	9.59	5.45	3.09	2.19
42	42	16.8	12.1	9.12	5.22	2.96	2.09
43	43	16.1	11.7	8.69	5.02	2.83	2.00
44	44	15.4	11.4	8.29	4.83	2.70	1.92
45	45	14.8	11.1	7.92	4.63	2.59	1.84
46	46	14.2	10.8	7.57	4.43	2.48	1.77
47	47	13.6	10.5	7.24	4.25	2.38	1.70
48	48	13.2	10.4	6.93	4.08	2.29	1.64
49	49	12.8	10.3	6.64	3.92	2.20	1.58
50	50	12.4	10.2	6.37	3.77	2.11	1.52
51	51	12.1	9.96	6.11	3.63	2.04	1.47
52	52	11.7	9.56	5.87	3.50	1.96	1.42
53	53	11.4	9.19	5.67	3.37	1.90	1.38
54	54	11.2	8.84	5.48	3.26	1.83	1.33
55	55	10.9	8.52	5.30	3.15	1.77	1.29
56	56	10.7	8.21	5.14	3.05	1.72	1.25
57	57	10.5	7.92	4.99	2.95	1.66	1.21
58	58	10.4	7.64	4.85	2.85	1.61	1.17
59	59	10.3	7.40	4.70	2.76	1.56	1.14
60	60	10.2	7.10	4.55	2.67	1.52	1.11

Table 6-12 Effective Elastic Static Resistance, r_{eff} (Continued)

Aspect Ratio = 1.25

Glass Dimensions (in.)		Effective Elastic Static Resistance (psi) for Glass Thickness, t, of --					
b	a	3/4 in.	5/8 in.	1/2 in.	3/8 in.	5/16 in.	1/4 in.
12	15	154.	105.	65.5	37.5	20.5	15.2
13	16.25	131.	89.5	55.8	32.0	17.9	13.5
14	17.5	113.	77.2	48.1	27.6	15.8	11.0
15	18.75	98.5	67.2	41.9	24.0	14.2	9.84
16	20	86.6	59.1	36.8	21.1	11.9	8.91
17	21.25	76.7	52.4	32.6	19.0	10.8	8.36
18	22.5	68.4	46.7	29.1	17.2	9.79	8.12
19	23.75	61.4	41.9	26.1	15.8	8.99	7.86
20	25	55.4	37.8	23.6	14.6	8.50	7.49
21	26.25	50.3	34.3	21.4	13.6	8.19	6.73
22	27.5	45.8	31.3	19.7	11.6	8.07	6.11
23	28.75	41.9	28.6	18.2	10.8	7.79	5.57
24	30	38.5	26.3	17.0	10.1	7.44	5.09
25	31.25	35.5	24.2	15.9	9.40	6.81	4.68
26	32.5	32.8	22.4	14.9	8.89	6.27	4.31
27	33.75	30.4	20.8	14.1	8.53	5.79	4.00
28	35	28.3	19.5	13.4	8.25	5.37	3.73
29	36.25	26.4	18.4	11.7	8.14	4.99	3.49
30	37.5	24.6	17.4	11.0	8.04	4.65	3.28
31	38.75	23.1	16.4	10.4	7.82	4.34	3.09
32	40	21.7	15.6	9.91	7.62	4.07	2.92
33	41.25	20.4	14.9	9.41	7.21	3.84	2.76
34	42.5	19.4	14.2	8.99	6.75	3.62	2.61
35	43.75	18.5	13.6	8.70	6.35	3.43	2.47
36	45	17.6	12.0	8.45	5.99	3.26	2.34
37	46.25	16.8	11.5	8.25	5.66	3.10	2.22
38	47.5	16.1	11.0	8.16	5.35	2.96	2.12
39	48.75	15.5	10.6	8.09	5.07	2.82	2.01
40	50	14.8	10.1	7.98	4.81	2.69	1.91
41	51.25	14.3	9.72	7.81	4.57	2.57	1.82
42	52.5	13.8	9.34	7.66	4.34	2.45	1.74
43	53.75	13.4	9.01	7.43	4.14	2.34	1.66
44	55	11.8	8.78	7.06	3.96	2.25	1.59
45	56.25	11.4	8.57	6.72	3.79	2.16	1.52
46	57.5	11.0	8.38	6.42	3.63	2.07	1.46
47	58.75	10.6	8.24	6.14	3.49	1.98	1.40
48	60	10.3	8.17	5.88	3.36	1.90	1.34
49	61.25	9.93	8.11	5.63	3.23	1.83	1.29
50	62.5	9.60	8.06	5.39	3.12	1.76	1.25
51	63.75	9.29	7.93	5.18	3.01	1.69	1.20
52	65	9.02	7.80	4.97	2.90	1.63	1.16
53	66.25	8.82	7.68	4.78	2.80	1.57	1.12

Table 6-12 Effective Elastic Static Resistance, r_{eff} (Continued)

Aspect Ratio = 1.50

Glass Dimensions (in.)		Effective Elastic Static Resistance (psi) for Glass Thickness, t, of --					
b	a	3/4 in.	5/8 in.	1/2 in.	3/8 in.	5/16 in.	1/4 in.
12	18	123.	83.8	52.3	29.9	16.3	11.9
13	19.5	105.	71.4	44.5	25.5	13.9	10.5
14	21	90.2	61.6	38.4	22.0	12.3	9.43
15	22.5	78.6	53.6	33.4	19.2	11.1	7.86
16	24	69.1	47.2	29.4	16.8	10.1	7.16
17	25.5	61.2	41.8	26.0	14.9	9.31	6.56
18	27	54.6	37.3	23.2	13.3	7.83	6.25
19	28.5	49.0	33.4	20.8	12.3	7.24	6.30
20	30	44.2	30.2	18.8	11.4	6.71	6.23
21	31.5	40.1	27.4	17.1	10.6	6.37	5.91
22	33	36.5	24.9	15.6	9.86	6.16	5.36
23	34.5	33.4	22.8	14.23	9.32	6.35	4.86
24	36	30.7	21.0	13.1	7.99	6.21	4.44
25	37.5	28.3	19.3	12.4	7.56	5.94	4.08
26	39	26.2	17.9	11.7	7.13	5.51	3.75
27	40.5	24.3	16.6	11.0	6.75	5.06	3.47
28	42	22.6	15.4	10.4	6.46	4.68	3.21
29	43.5	21.0	14.4	9.89	6.28	4.35	2.99
30	45	19.7	13.4	9.42	6.18	4.05	2.79
31	46.5	18.4	12.8	9.16	6.33	3.78	2.61
32	48	17.3	12.2	7.90	6.32	3.54	2.45
33	49.5	16.2	11.6	7.57	6.11	3.32	2.33
34	51	15.3	11.1	7.24	5.92	3.12	2.21
35	52.5	14.4	10.6	6.94	5.59	2.93	2.10
36	54	13.6	10.2	6.66	5.24	2.77	2.00
37	55.5	13.0	9.78	6.46	4.94	2.62	1.90
38	57	12.5	9.42	6.32	4.67	2.49	1.81
39	58.5	12.0	9.21	6.20	4.42	2.38	1.72
40	60	11.6	8.03	6.22	4.19	2.28	1.64
41	61.5	11.2	7.78	6.33	3.98	2.18	1.57
42	63	10.8	7.52	6.36	3.78	2.09	1.49
43	64.5	10.4	7.26	6.20	3.60	2.01	1.42
44	66	10.1	7.02	6.05	3.43	1.92	1.36
45	67.5	9.71	6.79	5.91	3.27	1.84	1.30
46	69	9.42	6.57	5.65	3.13	1.77	1.25
47	70.5	9.25	6.45	5.39	2.99	1.70	1.20
48	72	8.12	6.33	5.14	2.86	1.63	1.15

Table 6-12 Effective Elastic Static Resistance, r_{eff} (Continued)

Aspect Ratio = 1.75

Glass Dimensions (in.)		Effective Elastic Static Resistance (psi) for Glass Thickness, t, of --					
b	a	3/4 in.	5/8 in.	1/2 in.	3/8 in.	5/16 in.	1/4 in.
12	21	109.	74.2	46.3	26.5	14.4	10.2
13	22.75	92.6	63.2	39.4	22.6	12.3	8.91
14	24.5	79.9	54.5	34.0	19.5	10.6	8.01
15	26.25	69.6	47.5	29.6	17.0	9.46	7.32
16	28	61.2	41.7	26.0	14.9	8.52	6.20
17	29.75	54.2	37.0	23.1	13.2	7.85	5.65
18	31.5	48.3	33.0	20.6	11.7	7.30	5.19
19	33.25	43.4	29.6	18.5	10.67	6.27	4.89
20	35	39.1	26.7	16.7	9.71	5.79	4.70
21	36.75	35.5	24.2	15.1	8.96	5.38	4.64
22	38.5	32.4	22.1	13.8	8.36	5.05	4.50
23	40.25	29.6	20.2	12.6	7.87	4.84	4.31
24	42	27.2	18.6	11.6	7.43	4.70	4.02
25	43.75	25.1	17.1	10.7	7.11	4.65	3.79
26	45.5	23.2	15.8	9.97	6.17	4.54	3.51
27	47.25	21.5	14.7	9.37	5.82	4.41	3.24
28	49	20.0	13.6	8.83	5.52	4.19	3.00
29	50.75	18.6	12.7	8.38	5.23	3.96	2.78
30	52.5	17.4	11.9	8.01	5.02	3.77	2.60
31	54.25	16.3	11.1	7.66	4.86	3.53	2.43
32	56	15.3	10.4	7.35	4.73	3.30	2.29
33	57.75	14.4	9.93	7.11	4.68	3.09	2.16
34	59.5	13.5	9.46	6.27	4.64	2.90	2.05
35	61.25	12.8	9.02	6.00	4.56	2.73	1.94
36	63	12.1	8.62	5.74	4.46	2.58	1.85
37	64.75	11.4	8.30	5.51	4.36	2.44	1.76
38	66.5	10.8	8.00	5.30	4.18	2.32	1.67
39	68.25	10.3	7.73	5.10	4.01	2.21	1.59
40	70	9.91	7.47	4.97	3.86	2.11	1.51
41	71.75	9.52	7.26	4.85	3.72	2.02	1.44
42	73.5	9.15	6.51	4.75	3.53	1.93	1.38
43	75.25	8.81	6.29	4.70	3.36	1.85	1.32
44	77	8.50	6.07	4.67	3.20	1.78	1.26
45	78.75	8.24	5.86	4.64	3.05	1.71	1.20

Table 6-12 Effective Elastic Static Resistance, r_{eff} (Continued)

Aspect Ratio = 2.00

Glass Dimensions (in.)		Effective Elastic Static Resistance (psi) for Glass Thickness, t, of --					
b	a	3/4 in.	5/8 in.	1/2 in.	3/8 in.	5/16 in.	1/4 in.
12	24	97.6	66.6	41.5	23.8	13.0	9.05
13	26	83.1	56.7	35.4	20.3	11.0	7.81
14	28	71.7	48.9	30.5	17.5	9.52	6.87
15	30	62.4	42.6	26.6	15.2	8.31	6.29
16	32	54.9	37.5	23.4	13.4	7.43	5.81
17	34	48.6	33.2	20.7	11.9	6.72	4.98
18	36	43.4	29.6	18.5	10.6	6.26	4.54
19	38	38.9	26.6	16.6	9.49	5.86	4.19
20	40	35.1	24.0	14.9	8.56	5.12	3.93
21	42	31.9	21.7	13.6	7.85	4.72	3.74
22	44	29.0	19.8	12.4	7.25	4.40	3.59
23	46	26.6	18.1	11.3	6.73	4.11	3.48
24	48	24.4	16.6	10.4	6.39	3.92	3.42
25	50	22.5	15.3	9.56	6.08	3.75	3.33
26	52	20.8	14.2	8.84	5.79	3.62	3.24
27	54	19.3	13.2	8.23	5.15	3.51	3.04
28	56	17.9	12.2	7.73	4.85	3.45	2.81
29	58	16.7	11.4	7.27	4.58	3.40	2.61
30	60	15.6	10.7	6.86	4.36	3.32	2.42
31	62	14.6	9.98	6.57	4.15	3.25	2.27
32	64	13.7	9.36	6.32	3.99	3.10	2.14
33	66	12.9	8.80	6.08	3.86	2.90	2.02
34	68	12.2	8.31	5.87	3.74	2.72	1.91
35	70	11.5	7.91	5.66	3.64	2.56	1.81
36	72	10.8	7.53	5.07	3.56	2.41	1.72
37	74	10.3	7.18	4.84	3.49	2.28	1.63
38	76	9.73	6.86	4.64	3.45	2.17	1.56
39	78	9.24	6.62	4.46	3.41	2.06	1.48
40	80	8.78	6.42	4.30	3.36	1.97	1.41
41	82	8.37	6.23	4.14	3.30	1.88	1.34
42	84	8.03	6.05	4.02	3.25	1.80	1.28

Table 6-12 Effective Elastic Static Resistance, r_{eff} (Continued)

Aspect Ratio = 3.00

Glass Dimensions (in.)		Effective Elastic Static Resistance (psi) for Glass Thickness, t, of --					
b	a	3/4 in.	5/8 in.	1/2 in.	3/8 in.	5/16 in.	1/4 in.
12	36	81.1	55.4	34.5	19.8	10.8	7.53
13	39	69.1	47.2	29.4	16.9	9.18	6.41
14	42	59.6	40.7	25.4	14.5	7.92	5.57
15	45	51.9	35.4	22.1	12.7	6.90	4.94
16	48	45.6	31.2	19.4	11.1	6.06	4.42
17	51	40.4	27.6	17.2	9.86	5.43	3.98
18	54	36.1	24.6	15.3	8.79	4.92	3.61
19	57	32.4	22.1	13.8	7.89	4.48	3.15
20	60	29.2	19.9	12.4	7.12	4.10	2.86
21	63	26.5	18.1	11.3	6.46	3.77	2.64
22	66	24.1	16.5	10.3	5.88	3.48	2.45
23	69	22.1	15.1	9.40	5.44	3.08	2.25
24	72	20.3	13.8	8.63	5.06	2.84	2.08
25	75	18.7	12.8	7.95	4.71	2.66	1.92
26	78	17.3	11.8	7.35	4.40	2.49	1.82
27	81	16.0	10.9	6.82	4.13	2.34	1.74
28	84	14.9	10.2	6.34	3.88	2.18	1.66
29	87	13.9	9.48	5.91	3.65	2.04	1.58
30	90	13.0	8.86	5.56	3.44	1.91	1.51
31	93	12.2	8.30	5.26	3.11	1.83	1.47
32	96	11.4	7.79	4.98	2.93	1.76	1.43
33	99	10.7	7.32	4.72	2.78	1.70	1.41
34	102	10.1	6.90	4.48	2.64	1.63	1.39

Table 6-12 Effective Elastic Static Resistance, r_{eff} (Continued)

Aspect Ratio = 4.00

Glass Dimensions (in.)		Effective Elastic Static Resistance (psi) for Glass Thickness, t, of --					
b	a	3/4 in.	5/8 in.	1/2 in.	3/8 in.	5/16 in.	1/4 in.
12	48	75.7	51.7	32.2	18.5	10.1	7.02
13	52	64.5	44.0	27.5	15.7	8.57	5.99
14	56	55.6	38.0	23.7	13.6	7.39	5.16
15	60	48.5	33.1	20.6	11.8	6.43	4.52
16	64	42.6	29.1	18.1	10.4	5.66	3.99
17	68	37.7	25.8	16.1	9.20	5.01	3.56
18	72	33.7	23.0	14.3	8.20	4.49	3.19
19	76	30.2	20.6	12.9	7.36	4.05	2.87
20	80	27.3	18.6	11.6	6.65	3.67	2.54
21	84	24.7	16.9	10.5	6.03	3.34	2.30
22	88	22.5	15.4	9.59	5.49	3.06	2.10
23	92	20.6	14.1	8.77	5.03	2.81	1.94
24	96	18.9	12.9	8.05	4.63	2.52	1.79
25	100	17.5	11.9	7.42	4.28	2.32	1.67
26	104	16.1	11.0	6.86	3.97	2.15	1.56
27	108	15.0	10.2	6.36	3.70	2.00	1.46
28	112	13.9	9.49	5.92	3.45	1.88	1.38
29	116	13.0	8.85	5.52	3.22	1.76	1.30
30	120	12.1	8.27	5.15	3.02	1.66	1.23

UNDERGROUND STRUCTURES

6-33. Introduction

Underground structures are not usually used for production and handling of explosives since access for both personnel and explosives is more difficult than for an aboveground structure. However, an explosion may result in severe hazards from which an aboveground structure can not provide adequate protection and a buried structure will be required. An example might be a manned control building at a test site which must be located very close to a high-hazard operation involving a relatively large quantity of explosives.

There is limited test data available to predict the pressures acting on an underground structure. What test data that is available was developed for use in the design of protective structures to resist the effects of an attack with conventional weapons. The results of this data and the design procedures developed from it are given in the technical manual, "Fundamentals of Protective Design for Conventional Weapons," TM 5-855-1. The data presented may be expanded to include the design of structures subjected to accidental explosions. The pertinent sections are briefly summarized below.

A typical underground structure used to resist conventional weapons attack is shown in Figure 6-53. The burster slab prevents a weapon from penetrating through the soil and detonating adjacent to the structure. A burster slab is not mandatory, but if it is not used the structure will have to be buried much deeper. The burster slab must extend far enough beyond the edge of the building to form at least a 45 degree angle with the bottom edge of the building. It may have to be extended further, though, if it is possible for a bomb to penetrate at a very shallow angle, travel beneath the burster slab and detonate adjacent to the structure (see Figure 6-53). Sand is used as backfill because materials with high volume of air-filled voids and low relative densities are poor transmitters of ground shock. In addition, sand resists penetration better than soil.

6-34. Design Loads for Underground Structures

6-34.1. General

The pressure-time relationships for roof panels and exterior walls are determined separately. For the roof panel, an overhead burst produces the most critical loading while for an exterior wall a sideburst is critical. A general description of the procedure for determining the peak pressures and their durations is given below. For a more detailed description, including the required equations, see TM 5-855-1.

The magnitude of the ground shock is affected by:

1. The size of the explosive charge and its distance from the structure;

2. The mechanical properties of the soil, rock, and/or concrete between the detonation point and the structure; and

3. The depth of penetration at the time of detonation.

The stresses and ground motions are greatly enhanced as the depth of the explosion increases. To account for this effect a coupling factor is used. The coupling factor is defined as the ratio of the ground shock magnitude from a near surface burst to that of a fully buried burst. A single coupling factor applies to all ground shock parameters and depends on the depth of the explosion and whether detonation occurs in soil, concrete or air.

6-34.2. **Roof Loads**

A typical roof load (shown in Figure 6-54) consists of a free-field pressure P_o and a reflected pressure P_r. The reflected pressure occurs when the free-field pressure impinges on the roof panel and is instantly increased to a higher pressure. The amount of increase is a function of the pressure in the free-field wave and the angle formed between the rigid surface and the plane of the pressure front. However, TM 5-855-1 suggests that an average reflection factor of 1.5 is reasonable.

The pressures on the roof of an underground structure are not uniform across the panel, especially if the depth of the explosion is shallow. However, in order to use a single-degree-of-freedom analysis, a uniform load is required and hence an average uniform pressure must be determined. TM 5-855-1 presents figures that give the ratio peak pressures at the center of a roof panel to the average pressure across the entire panel. This ratio is a function of the support conditions and aspect ratio of the panel and the height of the burst above the roof.

For the most severe roof load the explosive is positioned directly over the center of the panel. The average free-field and average reflected pressures are calculated as described above. The duration of the pressure pulse also varies across the roof panel and a fictitious average duration t_o must be determined. TM 5-855-1 recommends calculating the duration of the peak free-field pressure pulse at a point located one quarter of the way along the short span and at the center of the long span. This duration is then used as the average duration of the entire panel. The peak free-field pressure and impulse are calculated using equations given in TM 5-855-1. The duration is found by assuming a triangular pressure-time relationship. The duration of the average reflected pressure t_r is given in TM 5-855-1 as a function of either the thickness of the structural element or the distance to the nearest free edge of the structure. The smaller of the two numbers should be used in analysis.

6-34.3. **Wall Loads**

The design loads on an exterior wall are determined using the procedures described in Section 6-34.2 for roof loads. However, in addition to the pressure wave traveling directly from the explosion, the wall may be subject to a pressure wave reflected off the ground surface or burster slab and/or a pressure wave reflected off a lower rock layer or water table.

The parameters of each wave (average reflected pressure, average free-field pressure, durations and time of arrival) are determined separately using procedures very similar to those described in Section 6-34.2. The total pressure-time history is equal to the superposition of the three waves as shown in Figure 6-55. The superposition results in a very complicated load shape. The response charts of Chapter 3 are not applicable for such a shape, therefore the load must be idealized. The actual load is transformed into a triangular load having the same total impulse (area under the actual load curve equal to the sum of the areas of the direct and reflected waves). The maximum pressure of the idealized load is equal to the maximum pressure of the actual load neglecting the short reflected peaks. The duration is then established as a function of the total impulse and maximum pressure (Figure 6-55). For an exact solution, the actual load curve is used in a single-degree-of-freedom computer program analysis or numerical analysis as given in Section 3-19.2.

6-35. Structural Design

6-35.1. Wall and Roof Slabs

The structural design of underground structures is very similar to the design of aboveground structures as described in Chapter 4. The effect of the soil is to modify the response of the structural components. The dead load of the soil reduces the resistance available to resist blast. At the same time a portion of the soil acts with the structural elements to increase the natural period of vibration. In the case of a wall, it is assumed that the mass of two (2) feet of soil acts with the mass of the wall. Whereas for a roof, the entire mass of the soil supported by the roof, or a depth of soil equal to one quarter of the roof span (short span for a two-way panel) whichever is smaller, is added to the mass of the roof.

The dynamic response of underground structures must obviously be limited to comparatively small deformations to prevent collapse of the structure due to earth loads. A protective structure subjected to conventional weapons attack should be designed for a ductility ratio of 5.0, as recommended by TM 5-855-1. This ratio may be increased to 10 if special provisions are taken. A maximum deflection corresponding to a support rotation of one (1) degree or a ductility ratio of 10.0 is permitted for underground structures subjected to accidental explosions.

Spalling is the ejection of material from the back face of a slab or beam. It results from high-intensity, close-in explosions. Fragment shields or backing plates, as shown in Figure 6-56, are of some value in protecting personnel and equipment. These steel plates must be securely anchored to the inside face of the concrete member. Tests have shown that the shock of a deep penetrating detonation to be enough to cause inadequate welds to fail over a large area, adding the whole steel plate to the concrete spall. A strongly attached plate adds about 10 percent to the perforation resistance of a concrete slab. For a further discussion of backing plates, see Chapter 4.

6-35.2. Burster Slab

For protective structures, a burster slab prevents a weapon from penetrating through the soil and detonating adjacent to the structure. Its thickness and length may have to be determined by a trial and error procedure in order to limit pressures on the structure to a given value. However, the minimum dimensions are shown in Figure 6-53. The minimum reinforcement is 0.1 percent in each face, in each direction or a total of 0.4 percent. In the design of structures subject to accidental explosions, the ground floor slab of the donor building serves a purpose similar to that of a burster slab. The floor slab helps to prevent fragment penetration and to attenuate the load.

6-36. Structure Motions

6-36.1. Shock Spectra

TM 5-855-1 gives equations for acceleration, velocity and displacements for underground structures. These simplified methods take into account the attenuation of the pressure wave as it transverses the structure. For a sideburst, the vertical acceleration, velocity and displacements are 20 percent of the horizontal values. The horizontal motions are uniform over the entire floor while vertical motions at the leading edge are twice those at midspan.

Once the peak in-structure acceleration, velocity and displacements have been determined an in-structure shock spectra can be developed using the principles of Chapter 2 of this manual.

6-36.2. Shock Isolation Systems

Chapter 1 presents the upper limits of the shock environment that personnel and equipment can tolerate. If the shock environment exceeds human tolerances and/or equipment "fragility levels" then shock isolation systems are required to protect personnel and sensitive equipment. Using the shock spectra developed as described above, shock isolation systems are designed as outlined in Sections 6-43 through 6-49.

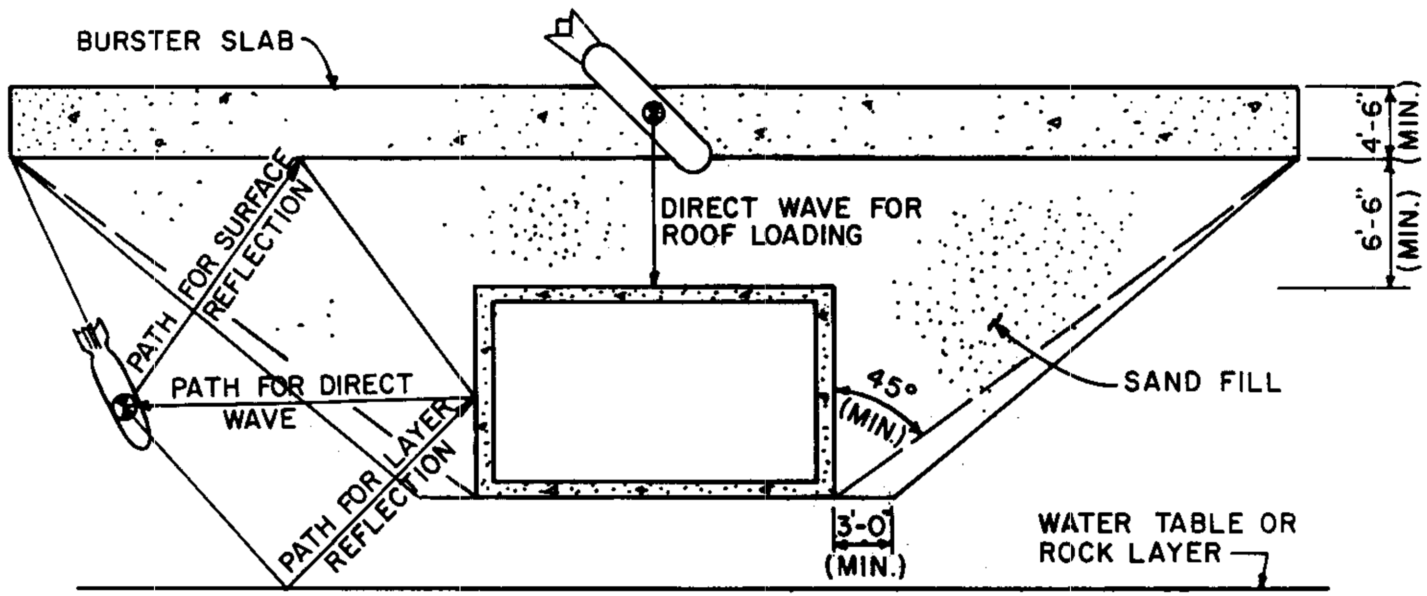


Figure 6-53 Geometry of a buried structure

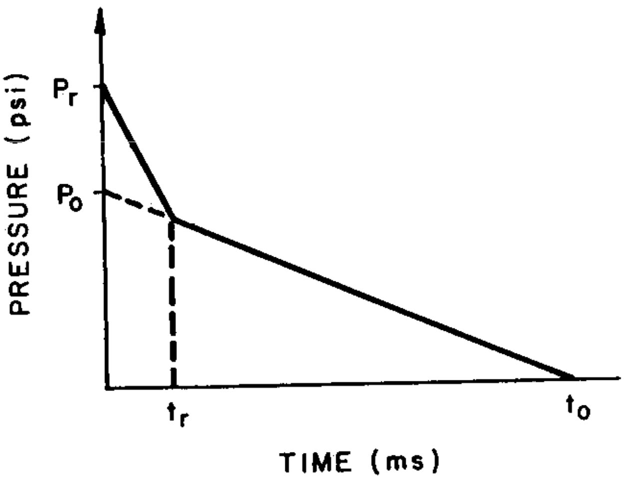


Figure 6–54 Typical roof panel load

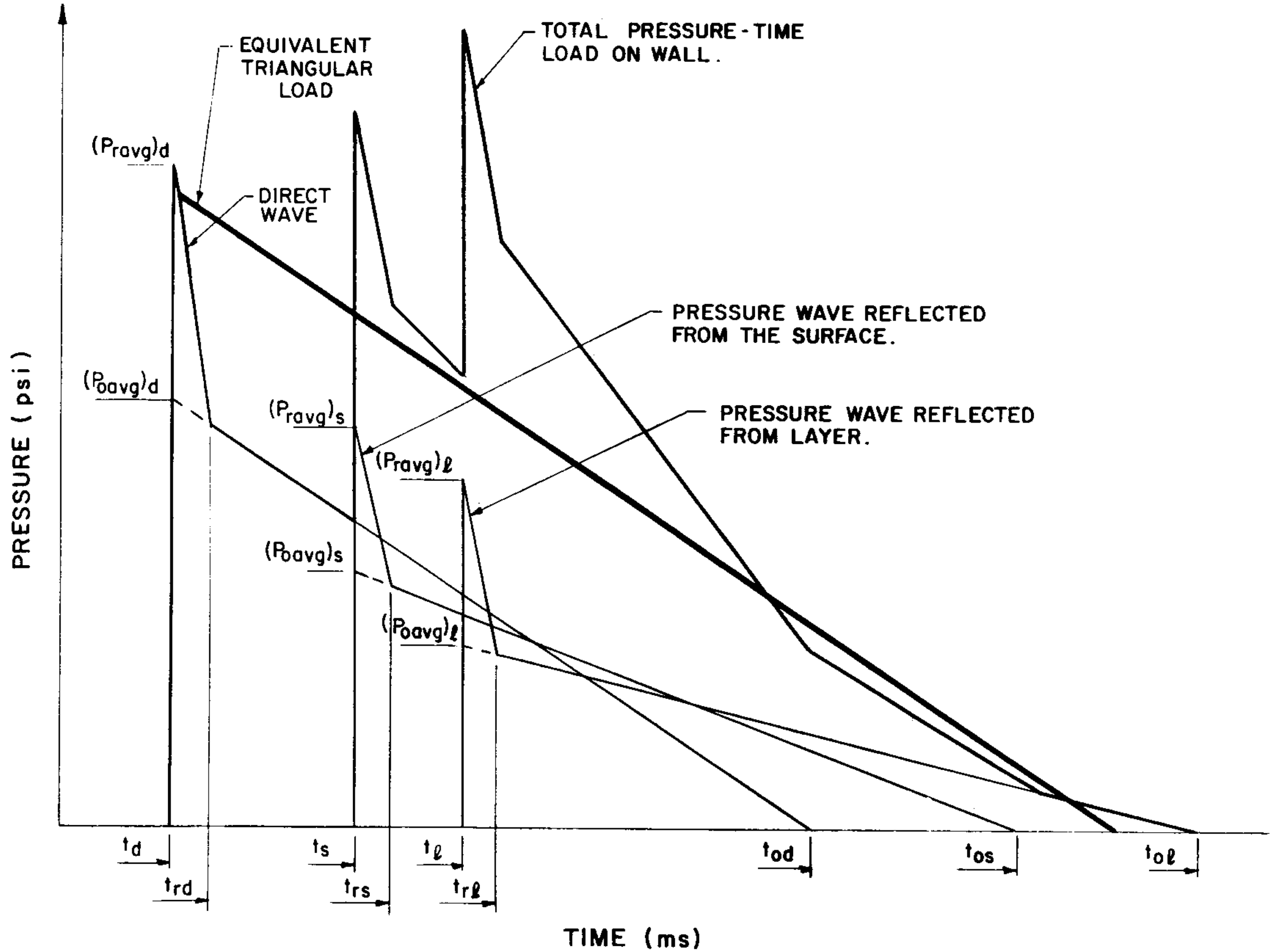
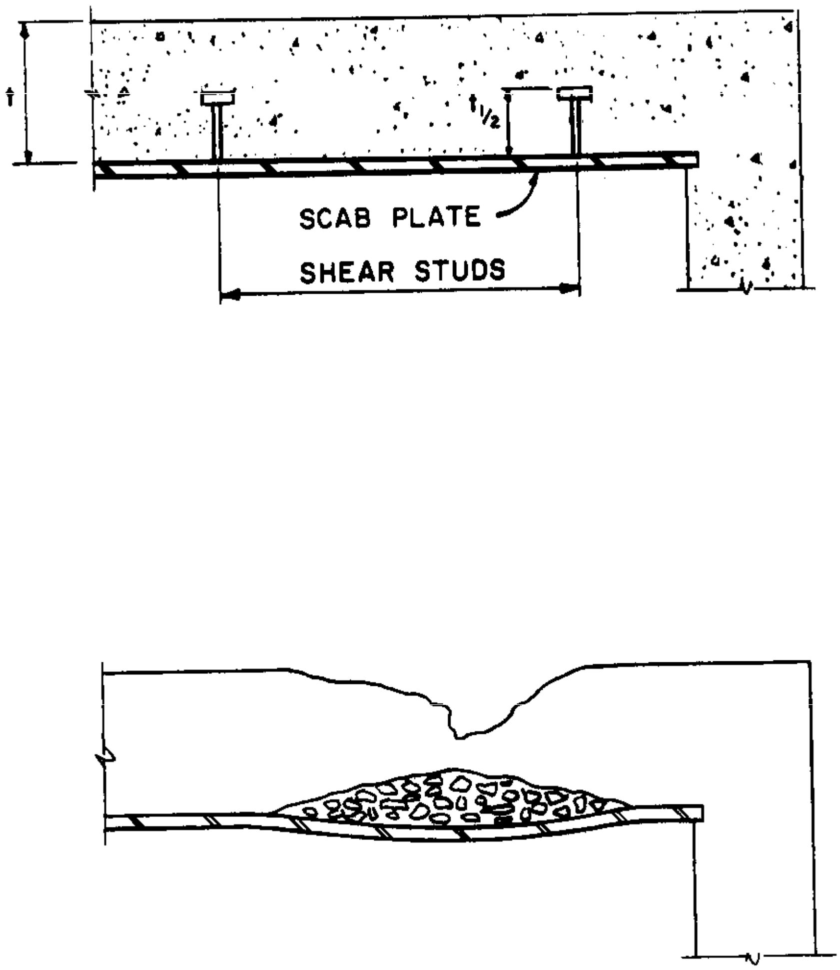


Figure 6–55 Contribution of three pressure waves on a wall



RETENTION OF SCAB MATERIAL

Figure 6–56 Spall plate

EARTH-COVERED ARCH-TYPE MAGAZINES

6-37. General

Certain types of earth-covered concrete-arch and steel-arch magazines have been approved and standardized for use by the Department of Defense Explosives Safety Board. These magazines provide definite advantages over other types of magazines. Among these advantages are:

1. Less real estate per magazine is required because of the decreased intermagazine separations permitted when approved magazines are used.

2. An almost infinite number of storage situations exists because magazines can be designed to any length.

3. Because of the reduced separation distances, less roads, fences, utilities, etc., are required.

Unlike the other structures discussed elsewhere in this manual, an earth-covered magazine is not designed to resist the damaging effects of its exploding contents. It is accepted that the magazine will be demolished if an internal explosion occurs. During such an incident, the inside of a large-span arch might experience initial blast pressures considerably in excess of 10,000 psi. Less than 100 psi could lift the arch completely out of the ground; therefore, the major portion of the protection is provided by the receiver magazines rather than the donor magazine.

Earth-covered magazines are utilized primarily to prevent propagation of explosion. These structures may also be used for operating buildings and can provide personnel protection. In such cases, separation distances greater than those required to prevent propagation of explosions will be necessary. In addition, a special evaluation of the structure is required. This evaluation must include the leakage of blast pressures into the protected area, the strength and attachment of easily damaged or lightly supported accessories which may become hazardous debris, the transmission of shock to personnel through the walls or floors, and overall movement of the magazines.

6-38. Description of Earth-Covered Arch-Type Magazines

A typical earth-covered arch-type magazine used for storing explosives has the following features:

1. A semicircular or oval arch constructed of reinforced concrete or corrugated steel used to form roof and sides.

2. A reinforced concrete floor slab, sloped for drainage.

3. A reinforced concrete rear wall.

4. A reinforced concrete headwall that extends at least 2-1/2 feet above the crown of the arch.

5. Reinforced concrete wingwalls on either side of the headwall. The wingwalls may slope to the ground or may adjoin wingwalls from adjacent

magazines. The wingwalls may be either monolithic or separated by expansion joints from the headwall.

6. Heavy steel doors in the headwall (either manually operated and/or motorized).

7. An optional gravity ventilation system.

8. Earth cover over the top, sides and rear of the structure. This cover must be at least 2 feet thick at the crown of the arch. The earth above the structure (within the spring line of each arch and between the head and rear walls) is sloped for drainage while beyond the outline of the structure the earth is sloped 2 horizontal to 1 vertical.

9. Its own built-in lightning protection and grounding systems.

A typical earth-covered steel arch magazine is illustrated in Figure 6-57.

6-39. Separation Distances of Standard Magazines

Numerous full scale tests of standard magazines have been performed over a period of several years. As a result magazine separation formulae have been established, which will prevent magazine-to-magazine propagation of explosions. All possible right angle arrangements have been considered, i.e., side-to-side, rear-to-rear, front-to-rear, etc. The standard magazines, which are at least equivalent in strength to those.tested, are listed in the DoD Standard, "DoD Ammunition and Explosives Safety Standards, 6055.9-STD." The required separation distances, as a function of the quantity of explosives stored in the structure are also given in the DoD Standard. A possible magazine arrangement is shown in Figure 6-58.

6-40. Design

Protection of magazines adjacent to a donor magazine is accomplished by combining the following factors:

1. The intensity of the pressure front moving from the donor magazine to receiver magazines diminishes rapidly as the distance traveled increases.

2. The earth cover over and around the donor magazine provides some confinement and tends to directionalize the explosive force both upward and outward from the door end of the magazine.

3. The earth around and over receiver magazines resists fragment penetrations and provides mass to the arch to resist the blast pressures.

4. The arch of receiver magazines is capable of resisting blast loads considerably in excess of the dead loads normally imposed on it.

Design of presently used magazines is essentially conventional except for two features, which are doors and arch. The doors are designed to withstand the dynamic forces from an explosion in a nearby magazine if the siting is in accordance with Figure 6-58. However, they provide almost no resistance to the effects of an explosion within the magazine. Also, the capacity of the doors to resist elastic rebound and negative phase pressures may be less than their capacity to resist positive phase pressures. Therefore, where personnel are concerned, all doors should be analyzed to determine their ultimate capacity to resist all the loadings involved. The arches used for the standard earth-covered magazines are the same as those used in the test structures to establish the required separation distances. These arches have not been dynamically designed for the blast loads and may be in excess of that required.

6-41. Construction

Effectiveness of earth-covered magazines is largely determined by the quality of construction. A few of the construction details that could be sources for problems in this type of structure are discussed below.

Moisture proofing of any earth-covered structure is usually difficult. This difficulty is increased with a steel-arch structure because of the many lineal feet of joints available for introducing moisture. For example, a large 26- by 80-foot magazine contains approximately 1,050 feet of edges. A sealant tape must be used that will not deteriorate or excessively deform under any anticipated environmental or structural condition.

Earthfill material should be clean, cohesive, and free from large stones. A minimum earth cover of two feet must be maintained. Surface preparation of the fill is usually required to prevent erosion of the 2-foot cover.

Restricting granular size of material reduces throwout of fragments in case of an accidental explosion and creates a more uniform energy absorbent over the top of the magazine.

Lightning protection is rather easily obtained in a steel-arch structure. All sections of steel-arch plate must be interconnected so that they become electrically continuous. In a concrete-arch magazine, the reinforcing steel must be interconnected. In effect, a "cage" is created about the magazine contents. Probably the most critical point for lightning protection is the optional ventilator stack which projects above the surrounding earth cover.

6-42. Non-Standard Magazines

Non-standard earth-covered magazines, that is magazines not listed in DoD Standard 6055.9-STD may also be used for explosive storage. However, if a "non-standard" earth-covered magazine or an aboveground magazine is used the separation distances must be increased. The DoD Standard 6055.9-STD includes the increased separation distances, as well as other criteria, for "non-standard" earth-covered and aboveground magazines.

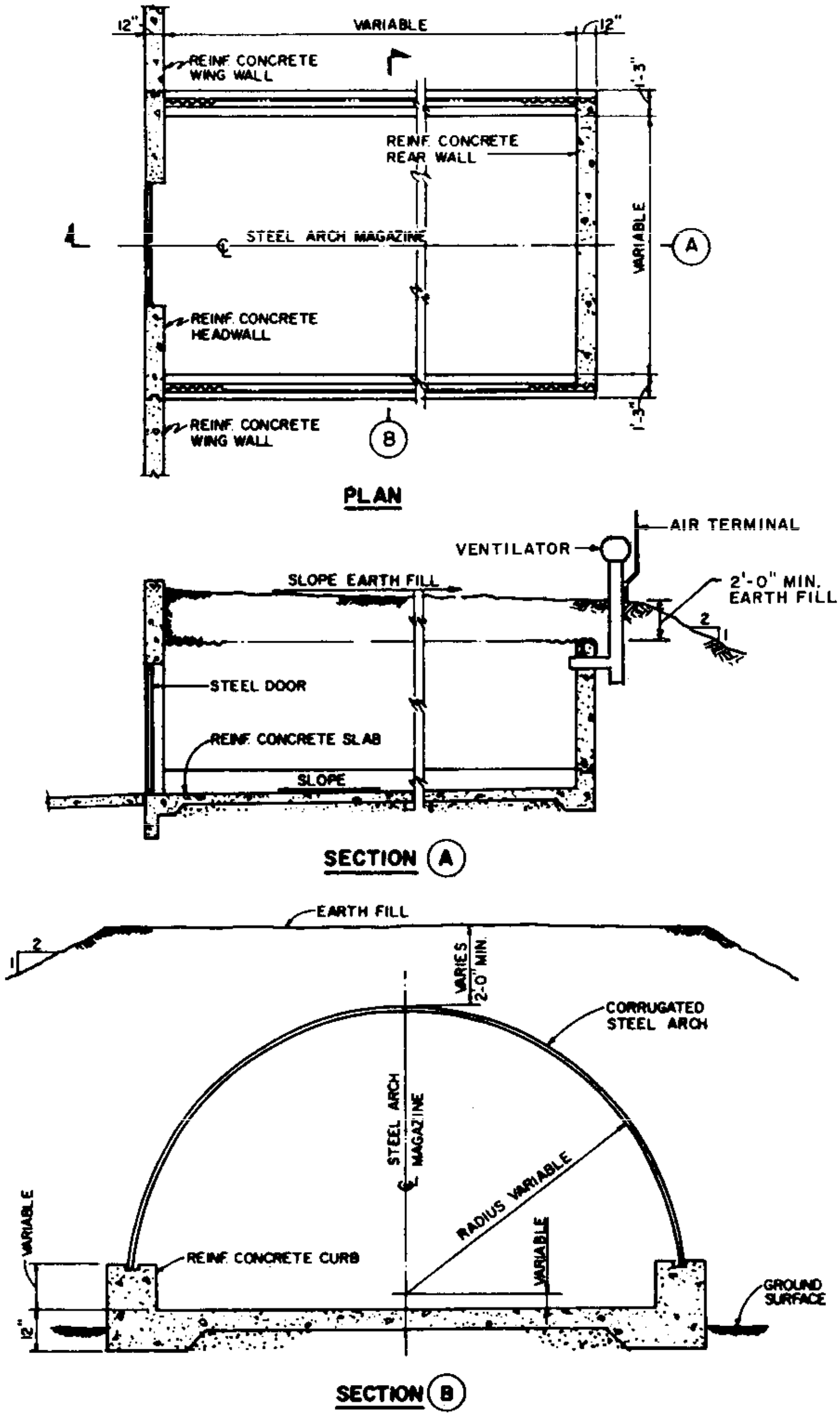


Figure 6-57 Typical earth-covered steel-arch magazine

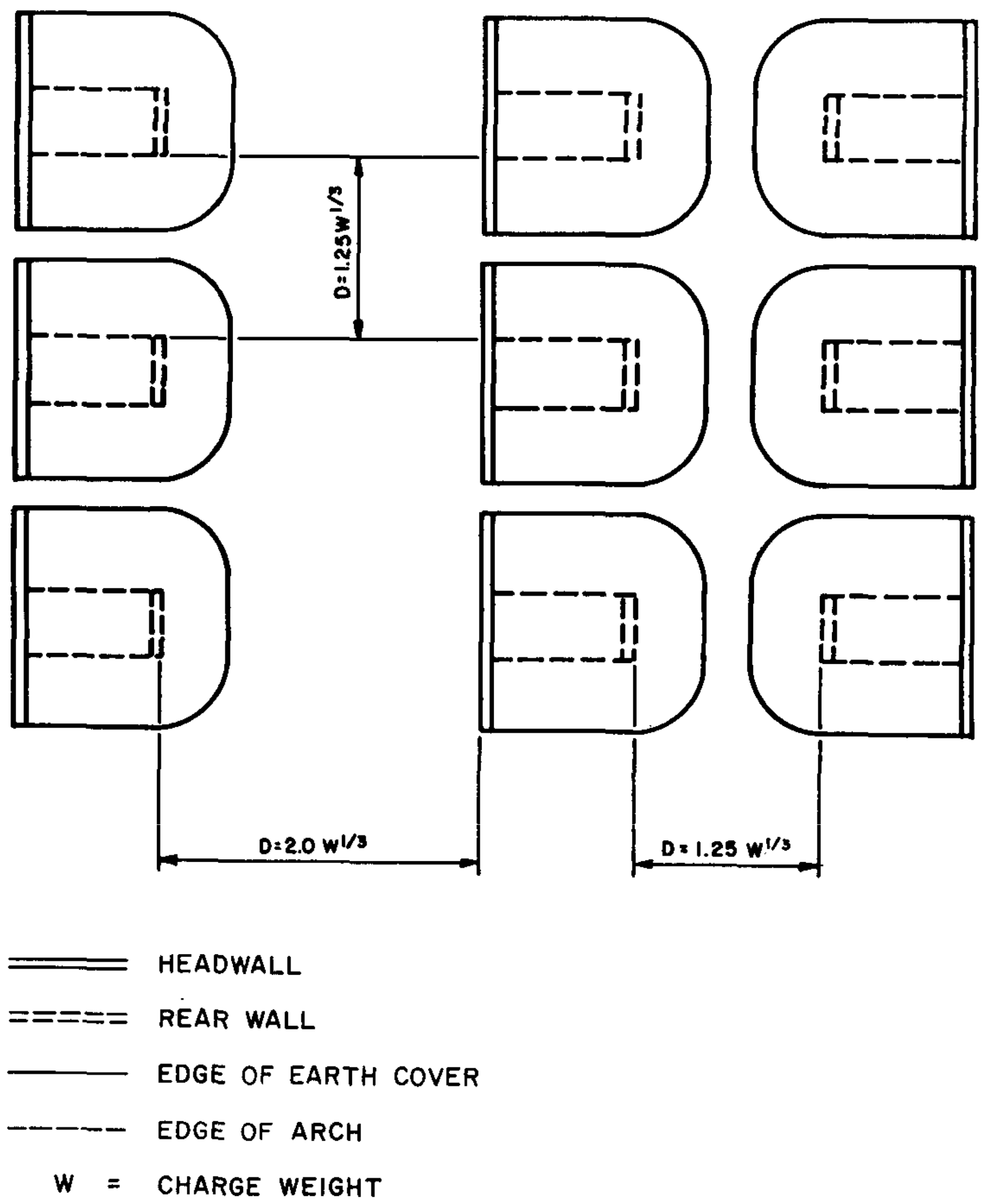


Figure 6-58 Minimum separation distances of standard magazines

BLAST VALVES

6-43. General

6-43.1. Applications

A prime concern of blast resistant structures is to restrict the flow of high pressures into or out of a structure. For donor structures pressures released may have to be restricted in order to limit pressures acting on adjacent structures to tolerable levels. Also, pressures leaking into acceptor structures must be limited to prevent pressure buildup beyond acceptable levels. In either case openings may have to be completely sealed to prevent the diffusion of contaminants.

The simplest, most economical way of limiting leakage pressure into or out of a protective structure is to restrict the number and size of air intake and exhaust openings. In a donor structure the leakage pressures may be further reduced by venting them through a stack. The stack increases the distance from the point of release of the pressure to the acceptor structure thereby attenuating the blast loading. Methods for predicting the pressures leaking out of a building and the pressure buildup within a building are discussed in Chapter 2. If the leakage pressure can not be reduced to acceptable levels or if contaminants are released during an explosion, the openings must be sealed with either blast valves or other protective closure device.

Blast valves may be either remote-actuated (closed mechanically by remote sensors) or blast-actuated (closed by the pressure wave itself). Both types can be non-latching or latching. A non-latching valve will open under negative pressures. A latching valve is one that can only be reopened manually. In addition, a blast-actuated valve can be double-acting. A double-acting valve will seal against the positive blast pressure, move in the opposite direction to seal against negative pressures and then reopen when pressures return to normal.

6-43.2. Remote-Actuated valves

Remote-actuated valves are dependent on external sensing devices which trigger the closure mechanism and close the valve before arrival of the blast wave. Actuating devices have been developed that are sensitive to the blast pressure of an explosion and react electrically to trigger protective closure systems. Other actuating devices sensitive to flash and thermal radiation are also available. The pressure sensing device is placed on a circumference at a predetermined radius from the valve (closer to ground zero) in order to compensate for time delays of electrical and mechanical control equipment and to permit valve closure before the blast arrives. Thermal sensors are designed to detect the characteristic pulse emitted by an explosion to prevent actuation by other sources such as lightning, fires, etc., which may occur with flash sensors. Remote-actuated valves present problems of protection against multibursts and button-up time for combustion-type equipment installed within the structure. In addition to problems of hardenability of the exposed sensor and suitability for multiburst operation, it is often necessary for sensors to initiate reopening of the valves as soon as dangerous pressures have subsided. In general, remote-actuated valves can not close fast enough to be effective during an H. E. explosion.

6-43.3. Blast-Actuated Valves

Self-acting blast-actuated valves, which close under the action of the blast pressure, overcome some of the disadvantages described above and present other factors to be considered. Since the valves are closed by pressure, they are not dependent upon sensing devices for operation. They can be automatically reopened after passage of the positive phase or latched closed during the negative phase if this is required. Double-acting valves automatically seal the opening during the negative phase. Since blast-actuated valves are closed by the blast, there is an inherent leakage problem to be considered due to the finite closing time. Although this is in the order of milliseconds for most valves, sufficient flow to cause damage may pass the port openings for certain valve designs and pressure levels. Effectiveness of closing at both high- and low-pressure ranges must be checked. The amount of blast entering depends on the closing time of the valve which, in turn, depends on the mass of the moving parts, disk diameter, and the distribution of pressures on both faces of the disk.

Ideally, a blast-actuated valve should possess the following characteristics: instantaneous closure or no leakage beyond the valve during and after closure, no rebound of moving parts, equal efficiency at all pressures below the design pressure, operational and structural reliability, minimum of moving parts, low-pressure drop through the valve at normal ventilation or combustion air flows, multiple detonation capability, durability, and easy maintenance.

Although instantaneous closure is not physically possible, the closing time can be reduced sufficiently to reduce the leakage to insignificant values. This may be accomplished by increasing the activating pressure-force-to--moving-mass ratio, decreasing the length of travel, permitting no deceleration during closure, and other methods.

6-43.4. Plenums

Blast valves, especially blast-actuated valves, will allow some pressure leakage. While the leakage pressure may not significantly increase the ambient pressure at some distance from the valve, there might be a "jetting effect" causing very high pressures in the immediate vicinity. A plenum may be used to protect against these high pressures. Two examples of plenums are, a plenum chamber and a plenum constructed of hardened duct work. A plenum chamber is a room where pressures attenuate by expansion. A hardened duct work plenum reduces the pressures by increasing the distance traveled (similar to the stack discussed in Section 6-43.1). For a donor structure, a plenum would only be necessary if contaminants are released during an explosion and the air must be filtered before being vented to the exterior. In that case, a plenum would be used to lower the pressures and prevent damage to the filters. A plenum in an acceptor structure would be used to prevent high leakage pressures from directly entering the building's air duct system and possibly causing local failure of the system.

Plenum chambers should be designed to avoid a buildup of interior pressure which would impede closing of the valve. The ratio of the area of the chamber cross section to that of the valve outlet should be preferably greater than 4:1 so as to diffuse leakage flow as quickly as possible. A chamber which has the prescribed necessary volume but has little change in area would act like a tunnel wherein entering pressures would encounter little attenuation in the length provided.

6-43.5. Fragment Protection

To ensure that blast valves function properly, they must be protected from fragments that may perforate the valves or jam them in an open position. One of several methods of accomplishing the required protection is by offsetting the opening from the blast valve by means of a blast-resistant duct or tunnel which would prevent the propagation of fragments to the blast valve. Another method is to enclose the blast valve in a concrete chimney. Other methods include using a debris pit or steel shields or debris cover attached directly to the blast valves.

6-44. Types of Blast Valves

6-44.1. General

Various types of blast valves have been developed and many of them are available commercially from suppliers both here and overseas (see Table 6-13). For present designs, air flow rates from about 300 cfm to 35,000 cfm can be obtained. Some valve designs are available in more than one size and can be either blast or remote-actuated. The pressure loss across the valve at the rated air flow is, in most cases, less than one inch of water.

The maximum incident pressure capability of available valves is above 50 psi and generally at least 100 psi. For shelter purposes, these valves may be overdesigned since the protection level for many shelters will be less than 50 psi. Except for cost factors, using a 100-psi valve for a 10-psi shelter design should not necessarily present technical problems since a valve must operate at all pressures below the maximum design level.

The best type of actuation (blast or remote) depends partly on the design pressure as previously discussed with regard to reaction time and operational considerations. For long arrival times (low pressure). a remote-actuated device can close the valve before the blast arrival whereas leakage may occur for a blast-actuated valve. At a high pressure (short arrival time),the closing time for the remote-actuated valve may be longer than the arrival time.

6-44.2. Blast Shield

If it can be assumed that the ventilation system can be closed off during a hazardous operation and kept closed until there is no danger of further blast, a relatively simple structural closure (blast shield), such as a steel plate can be utilized. This type of closure is especially useful for an exterior opening which would only be opened periodically, such as maintenance facilities where the release of toxic fumes from within the structure is required.

6-44.3. Sand Filter

In shelters where normal operational air requirements are small, sand filters are useful in the attenuation of leakage pressures. With this type of filter the pressures continue to increase throughout the positive phase. Thus this filter is good only for loads with a relatively short duration. A sketch of a 300-cfm sand filter is shown in Figure 6-59.

6-44.4. Blast Resistant Louvers

The blast resistant louver shown in Figure 6-60 is blast actuated and has a rated flow of 600 cfm at less than 1 inch water (gage) pressure drop. If a larger volume of air is required the louvers can be set into a frame and used in series (see Figure 6-61). Louvers can be used in acceptor structures subject up to 50 psi. A major drawback of the louvers is that there may be as much as 40 percent leakage across the valve, especially at lower pressures.

6-44.5. Poppet Valves

6-44.5.1. Applications

A poppet valve has many advantages. It has few moving.parts which might need repair. It can be blast or remote-actuated, latching, non-latching, or double-acting and is available in sizes from 600 to 5,000 cfm. A blast-actuated poppet valve has a very fast closing time, approximately 20 milliseconds, making it the only valve that reacts fast enough to be used in a containment cell.

A typical blast-actuated poppet valve is shown in Figure 6-62. The valve consists of an actuating plate, a valve seat, a backing plate that supports the actuating plate, and a spring which holds the valve open during normal operations. The normal air flow is around the actuating plate. A blast load will compress the spring and move the actuating plate against the valve seat thereby sealing the opening. As the blast pressure moves the actuating plate, some pressure will flow around the plate while it is closing.

The leakage pressures can be completely prevented by using a valve similar to the one schematically illustrated in Figure 6-63. In this valve the normal air flow is around the actuating plate through a length of duct. When the valve is subjected to a blast load, the pressure starts moving the actuating plate while at the same time flowing through the duct. The length of the duct must be long enough to ensure that the time it takes the blast pressures to flow through the duct (delay path) will be longer than the time required for the actuating plate to seal the valve. As an alternate to the long duct, an expansion chamber may be used to delay the blast.

6-44.5.2 Recommended Specification for Poppet Valves

Presented below is an example specification for the design, testing and construction of a poppet valve, but it may be adapted for other types of blast valves. This example specification is presented using the Construction Specification Institute (CSI) format and shall contain as a minimum the following:

1. APPLICABLE PUBLICATIONS: Except as otherwise stated herein all materials and work furnished in accordance with this specification shall comply with the following codes and standards.

1.1 Federal Specification (Fed. Spec.)

TT-P-37D	Paint, Alkyd Resin; Exterior Trim, Deep Colors
TT-P-645A	Primer, Paint Zinc Chromate Alkyd Type
TT-P-86G	Paint, Red- Lead-Base, Ready Mixed

1.2 American Society for Testing and Materials (ASTM)

A 53-81a	Pipe, Steel Black, and Hot-Dipped, Zinc-Coated, Welded and Seamless
D 2000-80	Rubber Products in Automotive Applications, Classification System For
E-709-80	Magnetic Particle Examination

1.3 American Institute of Steel Construction (AISC)

Specification for the Design, Fabrication and Erection of Structural Steel for Buildings
(Eighth Edition) with commentary

1.4 American Iron and Steel Institute (AISI)
304 17-7th

1.5 American Welding Society (AWS)
D.1.1 Structural Welding Code(latest edition)

2. SUBMITTALS: The following information shall be submitted for approval. Materials shall not be delivered to the site until approved shop drawings have been returned to the Contractor. Partial submittals, or submittals for less than the whole of any system made up of interdependent components will not be accepted. Submittals for manufactured items shall be manufacturer's descriptive literature, shop drawings, and catalog cuts that include the manufacturer's dimensions, capacity, specification reference, and all other information necessary to establish contract compliance.

2.1 Qualifications: The Contractor shall submit for approval, data to support the qualifications of the manufacturer and installer. A list of previously successfully completed jobs of a similar nature, indicating the name and address of the owner of the installation shall be included with the information.

2.2 Manufacturer's Data: Before executing any fabrication work, a completely marked and coordinated package of documents sufficient to assure full compliance with the drawings and specifications shall be submitted. The submittal shall include a complete technical evaluation of the capacity of each valve as described below. The drawings shall include: detailed fabrication and equipment drawings; assembly showing the complete installation, including methods for supporting the valves, subframes and frames; a listing of all materials and material specifications; surface finishes; fabrication, assembly and installation tolerances; locking devices and locking device release mechanisms; and a detailed sequence for installation of valves and frames in conjunction with other phases of construction. Structural fabrication drawings shall conform to the requirements of AISC and welding shall conform to AWS. Assembly and installation shall be based on field established conditions and shall be fully coordinated with architectural, structural, and mechanical systems. All aspects of any work developed in connection with the development of these valves shall be fully documented and become the property of the Government.

2.2.1 Standard Compliance: Where equipment or materials are specified to conform to requirements of the standards of organizations such as ANSI, NFPA, UL, etc., which use a label or listing as a method of indicating compliance, proof of such conformance shall be submitted for approval. The label or listing of the specified organization will be acceptable evidence. In lieu of the label or listing, the Contractor shall submit a notarized certificate from a nationally recognized testing organization adequately equipped and competent to perform such services, and approved by the Contracting Officer stating that the items have been tested with the specified organization's methods and that the item conforms to the specified organization's standards.

2.3 Preliminary Hydraulic Characteristics: Prior to Construction, submit with shop drawings an estimate of the hydraulic characteristics of each valve, under actual operating conditions. Ratings shall be based on tests or test data. All necessary corrections and adjustments shall be clearly identified. Corrections shall be established for actual altitude and air flow directions as shown on the drawings as well as hydraulic effects produced by mounting and/or connection of the valve.

2.4 Tests and Test Reports: Except as noted otherwise, the testing requirements for materials stated herein or incorporated in referenced documents, will be waived, provided certified copies of reports of tests from approved laboratories performed on previously manufactured materials are submitted and approved. Test reports shall be accompanied by notarized certificates from the manufacturer certifying that the previously tested material is of the same type, quality and manufacture as that furnished for this project.

2.5 Blast pressure analysis calculations and/or results of approved tests shall be submitted, for both the blast valve and subframe, for approval and shall conform to the requirement of the paragraph entitled, BLAST VALVE TESTS. The calculations and/or test results shall include all components of the valves and subframe subjected to the blast pressures. Calculations are not required for the frame embedded in the concrete. This frame shall conform to that shown on the drawings. However, the fabrications of the blast valve and associated subframe and embedded frame shall be the responsibility of the blast valve manufacturer. This manufacturer shall also be responsible for installation of this equipment.

2.6 Qualification of Welders: Before assigning any welder to work covered by this section of the specification, the Contractor shall submit the names of the welders to be employed on the work together with certification that each of these welders has passed the qualification test using procedures covered in AWS Standard D1.1.

2.7 Operational and Maintenance Manual: Operation and maintenance manuals shall be furnished by the Contractor. Complete manuals shall be furnished prior to the time of installation. The manual shall have a table of contents and shall be assembled to conform to the table of contents with tab sheets placed before instructions covering the subject.

2.8 Shop Test Reports: The Contractor shall furnish copies of shop inspection and test results of fabrication welding.

3. MATERIALS:

3.1 Structural steel pipe used for blast valve construction shall conform to ASTM A 53 seamless pipe.

3.2 All structural steel plate components of the valve shall consist of stainless steel and shall conform to AISI 304.

3.3 Spring type components shall consist of stainless steel and conform to AISI 17-7th, Condition C.

3.4 Blast Seal Material: Seals for blast valves shall conform to ASTM D 2000.

4. BLAST VALVE REQUIREMENTS: All blast valves shall be poppet type and shall have the following characteristics.

4.1 Pressure capacity: Each valve shall be capable of withstanding a sustained blast pressure of 100 psi as well as the impact force produced by the closing of the valve. The valve shall be designed to sustain elastic deformation when subjected to the above loads. The blast valve shall be capable of closing under a force of 15 pounds.

4.2 Temperature Capacity: Each valve shall be capable of satisfactory operation over a temperature range of 35° to 300° F.

4.3 Valve Actuation: Each valve shall be actuated by the blast overpressure. The valve shall be in the closed position 20 milliseconds after the onset of the blast front. The blast pressures are given on the drawings.

4.4 Valve Parameters: A minimum of 12-inch diameter blast valve shall be used. After the valve is closed by the blast overpressure, it shall remain in the closed position until manually opened. This shall require that the valve be equipped with a locking device which shall be located at the exterior side of the valve. A release mechanism for the locking devices shall be provided which shall be operated from a position immediately adjacent to the interior of the valve. Any penetration through the valve or the structure must be capable of being sealed against blast leakage through the penetration. The air flow capacity of the valves shall be 1500 SCFM (1710 ACFM) for the supply and return valves and 880 SCFM (1000 ACFM) for the exhaust valve. Total actual pressure drop across the valve with air movement in either direction shall not exceed one inch of water (gage). The valve and its operating parts shall be designed for a 20-year life and shall have an operating frequency of 10,000.

4.5 Blast Seals: Blast seals shall be provided between the face of each valve and subframe and between the subframe and the frame to provide a pressure tight condition. Seals shall be adjustable and easily replaceable. The seal shall be designed to be leakproof with a pressure differential across the seal of 100 psi.

4.5.1 Blast Seal Material: Seal material shall conform to ASTM D 2000. Four sets of blast seals shall be furnished with each valve. Three sets of the seals shall be packaged for long term storage.

4.5.2 Adhesive: Adhesive for blast seals shall be as recommended by the manufacturer of the seals. Sufficient adhesive shall be provided for installation of the packaged seals at a later date.

4.6 Field Removal: Blast valves shall have the capabilities of being completely field removed and disassembled.

5. FABRICATION:

5.1 Qualification of Manufacturer: The manufacture and installation of blast valves and frames shall be performed by the blast valve manufacturer who shall be fully responsible for valve operation. The manufacturer shall have complete facilities, equipment and technical personnel for the design, fabrication, installation and testing of complete blast valve assemblies.

5.2 General: The drawings indicate the location of the blast valves in the structure. The manufacturer shall carefully investigate the drawings and finished conditions affecting his work and shall design the units to meet the job condition and the dynamic loads. The blast valves shall be complete with gaskets, fasteners, anchors, mechanical operators, and all other equipment and accessories as required for complete installation.

5.3 Metalwork: Except as modified herein, fabrication shall be at a minimum, in accordance with the AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings. Welding of steel shall be in accordance with the requirements for AWS Specification D1.1. A welding sequence to reduce distortion and locked-up stresses to a minimum shall be used. All welded units shall be stress relieved. All welded members shall be post weld straightened free of twist and wind. Fabricated steel shall be well formed to shape and size, with sharp lines and angles. Exposed welds shall be ground smooth. Exposed surface of work, in place, shall have a smooth finish. Where tight fits are required joints shall be milled to a close fit. Corner joint shall be coped or mitered, well formed, and in true alignment. Permanent connections for all assemblies and components, except those requiring removal for installation and maintenance, shall be welded. Each valve and subframe shall be removable from the embedded frame.

5.3.1 Machining: Parts and assemblies shall be machine finished wherever necessary to insure proper fit of the parts and the satisfactory performance of the valves.

5.3.2 Weld Details: The types of edge preparation used for welds shall be chosen by the manufacturer to be the most suitable for the joint and position of welding. Where required, all groove welds shall be complete penetration welds with complete joint fusion. Groove weld edge preparations shall be accurately and neatly made. All full penetration groove joints shall be back-chipped and back welded where both sides are accessible. Where both sides are not accessible, backing strips not exposed to view may be left in place unless removal is required for clearance. Backing strip not removed shall be made continuous by welding ends and junctions.

5.3.3 Weld Tests: Inspection and tests of welds shall be as specified in AWS Specification D1.1. All welding shall be subjected to normal continuous inspection.

5.3.3.1 Nondestructive dye penetrator testing shall be performed for all welding in accordance with Method B of ASTM E 165 or ASTM E 709. Allowable defects shall conform to AWS Specification D1.1.

5.3.3.2 Penetration Welds: All full or partial penetration corners, tees and inaccessible butt joints shall be subjected to 100 percent ultrasonic examination. All penetration joints shall be considered to be tension joints. All tests shall be performed by a testing laboratory approved by the Contracting Officer. The testing laboratory shall be responsible for interpretation of the testing, which shall be certified and submitted in a written report for each test. In addition to the weld examination performed by the Contractor, the Contracting Officer reserves the right to perform independent examination of any welds at any time. The cost of all Government reexamination will be borne by the Government.

5.3.3.3 Correction of Defective Welds: Welds containing defects exceeding the allowable which have been revealed by the above testing shall be chipped or ground out for full depth and rewelded. This correction of the defected weld area and retest shall be at the Contractor's expense.

5.4 Metal Cleaning and Painting:

5.4.1 Cleaning: Except as modified herein, surfaces shall be cleaned to bare metal by an approved blasting process. Any surface that may be damaged by blasting shall be cleaned to bare metal by powered wire brushing or other mechanical means. Cleaned surfaces which become contaminated with rust, dirt, oil, grease, or other contaminants shall be washed with solvents until thoroughly clean.

5.4.2 Pretreatment: Except as modified herein, immediately after cleaning, steel surfaces shall be given a crystalline phosphate base coating; the phosphate base coating shall be applied only to blast cleaned, bare metal surfaces.

5.4.3 Priming: Treated surfaces shall be primed as soon as practicable after the pretreatment coating has dried. Except as modified herein, the primer shall be a coat of zinc chromate primer conforming to Fed. Spec. TT-P-645, or a coat of red lead paint, Type I or Type III conforming to Fed. Spec. TT-P-86G, applied to a minimum dry film thickness of 1.0 mil. Surfaces that will be concealed after construction and will require no overpainting for appearance may be primed with a coat of asphalt varnish, applied to a minimum dry film thickness of 1.0 mil. Damage to primed surfaces shall be repaired with the primer.

5.4.4 Painting: Shop painting shall be provided for all metalwork, except for non-ferrous metals and corrosion resistant metals and surfaces to be embedded in concrete. Surfaces to be welded shall not be coated within three inches of the weld, prior to welding. All machined surfaces in contact with outer surfaces and bearing surfaces shall not be painted. These surfaces shall be corrosion protected by the application of a corrosion preventive compound. Surfaces to receive adhesives for gaskets shall not be painted. Surfaces shall be thoroughly dry and clean when the paint is applied. No painting shall be done in freezing or wet weather except under cover; the temperature shall be above 45° F but not over 90° F. Paint shall be applied in a workmanlike manner and all joints and crevices shall be coated thoroughly. Surfaces which will be concealed or inaccessible after assembly shall be painted prior to assembly. Paint shall conform to Fed. Spec. TT-P37D.

6. BLAST VALVE TESTS:

6.1 Response Tests: The following static and dynamic response shall be performed to demonstrate the blast resistant capabilities of the blast value design. These tests shall be witnessed by the Contracting Officer.

6.1.1 Closure Time Test: Prior to shipment to the site, the Contractor shall perform a test to demonstrate that the closure of the blast valve will not exceed the 20 milliseconds specified. A suggested method for recording the valve closure is with the use of a high speed camera.

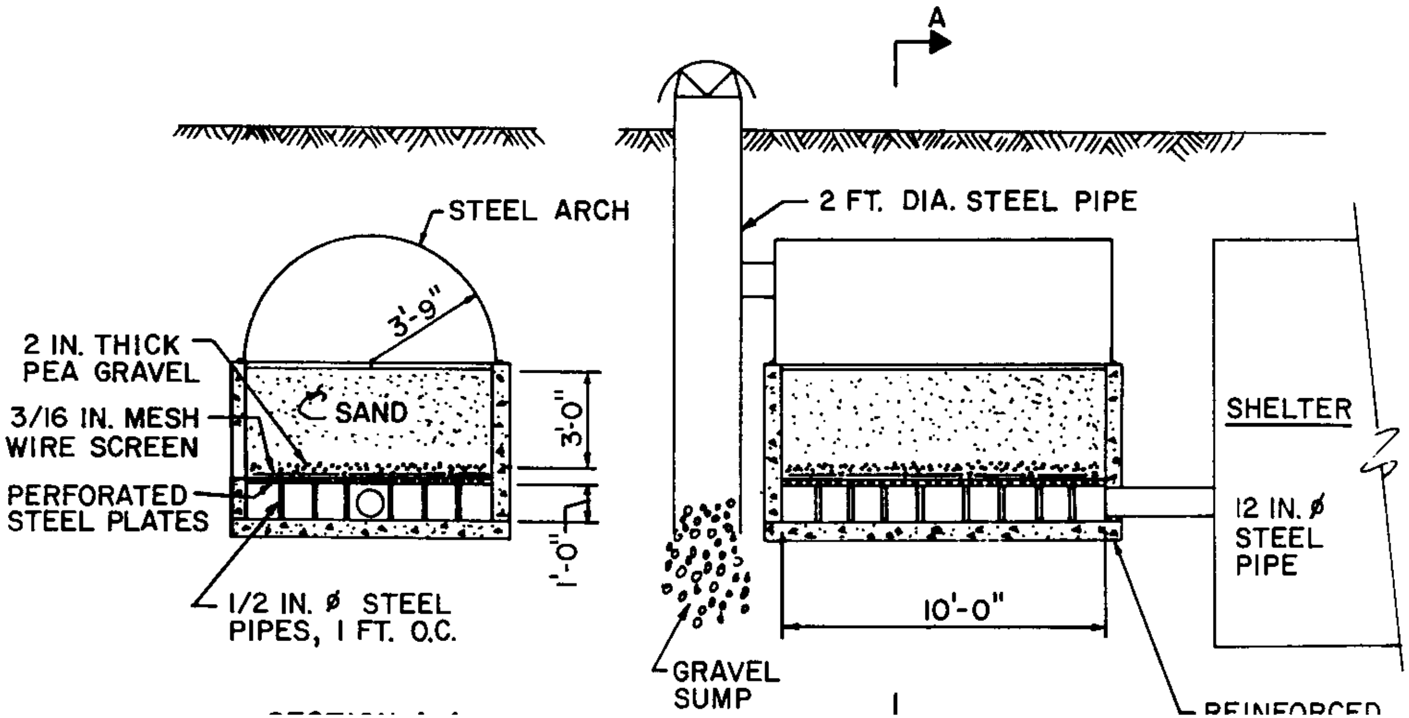
6.1.2 Static Pressure Test: Prior to shipment to the site, the Contractor shall perform a pneumatic test to demonstrate the static capacity of the blast valve design. The valve must sustain the pressure of 100 psi for a minimum of two hours. The total pressure loss during that period shall not exceed 1 psi.

6.1.3 Dynamic Pressure Test: Prior to shipment to the site, the Contractor shall perform a test to demonstrate the dynamic capacity of the blast valve design. This test shall simulate the combined effects of impact forces produced by the valve closure system and the blast load. This test may be replaced by design analyses which demonstrate that the head and frame of the valve shall have the capability to resist the stresses produced by the above forces. The blast load as indicated on the drawings shall be used for this analysis.

6.1.4 Blast Tests: If the effects of one or more of the above blast valve tests have been demonstrated by prior blast valve tests on similar valves, then the results of these tests shall be submitted for review; and the above test performances may not be required.

6.2 Test: After installation, a trip test shall be performed and demonstrated to the operating personnel.

6.3 Hydraulic Characteristics: Prior to shipment to the site, the Contractor shall perform a final test to establish the hydraulic characteristics of each valve and provide the necessary corrections and adjustments as stated previously.


A
STEEL ARCH
3'-9"
2 IN. THICK
PEA GRAVEL
SAND
3/16 IN. MESH
WIRE SCREEN
PERFORATED
STEEL PLATES
3'-0"
1'-0"
1/2 IN. ø STEEL
PIPES, 1 FT. O.C.
2 FT. DIA. STEEL PIPE
GRAVEL
SUMP
10'-0"
SHELTER
12 IN. ø
STEEL
PIPE

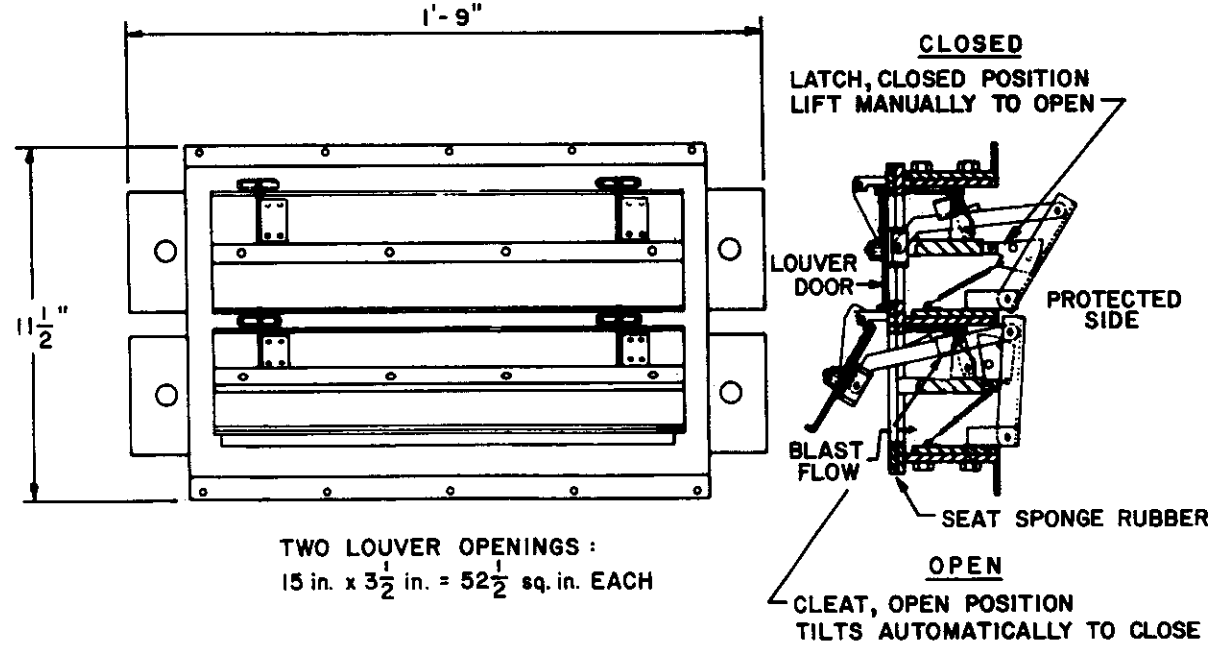


Figure 6-60 Blast-actuated louver

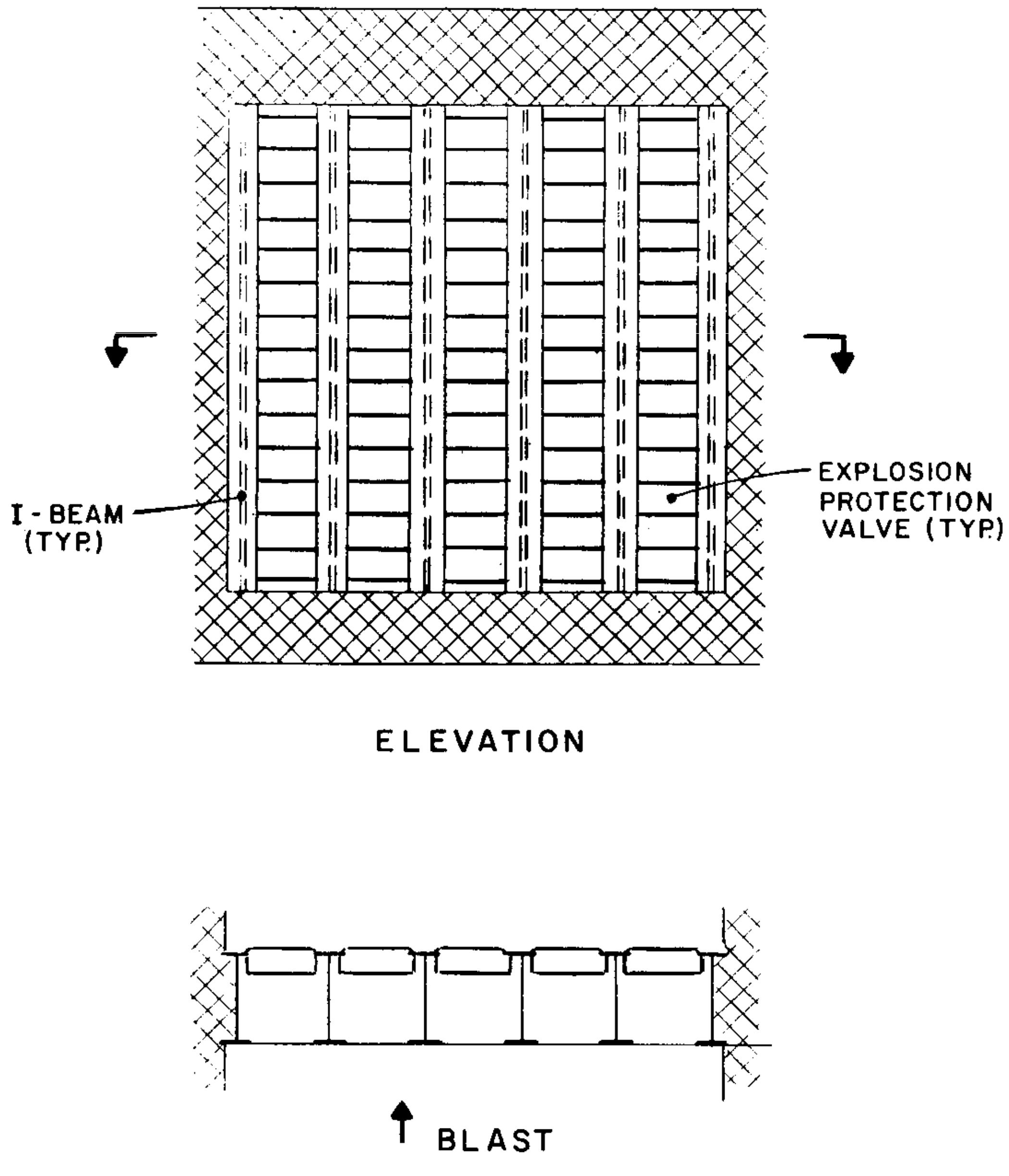


Figure 6-61 Arrangement of multiple louvers for a large volume of air

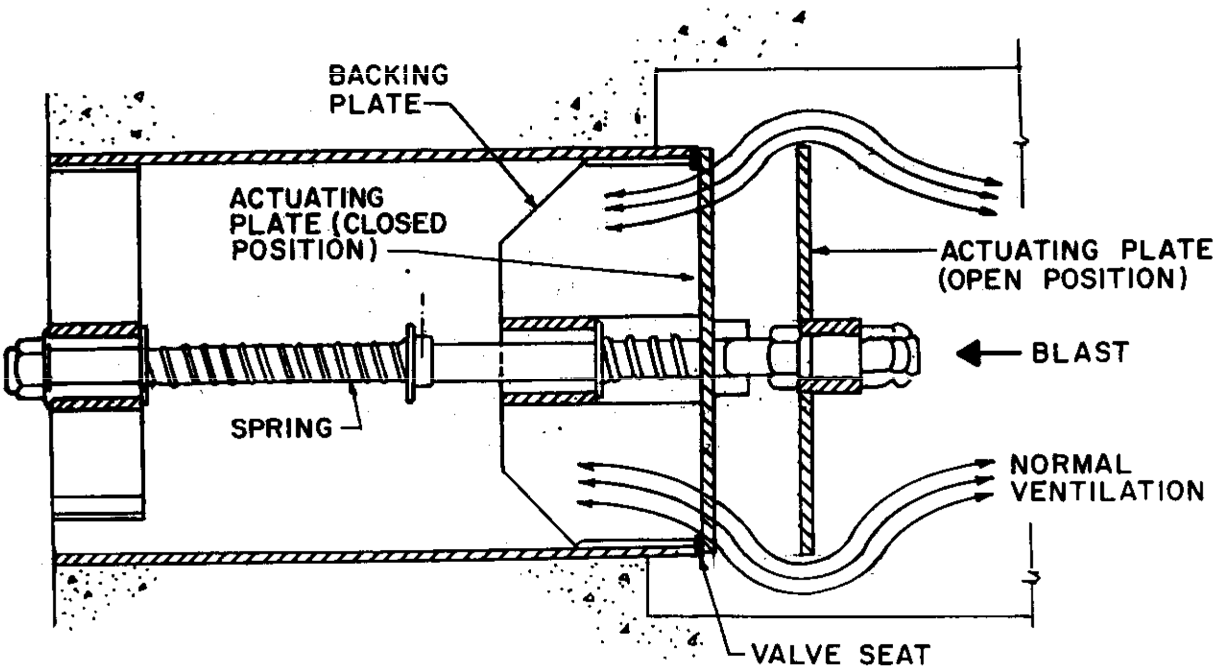


Figure 6-62 Typical blast-actuated poppet valve

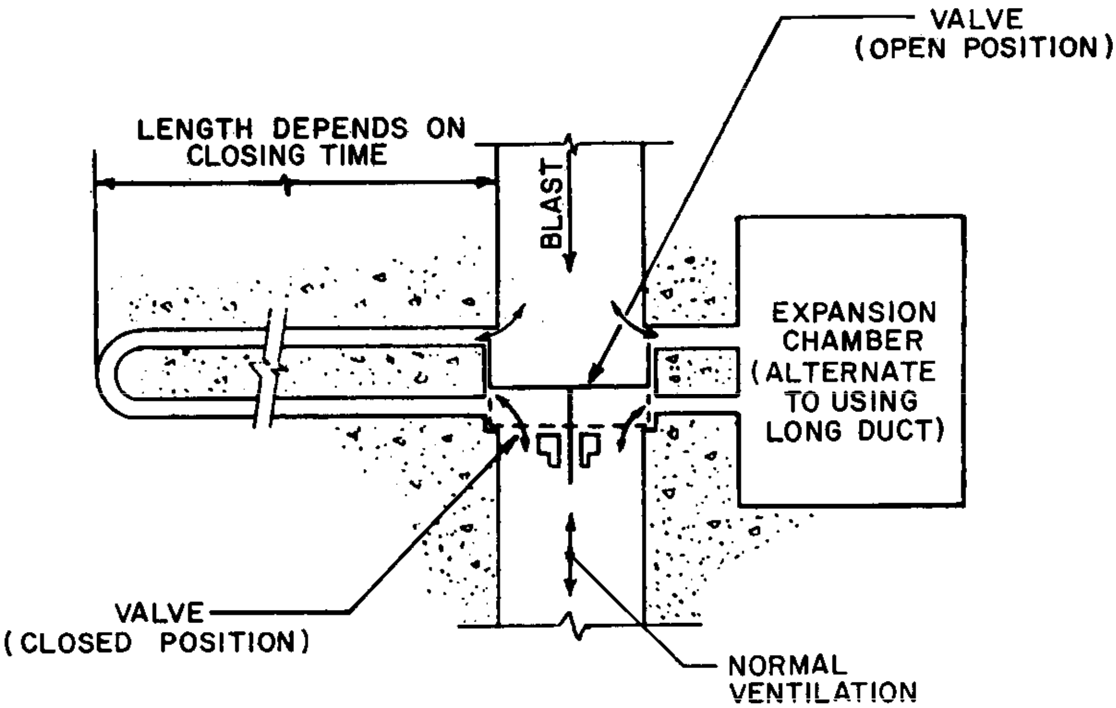


Figure 6–63 Time delay path

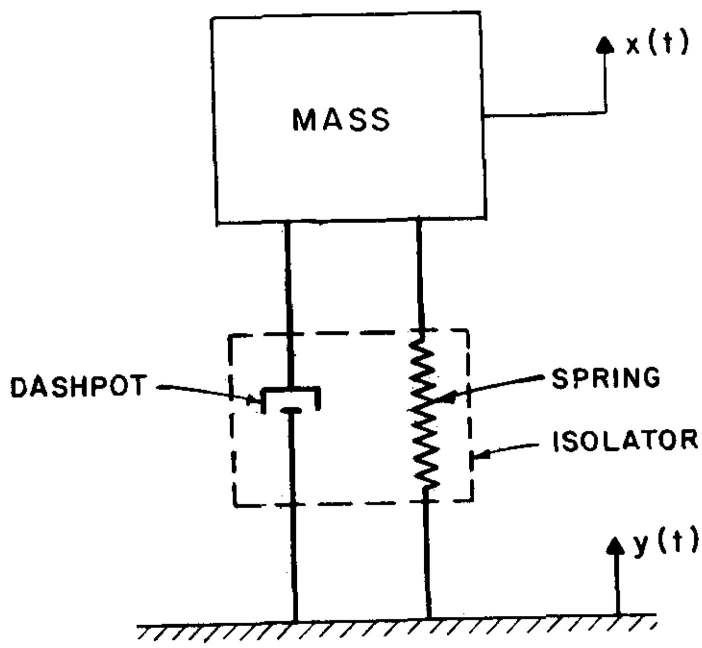


Figure 6-64 Idealized model of shock isolated mass

Table 6-13 Blast Valves

Name of Valve		Type (Actuated)	Blast Characteristics: Pressure (psi)	Blast Characteristics: Closing Time (ms)	Locking Mech-anisms	Air Flow Rates (cfm)	Tested
U.S. Army Chemical Corps M-1		Blast	100	*	Yes	300	Field
U.S. Army Chemical Corps E191R1		Blast	50	*	Yes	600	Shock tube and field
Office of Civil Defense	12"0	Blast and Remote	100	*	Yes	600	Field
	16"0		50	50	Yes	1,200	
	24"0		50	*	Yes	2,500	
Office of Corps of Engineers		Blast and Remote	*	20	Yes	5,000	Shock tube
Bureau of Yards and Docks		Remote	*	500	Yes	2,200 to 30,000	*
Temet USA, Inc.		Blast	100	20	Yes	2,250 to 3,700	Shock tube
Jaern and Plat	4"0		280	*		150	Field
	8"0					600	
	14"0					1,750	
Technical Facilities WS-107 A2		Remote	*	*		35,000	*
American Machine Foundry		Blast	100	*	Yes	80	*
Luwa		Blast	15 to 160	2	No	450	Shock tube
Suffield Experimental Stations		Blast	4 to 20	*		*	*
New Naval Civil Engineering Lab	Brechenridge	Blast	100	*	No	600	Compressed Air
	Bayles-Denny		5 to 100	*	No	180	
	Stephenson		5 to 104	*	No	125	
Artes Machinery (Sand Filtered)		Blast	100	*	No	300	Field

* Unknown

SHOCK ISOLATION SYSTEMS

6-45. Introduction

Previous sections have presented methods for the prediction of blast and fragment effects associated with the detonation of explosives and the design or analysis of structures to withstand these effects. In the design of shelters, an important part of the design process is to insure the survival of personnel and equipment. It is possible that the structure could withstand the air blast and ground shock effects but the contents be so severely damaged by structure motions that the facility could not accomplish its intended function. A similar problem is in the design of shelter type structures that houses sensitive explosives. These explosives must be protected from structure motions since these motions could result in initiation of the explosive. This section deals with the protection of vulnerable components from structure motions due to air blast and ground shock.

6-46. Objectives

The objective of shock isolation in protective design applications is to reduce the magnitude of motions transmitted by a vibrating structure to personnel or shock sensitive equipment. These motions must be attenuated to levels tolerable to personnel and to be various pieces of equipment used in the facility. A second consideration in some cases is to reduce the magnitude of motions transmitted by vibrating equipment to its supports. These latter motions can be significant for equipment mounted on shock isolated platforms.

The general functional objectives of a shock isolation system are:

1. Reduce input motions to acceptable levels.
2. Minimize rattle space requirements consistent with system effectiveness and cost.
3. Minimize coupling of horizontal and vertical motions.
4. Accommodate a spectrum of inputs of uncertain waveforms.
5. Limit the number of cycles of motion of the isolated body.
6. Support the system under normal operating conditions without objectionable motions.
7. Maintain constant attitude under normal operating conditions.
8. Accommodate changes in load and load distribution.
9. Maintain system vibration characteristics over long periods of time.
10. Interface properly with other components or parts.
11. Require minimum maintenance

6-47. Structure Motions

Ground shock results from the energy which is imparted to the ground by an explosion. Some of this energy is transmitted through the air in the form of air-blast-induced ground shock and some is transmitted through the ground as direct-induced ground shock. Both of these forms of ground shock when imparted to a structure will cause the structure to move in both a vertical and horizontal direction. Movement of the structure imparts motions to items attached to the structure's interior. Motion of interior items is obtained from a response spectrum. This is a plot giving the maximum responses (in terms of displacement, velocity, and acceleration) of all possible linear single-degree-of-freedom systems which may be attached to the structure due to a given input motion. Therefore, having the spectra for the structure and given input motion, the maximum response of any item within the structure is obtained based on the natural frequency of the item. Methods for preparing response shock spectra are presented in Chapter 2 of this manual.

In addition to motion of the structure as a whole, the exterior walls and roof respond to the direct application of the blast load. Methods for calculating the response of these elements are given in Chapter 3 of this manual using the parameters given in Chapter 4 and 5 for concrete and steel, respectively. Maximum displacements, velocities, and accelerations of these elements can be determined in a straightforward manner. These quantities can be used to determine effects on items attached or located near walls or roofs.

6-48. Shock Tolerance of Personnel and Equipment

The requirement for shock isolation is based upon the shock tolerance of personnel and/or critical items of equipment contained within the protective structure. If the predicted shock input exceeds the shock tolerance of personnel, a shock isolation system is required. If the shock input exceeds the shock tolerance of equipment, the equipment can either be ruggedized to increase its shock tolerance or it can be shock isolated. There are practical limits to ruggedization and the costs may exceed those of an isolation system. If the input does not exceed the shock tolerance of the equipment, it can be hard-mounted to the structure.

6-48.1. Personnel

The effects of structural motions on personnel depend on the magnitude, duration, frequency, and direction of the motion, as well as their position at the time of the loading. The shock tolerance of personnel is presented in Chapter 1 of this manual.

6-48.2. Equipment

In many cases, the need for shock isolation of equipment must be established before detailed characteristics of the system components are established. Further, because of the constraints of procurement procedures, shock isolation systems must be designed and built prior to specific knowledge of the equipment to be installed. In such instances, the choice lies between specifying minimum acceptable shock tolerances for the new equipment or using whatever data is available for similar types of equipment.

The most practical means of determining the shock tolerance of a particular item of equipment is by testing. However, even experimental data can be of questionable value if the test input motion characteristics differ greatly from those that would actually be experienced by the equipment. Since testing of equipment may not be practical in many cases due to the amount of time allotted from the inception of a project to its completion, procurement procedures, and cost limitations, it is often necessary to rely on data obtained from shock tests of similar items. The shock capacity of various types of equipment is presented in Chapter 1 of this manual.

6-49. Shock Isolation Principles

6-49.1. General Concepts

A full treatment of the problem of shock isolation systems is not possible in this manual. The following discussion provides an introduction to the subject and presents some of the important characteristics of shock isolation systems.

In general, the analytical treatment of shock isolation systems is based upon the principles of dynamic analysis presented in Chapter 3. In most cases, the actual system can be represented by a simplified mathematical model consisting of a rigid mass connected by a spring and dash pot as shown in Figure 6-64. The figure represents the simplest case, that of a single-degree-of-freedom system restrained to move in only one direction. Actually, an isolation system would have at least six degrees of freedom, i.e., three displacements and three rotations. Under certain conditions, these six modes can be uncoupled and the system analyzed as six single-degree-of-freedom systems.

The single-degree-of-freedom system shown in Figure 6-64 can be used to illustrate the importance of some of the parameters affecting the effectiveness of shock isolation systems in general. The isolator is represented by the linear spring and viscous damping device enclosed within the dotted square. The suspended mass is taken to be a rigid body. It is assumed that the base of the system is subjected to a periodic sinusoidal motion whose frequency is f. The undamped natural frequency of the system is f_n and is given by:

$$f_n = \frac{1}{2\pi}\left[\frac{386.4\ K}{W}\right]^{1/2} \tag{6-62}$$

where

f_n = natural frequency of vibration

K = unit stiffness of spring

W = weight supported by spring

When the frequency of the disturbing motion f is small compared to the natural frequency f_n of the single-degree-of-freedom system, the displacement of the mass is approximately equal to the displacement of the base. When the frequency of the base motion is several times that of the system, the motion of the mass is a small fraction of the base motion. When the ratio of frequencies become large (20 to 30), the system can not respond to the base motion to any significant degree. At frequency ratios near one, large motions of the mass are possible and the magnitude is strongly affected by the amount of damping in the system.

One obvious shock isolation approach is to use a low frequency suspension system so that the ratio of frequencies is always large. However, low frequency (referred to as soft) systems possess the undesirable characteristic of larger static and dynamic displacements and greater probability of coupling between modes of vibration. Although soft systems may be acceptable under some conditions, the obvious constraint that will preclude their use is a limit on the relative motion between the suspended mass and its supports or adjacent parts of the facility. This relative motion determines the amount of rattle space that must be provided to avoid impact between the mass and other fixed or moving parts of the facility.

The acceleration of the mass is a function of the forces applied to the mass by the spring and damping devices. In the case of a linear undamped spring, the force is a function of the relative displacement between the mass and its support. In viscous damping devices, the damping force is a function of the percent damping and the relative velocity between the mass and its supports. Acceleration limits for the critical items will impose restraints on spring stiffness and the amount of damping in the isolation system. In practice, a compromise combination of spring stiffness and damping is necessary to minimize input motions to the mass for a specified allowable rattle space or to minimize the rattle space required for specified allowable motions of the mass.

The need to avoid resonance (ratio of the frequency of the base motion to the natural frequency of the isolation system equal to one) is obvious. The structural motions resulting from an explosion are not steady-state sinusoidal in nature. However, these motions are of an oscillatory type and the displacement-frequency relationships discussed above are applicable.

A more detailed discussion of the effects of load duration, nonlinear springs, damping, and system frequency on response can be obtained from publications listed in the bibliography.

The basic objective in shock isolation is to select a combination of isolation system properties which will reduce the input motions to the desired level. In design, it is a straightforward process. System properties are assumed and an analysis is performed to determine its response to the input motions. If the shock tolerance and rattle space criteria are not satisfied, the system must be altered and the analysis repeated until the criteria are satisfied.

6-49.2. Single-Mass Dynamic Systems

A single mass system can have six degrees of freedom, that is, translation in three orthogonal axes and three rotations. The system can also be classified as coupled or uncoupled.

A coupled system is one in which forces or displacements in one mode will affect or cause a response in another mode. For example, a vertical displacement of a single rigid mass might also cause rotation of the mass. An uncoupled system, on the other hand, is one where forces or displacements in one mode do not generate a response in another mode. If the system is completely uncoupled, base translations in any one of the three orthogonal directions will cause translations of the mass in that direction only. Similarly, a pure rotation of the base about any one of the three orthogonal principal inertia axes with their origin through the mass center, will cause only pure rotations of the body about that axis. The principal inertia axes are those about which the products of inertia vanish. The principal elastic axes of a resilient element (isolator) are those axes for which an unconstrained element will experience a displacement collinear with the direction of the applied force. If the principal elastic axes and the principal inertia axes of the shock isolation system coincide with the origin or point of intersection of both sets of axes at the center of gravity of the mass, the modes of vibration are uncoupled. Such a system is also referred to as a balanced system.

In Figure 6-65, if all the springs have the same elastic stiffness, the elastic center will be located at point A, which, in this case, is at the center of the individual springs. If the suspended mass is of uniform density, its center of gravity is also located at A, and the system is uncoupled for motion input through the springs. Some systems may be uncoupled only for motions in a particular direction. If point B in Figure 6-65 is the center of gravity of the mass, a horizontal motion in the direction parallel to the X-axis of the structure would cause only a horizontal motion of the mass. A vertical motion of the structure would cause both a vertical and rotational motion of the mass. In this case, the vertical and rotational modes are coupled. If the center of gravity were located at point C, then vertical, horizontal and rotational modes are coupled. If the characteristics of the mass and shock isolation system are such that the modes of vibration can be uncoupled, the system can be analyzed as a series of independent single-degree-of-freedom systems. The response of each of these systems can be determined on the basis of input motions and isolator properties in a direction parallel to or about one of the principal inertia axes. The response in each one of these modes can be summed in various ways to obtain the total response of the system. The sum of the maximum responses would neglect differences in phasing and should represent an upper limit of the actual motions. It is recommended that the square root of the sum of the squares of the maximums (root mean square values) be used to represent a realistic maximum response since it is unlikely that response will occur simultaneously in all modes. Superposition of modal response is appropriate for elastic systems only.

A dynamically balanced shock isolation system offers advantages other than a simplification of the computation effort. A balanced system results in reduced motions during oscillation. As a result of the absence of coupling of modes in a balanced system and the usually small, if any, rotational inputs to the system in protective construction applications, rotational motions of the shock isolated mass will be minimized. This is particularly important for large masses where small angles of rotation can result in large displacements at locations far from the center of gravity.

Because of the advantages of a dynamically balanced system, various approaches are taken to minimize coupling of modes. One criterion is that frequencies in the six modes should be separated sufficiently to avoid resonance between modes. Because of the importance of minimizing rotational modes of response, it is suggested that extremely low stiffnesses in these modes should be avoided. For the analysis of multiple degrees of freedom, single-mass systems where the various modes of response are coupled, the modal method of analysis or numerical integration techniques can be utilized. The modal method of analysis requires solution of simultaneous equations of motion to determine characteristic shapes and frequencies of each mode and is limited to the elastic case. The numerical techniques do not require prediction of mode shapes and frequencies and will handle both elastic and inelastic response. If the dynamic system is also a multiple mass system, the above methods can be utilized to analyze the system. While an in-depth discussion is beyond the scope of this manual, a complete discussion of these methods can be found in publications listed in the bibliography.

6-49.3. Shock Isolation Arrangements

6-49.3.1. Individual versus Group Mounting

The two basic approaches to shock isolation in protective construction are to provide individually tailored systems for each component and to group together two or more items on a common platform. In the latter case, the system is selected to satisfy the requirements of the most critical item. In some cases, where the shock tolerance of the various items differs greatly, a combination of the two approaches may be the most effective solution. Although the relative location or size of some items may make individual mounts the more practical approach in certain cases, group mounting will generally be as reliable and the least costly solution.

Where personnel must be protected, a platform is the most practical solution. Except for extremely sensitive equipment, the shock tolerance of the personnel will govern the design of the system. The combination of personnel and equipment on the same platform will permit the personnel to move freely (however cautiously) between items of equipment. Where personnel are not required to be mobile, but rather may be able to remain seated while operating the equipment during hazardous periods, the shock tolerance of the personnel are greatly increased. This increased tolerance will reduce the shock isolation requirements while at the same time affording a higher degree of protection for personnel since they are protected from the unknown consequences of falling.

There are several advantages of group mounted systems. A group mounted system is less sensitive to variations in weights of individual items of equipment because of the larger combined weight of all items and the platform. With a number of items there is a greater flexibility of controlling the center of gravity of the total mass. In fact, ballast may be added to the platform to align the center of gravity with the principal axis to form a balanced system. A group mounted system generally requires less rattle space than several independently mounted items. Also, the interconnections between components is greatly simplified if they are all mounted on a single platform. Finally, an important advantage of group systems is cost. Individual mounts will require a large number of isolator units. Although larger, more costly, units are required for the group mounting system, fewer units are required and the cost per pound of supported load will be much lower.

6-49.3.2. Platform Characteristics

A platform for group mounted systems offers great flexibility in controlling the center of gravity of the supported masses to produce a balanced system where modes of vibration are uncoupled. Ballast may be securely anchored to the platform at locations which would move the center of gravity of the total mass to coincide with the elastic center of the isolation system. The determination of the weight and location of this ballast can be greatly simplified by uncoupling the effect of adding weight in the x and y directions of the principle elastic axes. This uncoupling can be accomplished by locating the ballast symmetrically about the x axis when moving the location of the center of gravity in the y direction. In this manner, the location of the center of gravity may be altered independently about the elastic center in the x and y directions. If for practical reasons the ballast cannot be located symmetrically about a principle axis, then the two directions must be considered simultaneously.

Providing additional ballast in excess of that required to balance the platform provides for future changes in equipment or the addition of new equipment without actually changing the isolation system. The springs will not require replacement nor will the structural members of the platform need to be increased in size. Additional equipment is placed on the platform and ballast is removed and/or relocated to balance the new equipment arrangement. To provide for future equipment changes, it is suggested that additional ballast equal to 25 percent of the weight of the equipment and the required ballast be distributed on the platform. The location of this ballast must not change the center of gravity of the existing balanced system. If future needs have been established, the platform and isolators would be designed for the future equipment. However, ballast would be provided to compensate for the weight of the future equipment and balance the system for the existing equipment.

The stiffness of the platform must be large enough to insure that the platform and associated group mounted equipment can be treated as a rigid body. This criterion is usually satisfied if the lowest natural frequency of any member of the platform is at least five (5) times the natural frequency of the spring mass system. When large, heavy items of equipment are involved, platforms meeting this stiffness criteria may not be practical. In such cases, the platform equipment configuration should be treated as a.multi-mass system.

6-49.3.3. Isolator Arrangements

There are many ways to support a shock isolated item. Some desirable features have been discussed previously in connection with dynamically balanced systems. The isolators may be positioned in many ways. The more important factors affecting the selection of an isolator arrangement are:

1. The size, weight, shape and location of the center of gravity of the suspended mass;
2. The direction and magnitude of the input motions;
3. Rotation of the lines of action of the devices should be small over the full range of displacements of the system to avoid system nonlinearities;

4. Coupling of modes should be minimized;

5. Static and dynamic instability must be prevented;

6. It is desirable in most cases, and necessary in some, that the system return to its nominal position;

7. Space available for the isolation system; and type of isolation devices used.

Some of the more common isolator arrangements are shown in Figures 6-67 and 6-68. The systems shown are assumed to have the same arrangements of isolators in a plane through the center of gravity (c.g.) and perpendicular to the surface of the page. The dynamically balanced system (intersection of the elastic axes and the principal inertia axes located at point A) shown in Figure 6-65, is probably the least common of all suspension systems.

6-49.3.4. Base-Mounted Isolation Systems

In Figure 6-66a, the mass is supported on four (4) isolators. These isolators must provide horizontal, vertical and rotational stiffnesses in order for the system to be stable under all possible motions. There will be coupling between horizontal displacements and rotations about horizontal axes. This arrangement and that shown in Figure 6-66b are appropriate in those cases where there are no convenient supports for horizontal isolators.

The arrangement of Figure 6-66b is preferred since the line of action of the isolators can be directed towards the c.g. of the mass to allow decoupling of some modes. As in the case of Figure 6-66a, the isolators must possess adequate stiffness in axial and lateral directions to insure stability under static and dynamic conditions.

In Figure 6-66c, the isolators are oriented parallel to the three orthogonal system axes. This arrangement provides system stability even when the isolators possess only axial stiffness. If the c.g. of the suspended mass is located as shown, decoupling of modes is possible. While the lines of action of the isolators pass through the c.g. under static conditions, response of the system to base motions will obviously alter its geometry. When the line of action of the isolators is changed due to displacement of the mass relative to its supports, coupling of the modes of vibration will be introduced. The degree of coupling is affected by the magnitude of the displacements and the length of the isolators. Consequently, isolator properties and arrangement should be selected so as to minimize the effects of displacements.

6-49.3.5. Overhead Pendulum Systems Using Platforms

Two arrangements of overhead pendulum shock isolation devices using platforms to support the sensitive components are shown in Figure 6-67. In both cases, the center of gravity of the suspended mass is relatively low. These types of suspension systems have been used extensively in protective structures for various conditions including individual small and large items, multiple items of various sizes as well as a combination of personnel and equipment supported on various sized platforms. The overhead pendulum system normally uses swivel joints at the points of attachment so that the system may swing freely. Horizontal input motions cause the pendulum to swing. Gravity provides the horizontal restoring force or stiffness. This force is a function of the total weight of the suspended mass. The natural frequency of vibration of the pendulum is a function of the length of the pendulum and is given by:

$$f_n = \frac{1}{2\pi}\left[\frac{386.4}{L}\right]^{1/2} \tag{6-63}$$

where

f_n = natural frequency of vibration
L = length of pendulum

Each pendulum arm includes an isolator which establishes the stiffness of the system in the vertical direction. These isolators can introduce nonlinearities and coupling between the pendulum and vertical spring modes. The system is linear for small angular displacements, that is, when the angular change Θ of the pendulum arm from the vertical position is approximately equal to the sine of the angle ($\Theta = \sin \Theta$). The system can be considered uncoupled if the pendulum frequency is not near one half of the vertical spring frequency. If the pendulum frequency is in the vicinity of one half the vertical frequency, the interchange of energy between the modes can lead to pendulum motions greatly exceeding those predicted by linear assumptions.

In a shock spectra maximum displacements occur at low frequencies, maximum velocities at intermediate frequencies, and maximum accelerations at high frequencies. Since most pendulum systems have low natural frequencies, they are displacement sensitive. These systems attain maximum displacements and minimum accelerations. Consequently, they will normally require greater rattle space than other systems while at the same time providing maximum protection against horizontal accelerations at minimum costs. It should be realized that for explosions, maximum displacements are comparatively small and can be accommodated. One of the main advantages of overhead pendulum systems is that they do not require horizontal stiffness elements. Their attractiveness is greatly diminished in those cases requiring horizontal damping because of large motions.

The swivel joint attaching the pendulum arm to the platform determines the location of the horizontal elastic axis of the system. Figure 6-58b illustrates two ways of varying the point of attachment of the pendulum arm to the platform. The horizontal elastic axis is raised to coincide with the center of gravity of the suspended mass at the equilibrium position and help minimize coupling between modes of response. At the left side of the platform the isolator is contained in a housing rigidly attached to the platform. At the right side, a structural member is rigidly attached to the platform and the isolator is included in the pendulum arm. In addition to supporting personnel and equipment, overhead pendulum systems can be used to shock-isolate building utilities. Individual utility runs may be isolated or several different utilities may be supported on a single platform. A single platform may cover an entire room and all building services may be supported. They would include a hung ceiling, lighting fixtures, utility piping, HVAC ducts, electrical cables and process piping. Of course, flexible connections must be used when connecting the services to the building or equipment.

6-50. Shock Isolation Devices

6-50.1. Introduction

A fundamental element of every shock isolation system is some sort of energy storage or energy dissipative device. These devices must be capable of supporting the items to be isolated under static and dynamic conditions and, at the same time, prevent transmission of any harmful shock loads to the items. In most cases, the isolator must have elastic force-displacement characteristics so that the system will return to a nominal equilibrium position after the dynamic loads have been applied. The desirable features of these devices include:

1. The dynamic force-displacement relationship of the isolator should be predictable for all directions in which it is required to provide stiffness.

2. The isolator should have low mass in order to minimize transmission of high frequency motions to the supported mass.

3. The frequency of the isolator should remain constant with changes in load, that is, its stiffness should vary in direct proportion to the load it supports. This allows the system to remain dynamically balanced throughout changes in the position of the supported mass.

4. The static position of the isolator should be adjustable so that the system can be leveled and returned to its nominal position should the suspended load change.

5. The isolator should have high reliability, long service life and low cost.

The various types of isolators used in most protective construction applications possess these characteristics in varying degrees. Any real isolator has some mass, and in some applications, the mass can be quite large and must be considered in the final analysis. Nonlinear force-displacement characteristics are often accepted to gain some other advantage. In energy dissipative systems, it may be necessary to provide other means of restoring the system to its original position. In general, most devices are some compromise combination of the desirable features which best suit the particular design situation.

The inclusion of energy dissipative (damping) devices in the isolation system offers several significant advantages, that is, damping can:

1. Reduce the severity of output motion response;

2. Reduce the effect of coupling between modes, thus reducing rattle space requirements;

3. Restore the system to an equilibrium position more quickly;

4. Decrease the sensitivity of the system to variations in input motions.

Damping can be provided internally in some isolation devices such as in liquid springs, but must be added externally in others such as those systems using helical coil springs. Different types of damping offer advantages and disadvantages which must be evaluated in the design process. A damping device may be effective in attenuating low frequency components of input motions but can increase the severity of high frequency components. Also, a damping device could prevent the system from returning to its nominal equilibrium position. Thus, care must be exercised in either designing a system employing isolator.devices possessing inherent damping characteristics or adding damping devices, if the isolation system is to perform properly.

There are numerous types of isolators which can be used to accomplish the shock isolating function. In the design of protective structures for explosions, the induced building motions are not usually severe and the maximum building displacements are relatively small. As a result, shock isolation systems using helical coil springs (Figure 6-68) are by far the most common system employed. The reasons for the extensive use of helical springs should be obvious from the discussion below. Other shock isolation devices which may also be used are presented, in less detail, below.

It should be noted that the protective design engineer does not furnish the design for the shock isolator. The engineer designs the shock isolation system to be used but does not design the isolators (in most cases, a helical coil spring). Rather, specifications are furnished which define the desired characteristics of the isolator. For a helical spring, the specifications may include some or all of the following: maximum load, maximum static deflection, maximum dynamic deflection, spring stiffness, maximum height, maximum diameter, and factors of safety regarding allowable stresses and bottoming of the spring. It must be realized that as the number of specified parameters increase, the options available to the spring manufacturer are decreased.

6-50.2. Helical Coil Springs

A helical coil spring is fabricated from bar stock or wire which is coiled into a helical form. Figure 6-68 illustrates several spring mounts.

The helical coil spring has numerous advantages and comparatively few disadvantages. The advantages are that the spring is not strain-rate sensitive, self-restoring after an applied load has been removed, resists both axial and lateral loads, linear spring rate and requires little or no maintenance. For most applications, the coil spring usually requires a larger space compared to other available shock isolators, and the spring cannot be adjusted to compensate for changes in loading conditions. If the weight of the supported object is changed, it is necessary to either change the spring or add additional springs. For most purposes, the helical coil spring can be considered to have zero damping. If damping is required, it must be provided by external means.

Helical coil springs may be used in either compression or extension. The extension springs are not subject to buckling and may offer a more convenient attachment arrangement. However, extension spring attachments are usually more costly and cause large stress concentrations at the point of attachment. For shock isolation applications, coil springs are generally used in compression. Buckling which can be a problem with compression springs, can be overcome by proper design or through the use of guides which are added either internally or externally to the coils. The discussion below will be concerned primarily with compression springs unless otherwise stated.

Helical coil springs may be mounted in two ways, the ends are either clamped or hinged. In most shock isolation applications, the spring ends are clamped since this method greatly increases the force required to buckle the spring. If space is at a premium, the energy storage capacity may be increased by nesting the springs (placing one or more springs inside the outermost spring). When nesting springs, it is advisable to alternate the direction of coils to prevent the springs from becoming entangled.

Although permanent set may be acceptable in some instances, it is normally required that the system return to its original position after being loaded. This can be accomplished in various ways, but the most common approach in the case of helical coil springs is to prevent inelastic action of the spring.

Helical coil springs are capable of resisting lateral load. While it is possible to use springs in this application, care should be exercised. There are possible arrangements which avoid subjecting the springs to this.type of loading.

While the actual design of the helical coil spring is done by the manufacturer, the engineer must be certain that the springs he is specifying can actually be obtained and the space he has allocated for the springs are sufficient. Therefore, preliminary spring sizes must be obtained by the engineer to suit his intended application. It is suggested that available manufacturer's data be used for this purpose.

6-50.3. Torsion Springs

Torsion springs provide resistance to torque applied to the spring. In shock isolation applications, the torque is usually the result of a load applied to a torsion lever which is part of the torsion spring system. A typical torsion spring shock isolation system is illustrated in Figure 6-69.

Since the axis of a torsion spring is normal to the direction of displacement, it can be used advantageously when space in the direction of displacement is limited. Torsion springs have linear spring rates, are not strain-rate sensitive, are self-restoring, and require little or no maintenance. Torsion springs can not be adjusted to compensate for changes in weight of shock isolation equipment, and damping must be provided by external means. The axial length of some types may preclude their use when space is limited.

There are three basic types of torsion springs; (1) torsion bars, (2) helical torsion springs, and (3) flat torsion springs. The type to be used will depend upon the space available and the capacity required. The torsion bar is normally used for light to heavy loads, the helical torsion spring for light to moderate loads, and the flat torsion spring for light loads. The torsion bar is the type most commonly found in protective structure applications and is most commonly used where large loads must be supported.

6-50.4. Pneumatic Springs

Pneumatic springs are springs whose action is due to the resiliency of compressed air. They are used in a manner similar to coil springs. The two basic types are the pneumatic cylinder with single or compound air chambers and the pneumatic bellows. The pneumatic cylinder is shown schematically in Figure 6-70.

Pneumatic springs have the advantage of being adjustable to compensate for load changes. The spring rate can be made approximately linear over one range of deflection but will be highly nonlinear over another. They are quite versatile due to the variety of system characteristics which can be obtained by regulation of the air flow between the cylinder chamber and the reservoir tank. Some of the possible variations include:

1. Velocity-sensitive damping by a variable orifice between chamber and reservoir;

2. Displacement-sensitive damping by a variable orifice controlled by differential pressure between chamber and reservoir;

3. A nearly constant height maintained under slowly changing static load by increasing or decreasing the system air content using an external air supply and a displacement-sensitive servo-system controlling inlet and exhaust valves;

   A constant height under widely varying temperatures achieved by the same system described for maintaining a constant height.

The disadvantages of pneumatic springs include higher cost and more fragile construction. They have a limited life span in comparison to mechanical springs and must be maintained. Also these springs provide resistance for axial loads only.

6-50.5. Liquid Springs

A liquid spring consists of a cylinder, piston rod, and a high pressure seal around the piston rod. The cylinder is completely filled with a liquid, and as the piston is pushed into the cylinder, it compresses the liquid to very high pressures. The configurations of liquid springs are divided into three major classes according to the method of loading. The classes are simple compression, simple tension and compound compression-tension. Although they are loaded in different ways, all three types function as a result of compression of the liquid in the cylinders. Schematics of the tension and compression types are shown in Figure 6-71. The compound spring is merely a more complex mechanical combination of the two basic types. The tension type is the more common in protective construction applications. The cylinders are often fitted with ported heads to guide the piston and provide damping. Damping can also be provided through the addition of drag plates to the piston rods.

Liquid springs are very compact devices with high, nearly linear, spring rates. They can be adjusted to compensate for load changes, are self-restoring and can absorb larger amounts of energy. They are highly sensitive to changes in temperature and fluid volume changes. Because liquid springs normally operate at high pressures, high quality, close tolerance seals are required around the piston. Friction between the seal and piston provides appreciable damping and increases the spring rate from 2 to 5 percent. Liquid springs are high pressure vessels requiring high quality materials and precision machine work, and as a result, they are expensive. However, they are difficult to equal as compact energy absorption devices.

6-50.6. Other Devices

6-50.6.1. Introduction

The helical, torsion, pneumatic and liquid springs are the more common types of isolators for larger masses. There are other devices especially suited for particular applications and .smaller loads. Some of these isolators are discussed below.

6-50.6.2. Belleville Springs

Belleville springs, also called Belleville washers or coned-disc springs, are essentially spring steel washers which have been formed into a slightly conical shape. A typical Belleville spring is illustrated in Figure 6-72.

The main advantage of Belleville springs over other types of springs is the ability to support large loads at small deflections with minimum space requirements in the direction of loading. They are useful in applications requiring limited shock attenuation and as back up systems to reduce shock in the event of bottoming of coil springs. They are relatively inexpensive and readily available in capacities up to 60,000 pounds. Changes in loading conditions are accommodated by the addition or removal of units.

6-50.6.3. Flat Springs

A flat spring is simply a steel beam or plate whose physical dimensions and support conditions are varied to provide the desired force displacement relationship. The two basic configurations are the simple spring with one element and leaf springs with multiple elements. Flat springs normally require only a limited amount of space in the direction of displacement and provide linear, non-strain-rate sensitive and self-restoring spring. They require little or no maintenance. Single element flat springs can be considered to have no damping while leaf springs will exhibit some damping due to the friction between individual elements.

6-50.6.4. Solid Elastomer Springs

Solid elastomer springs are made from rubberlike materials. They are often called shock mounts because of their wide use in shock isolation applications. They are normally used in medium to light duty applications and represent an. economical solution to the isolation of small items of equipment. However, these springs will allow only small displacements. These springs are fabricated from a wide variety of natural and synthetic rubbers and compounds and in numerous sizes and shapes to satisfy a wide range of applications. Because of the range in capacity and characteristics of commercially available units, only in unusual cases is it necessary to design a unit.

In most applications, the solid elastomer spring will require little space and exhibits good weight to energy storage ratios. Use of these springs requires consideration of the operating environment. The desirable properties of some elastomers can be significantly degraded when exposed to low or high temperatures, sunlight, ozone, water or petroleum products.

The response of elastomeric springs is nonlinear in most applications because of the nonlinear stress-strain properties of elastomers. The springs are self-damping because of the viscoelastic properties of the elastomers. They are almost always in compression because of bonding limitations. These springs will only permit comparatively small displacements.

6-51. Hardmounted Systems

Some items of equipment do not require shock isolation because the predicted motions at their point of attachment to the supporting structure does not exceed their shock tolerance. Those items can normally be hardmounted to the supporting structure. A hardmount is a method of attachment which has not been specifically designed to provide a significant reduction in the input motions to the equipment. Since all methods of attachment exhibit some flexibility, there is no precise division between shock isolators and hardmounts. Both types of devices will modify input motions to some degree. However, the modification of input motions produced by hardmounts will generally be small while shock isolators can greatly affect these motions.

In contrast to shock isolation systems, hardmounted systems will normally exhibit natural frequencies much higher than those corresponding to the lower modes of vibration of the supporting structure. Although this characteristic offers the advantage of reduced rattlespace, it also provides for the more efficient transmission of higher frequency components of the support structure motion to the attached item. Thus, it would appear that a more exact structural analysis is required for hardmounted systems in order to include higher modes of vibration. In practice, the need for exact analyses is at least partially offset by higher factors of safety in mount design and equipment shock tolerance. However, such an approach can lead to unrealistic attachment designs. A more practical approach is to choose, or design, attachments which limits the fundamental frequency of the hardmounted system. A lower frequency system provides some attenuation of higher frequency input motions, and reduces the possibility of resonance with high frequency motions resulting from stress wave reflections within structural elements. Although the choice of a natural frequency will depend on the properties of the supporting structure and the hardmounted equipment, fundamental frequencies in the range of 10 to 1000 cycles per second are reasonable for most applications. The approach chosen for hardmount design is normally a combination of higher safety factors and the use of lower frequency systems. The design will be based upon considerations of cost, importance of the item supported, the size and weight of the item, and the consequence of failure of the attachment system.

The use of shock spectra to define the input motions of hardmount systems is considered adequate for final design of all simple hardmount systems of a non-critical nature. It is also considered adequate for preliminary design of critical systems and those whose representation as a single degree of freedom system is questionable. However, it is recommended that the final design be performed using a more exact dynamic analysis wherever practical.

6-52. Attachments

6-52.1. Introduction

In a shelter type structure subjected to air blast and ground shock effects, all interior contents must be firmly attached to the structure. This attachment insures that the building contents will not be dislodged and become a source of injury to personnel or damage to critical equipment. The building contents would include not only equipment which is either shock isolated or hardmounted (attached directly to structure) but also the building utilities as well as interior partitions and hung ceilings. The building utilities would include all piping (such as process, potable water, sanitary, fire protection, etc.), HVAC ducts, electrical cables, light fixtures and electrical receptacles.

6-52.2. Design Loads

An object subjected to a shock loading produces an inertial force which acts through its center of gravity. The magnitude of this force is given by:

$$F = Wa$$

where

F = inertial force

W = Weight of object

a = acceleration in g's

Accelerations may be imparted to the object in one or more directions producing inertial forces in the respective directions. These inertial forces are resisted by the reactions developed at the object's supports. All inertial forces are assumed to be acting on the object concurrently. The support reactions are obtained by considering the static equilibrium of the system.

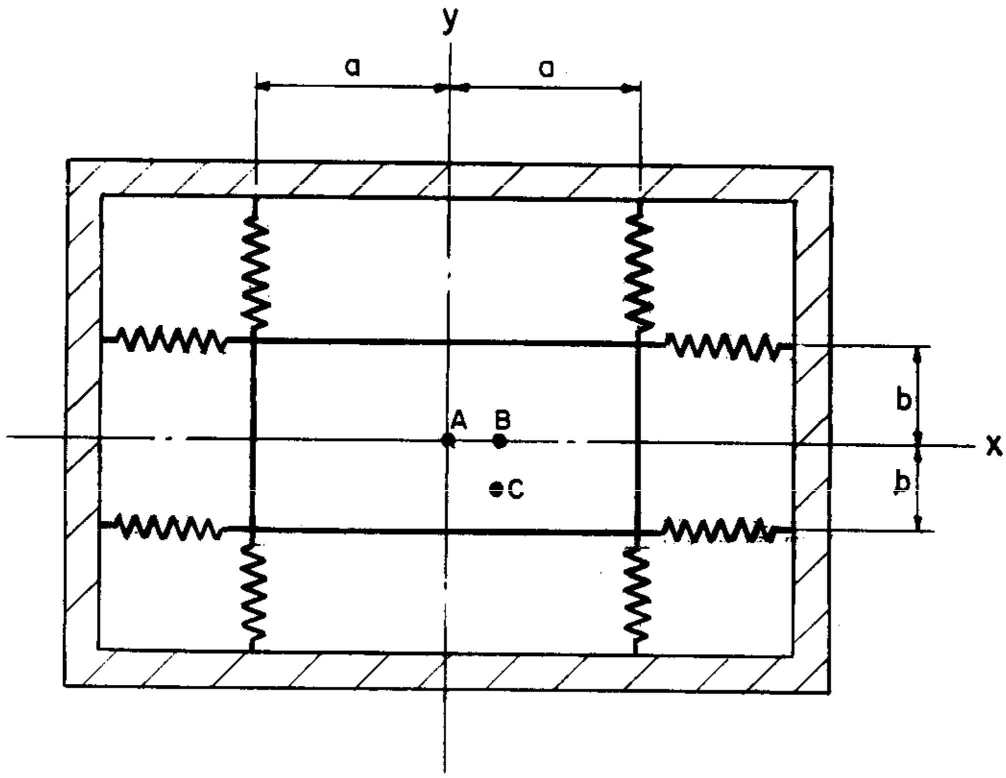


Figure 6-65 Shock isolation system

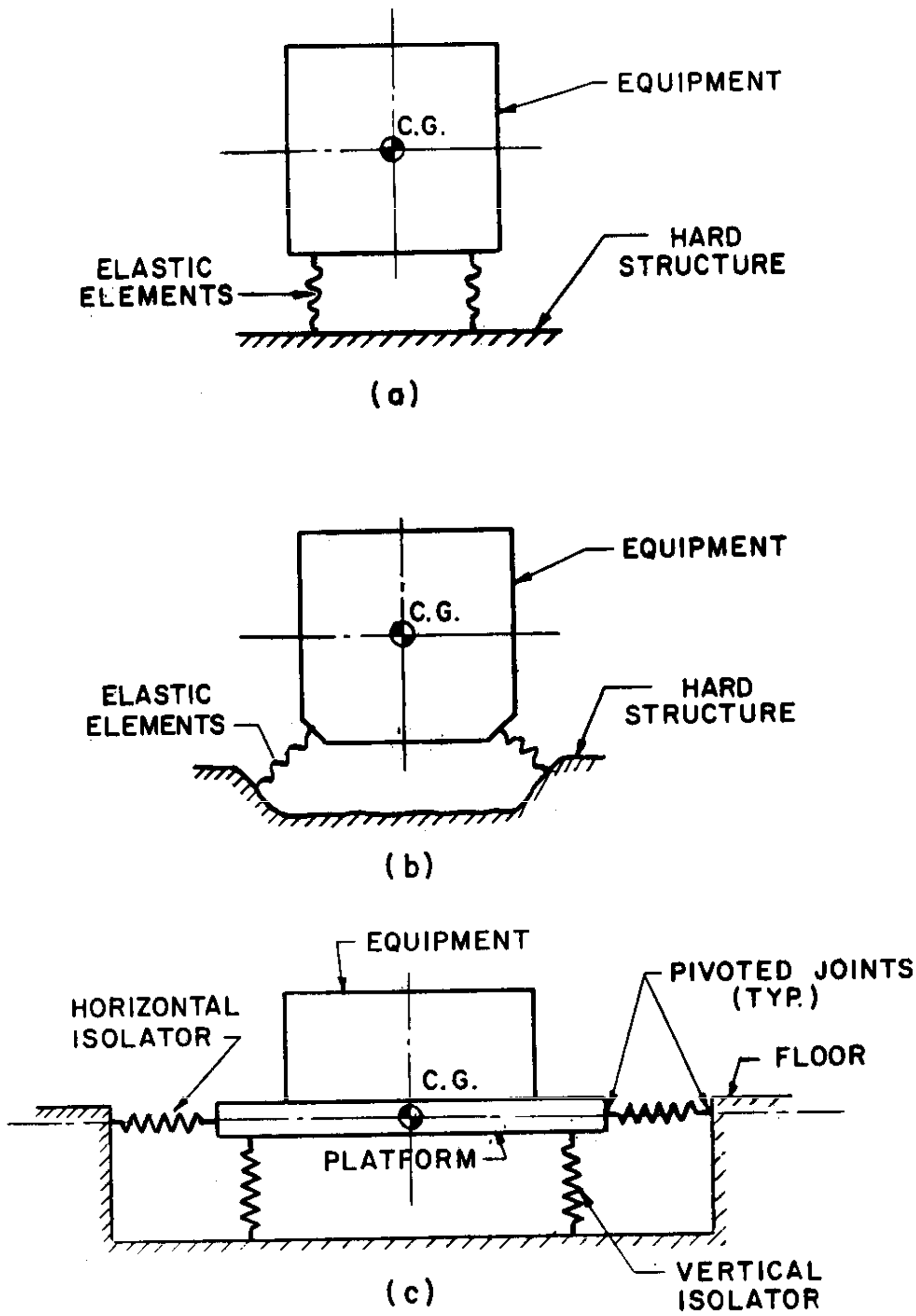


Figure 6–66 Base-mounted isolation systems configuraiton

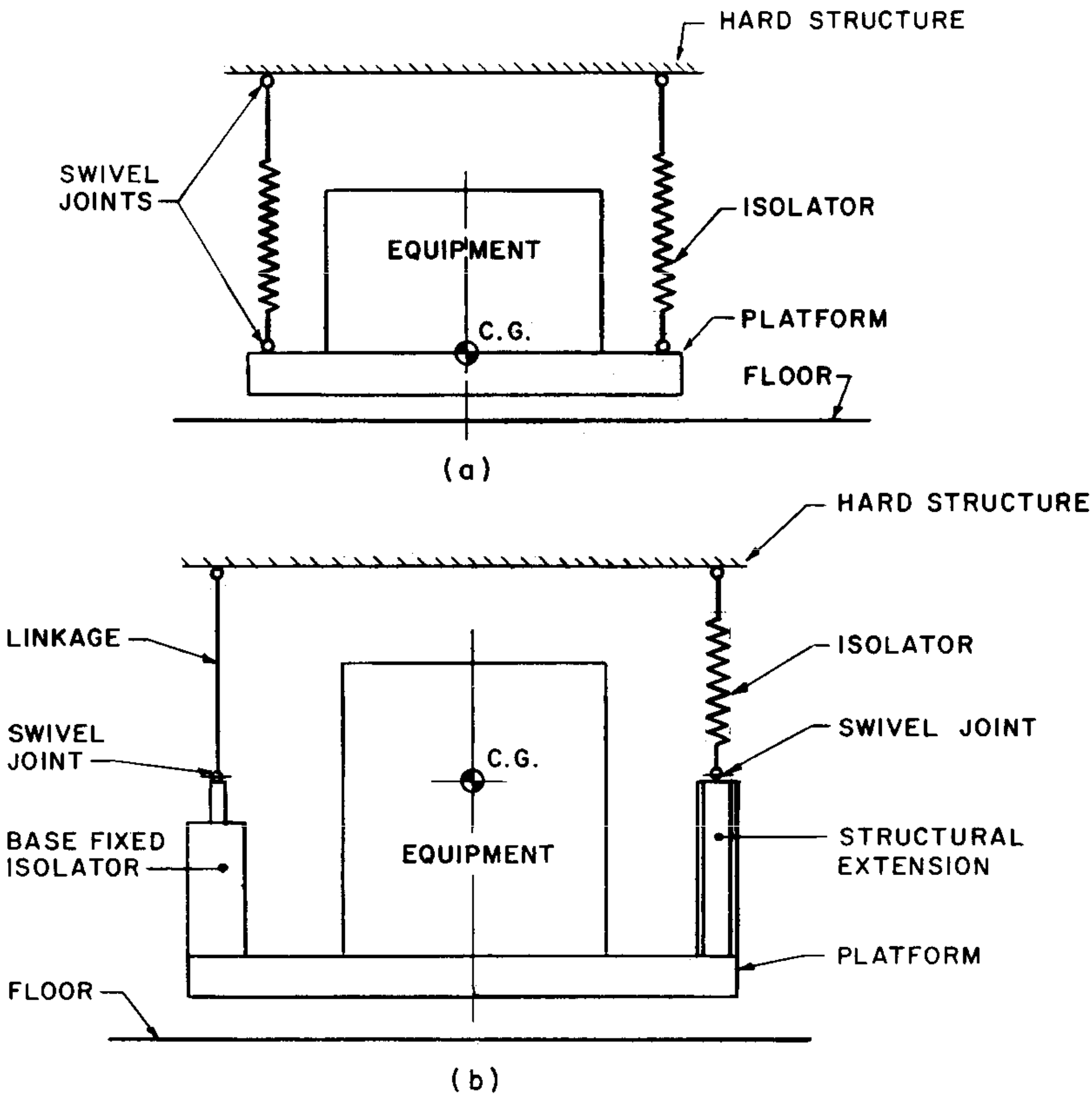


Figure 6–67 Overhead pendulum shock isolation systems using platforms

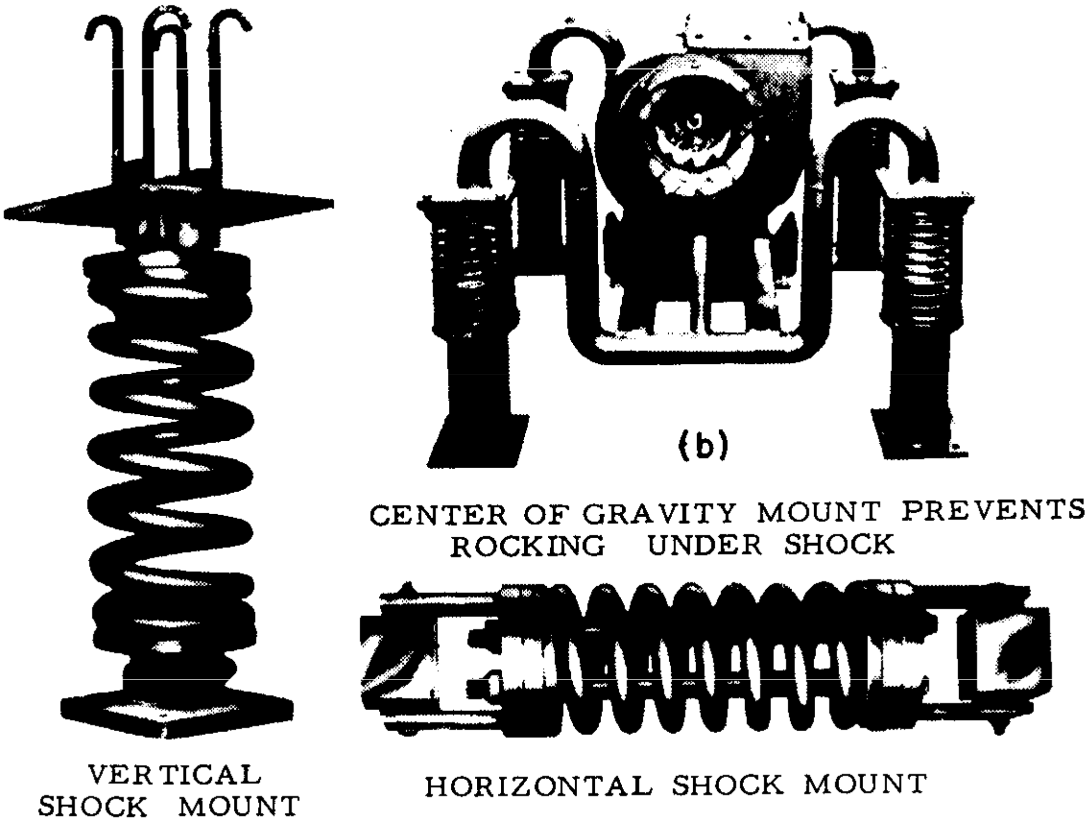


Figure 6-68 Helical compression spring mounts

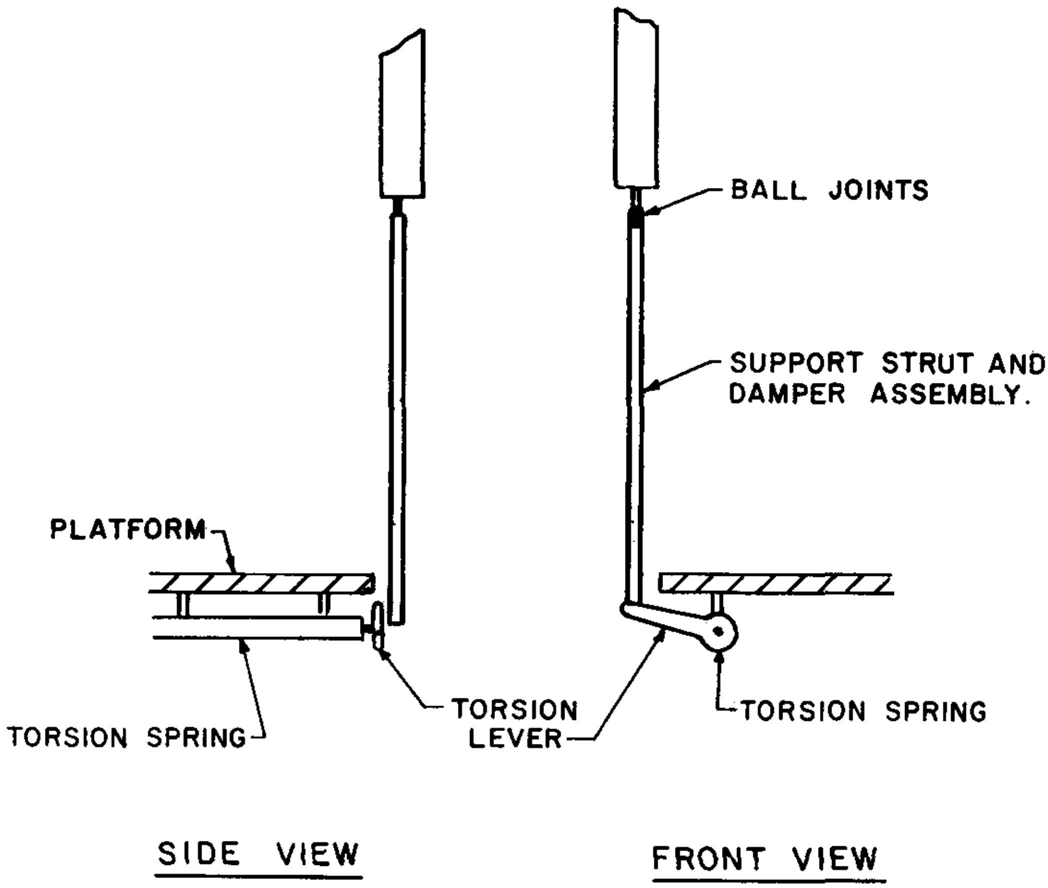
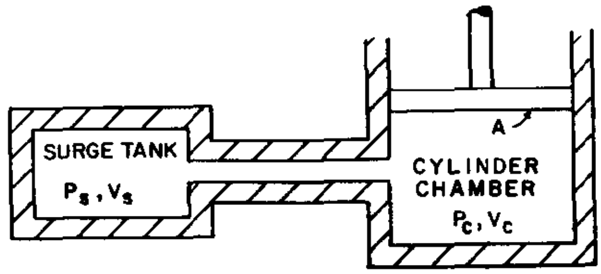


Figure 6-69 Typical torsion spring shock isolation system



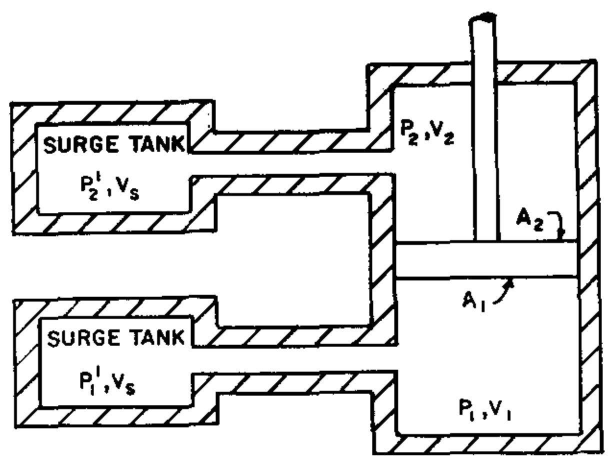


Figure 6–70 Schematic of single and double acting pneumatic cylinders

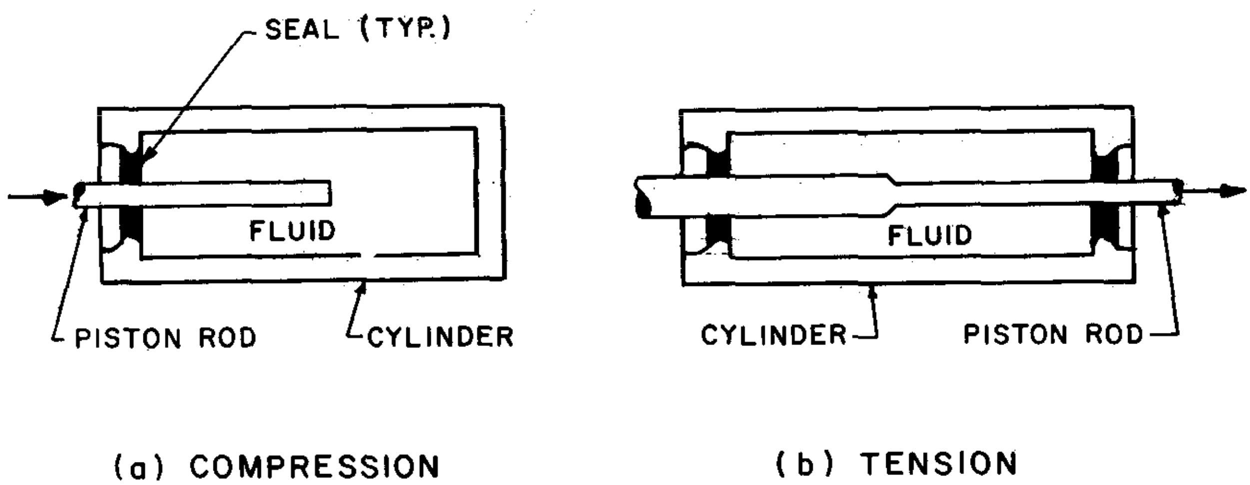


Figure 6–71 Schematic of liquid springs

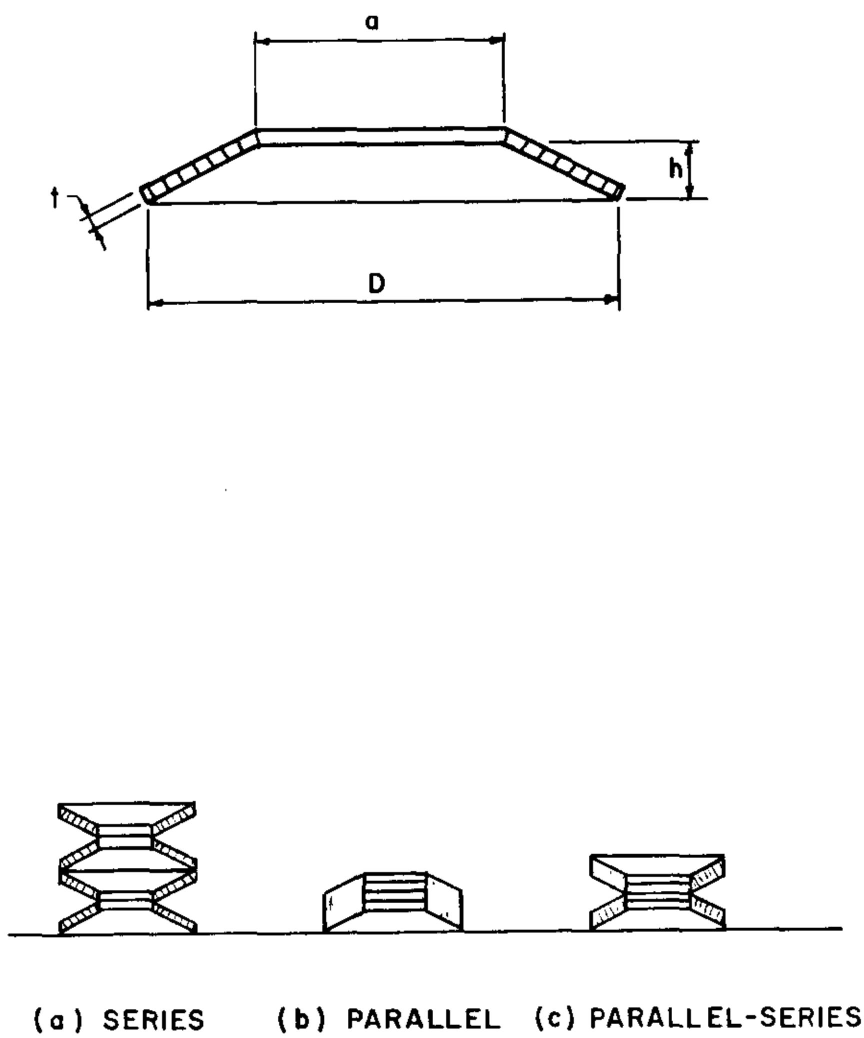


Figure 6-72 Belleville springs

APPENDIX 6B

LIST OF SYMBOLS

a	(1) acceleration (in./ms^2) (2) depth of equivalent rectangular stress block (in.) (3) long span of a panel (in.)
A	area (in.^{2})
A_a	area of diagonal bars at the support within a width b (in.^{2})
A_d	door area (in.^{2})
A_g	area of gross section (in^2)
A_n	net area of section (in.^{2})
A_o	area of openings (ft^2)
A_{ps}	area of prestressed reinforcement (in.^{2})
A_s	area of tension reinforcement within a width b (in.^{2})
A'_s	area of compression reinforcement within a width b (in.^{2})
A_s^-	area of rebound reinforcement (in.^{2})
A_{sH}	area of flexural reinforcement within a width b in the horizontal direction on each face (in^2)*
A_{sV}	area of flexural reinforcement within a width b in the vertical direction on each face (in^2)*
A_v	total area of stirrups or lacing reinforcement in tension within a distance, s_s or s_1 and a width b_s or b_1 (in.^{2}).
A_I, A_{II}	area of sector I and II, respectively (in.^{2})
b	(1) width of compression face of flexural member (in.) (2) width of concrete strip in which the direct shear stresses at the supports are resisted by diagonal bars (in.) (3) short span of a panel (in.)
b_s	width of concrete strip in which the diagonal tension stresses are resisted by stirrups of area A_v (in.)
b_1	width of concrete strip in which the diagonal tension stresses are resisted by lacing of area A_v(in.)
B	(1) constant defined in paragraph (2) peak blast overpressure capacity
C	shear coefficient

* See note at end of symmbols

c — (1) distance from the resultant applied load to the axis of rotation (in.)
(2) damping coefficient
(3) distance from extreme compression fiber to neutral axis (in.)

c_I, c_{II} — distance from the resultant applied load to the axis of rotation for sectors I and II, respectively (in.)

C_{cr} — critical damping

C_d — shear coefficient for ultimate shear stress of one-way elements

C_D — (1) drag coefficient
(2) coefficient for center deflection of glass

$C_D q$ — drag pressure (psi)

$C_D q_o$ — peak drag pressure (psi)

C_E — equivalent load factor

C_f — post-failure fragment coefficient (lb_2-ms_4/$in._8$)

C_H — shear coefficient for ultimate shear stress in horizontal direction for two-way elements*

C_L — leakage pressure coefficient

C_M — maximum shear coefficient

C_R — force coefficient for shear at the corners of a window frame

C_r — coefficient for effective resistance of glass

$C_{r\alpha}$ — peak reflected pressure coefficient at angle of incidence α

c_s — dilatational velocity of concrete (ft/sec)

C_s — shear coefficient for ultimate support shear for one-way elements

C_{sH} — shear coefficient for ultimate support shear in horizontal direction for two-way elements*

C_{sV} — shear coefficient for ultimate support shear in vertical direction for two-way elements*

C_T — coefficient for period of vibration for glass

C_u — impulse coefficient at deflection X_u (psi-ms^2/$in.^2$)

C'_u — impulse coefficient at deflection X_m (psi-ms^2/$in.^2$)

* See note at end of symnbols

C_v — shear coefficient for ultimate shear stress in vertical direction for two-way elements*

C_x — shear coefficient for the ultimate shear along the long side of window frame

C_y — shear coefficient for the ultimate shear along the short side of window frame

C_1 — (1) impulse coefficient at deflection X_1 (psi-ms^2/in.^{2})
(2) parameter defined in figure
(3) ratio of gas load to shock load

C'_1 — impulse coefficient at deflection X_m (psi-ms^2/in.^{2})

C_2 — ratio of gas load duration to shock load duration

d — distance from extreme compression fiber to centroid of tension reinforcement (in.)

d' — distance from extreme compression fiber to centroid of compression reinforcement (in.)

d_c — distance between the centroids of the compression and tension reinforcement (in.)

d_{co} — diameter of steel core (in.)

d_e — distance from support and equal to distance d or d_c (in.)

d_i — inside diameter of cylindrical explosive container (in.)

d_1 — distance between center lines of adjacent lacing bends measured normal to flexural reinforcement (in.)

d_p — distance from extreme compression fiber to centroid of prestressed reinforcement (in.)

d_1 — diameter of cylindrical portion of primary fragment (in.)

D — (1) unit flexural rigidity (lb-in.)
(2) location of shock front for maximum stress (ft)
(3) minimum magazine separation distance (ft)

D_o — nominal diameter of reinforcing bar (in.)

D_E — equivalent loaded width of structure for non-planar wave front (ft)

DIF — dynamic increase factor

DLF — dynamic load factor

* See note at end of symbols

e	(1) base of natural logarithms and equal to 2.71828... (2) distance from centroid of section to centroid of prestressed reinforcement (in.)
$(2E')^{1/2}$	Gurney Energy Constant (ft/sec)
E	modulus of elasticity
E_c	modulus of elasticity of concrete (psi)
E_m	modulus of elasticity of masonry units (psi)
E_s	modulus of elasticity of reinforcement (psi)
f	(1) unit external force (psi) (2) frequency of vibration (cps)
f'_c	static ultimate compressive strength of concrete at 28 days (psi)
f'_{dc}	dynamic ultimate compressive strength of concrete (psi)
f'_{dm}	dynamic ultimate compressive strength of masonry units (psi)
f_{ds}	dynamic design stress for reinforcement (psi)
f_{du}	dynamic ultimate stress of reinforcement (psi)
f_{dy}	dynamic yield stress of reinforcement (psi)
f'_m	static ultimate compressive strength of masonry units (psi)
f_n	natural frequency of vibration (cps)
f_{ps}	average stress in the prestressed reinforcement at ultimate load (psi)
f_{pu}	specified tensile strength of prestressing tendon (psi)
f_{py}	yield stress of prestressing tendon corresponding to a 1 percent elongation (psi)
f_s	static design stress for reinforcement (a function of f_y, f_u & ϕ (psi)
f_{se}	effective stress in prestressed reinforcement after allowances for all prestress losses (psi)
f_u	static ultimate stress of reinforcement (psi)
f_y	static yield stress of reinforcement (psi)
F	(1) total external force (lbs) (2) coefficient for moment of inertia of cracked section (3) function of C_2 & C_1 for bilinear triangular load

F_o	force in the reinforcing bars (lbs)
F_E	equivalent external force (lbs)
g	(1) variable defined in table 4-3 (2) acceleration due to gravity (ft/sec^2)
G	shear modulus (psi)
h	(1) charge location parameter (ft) (2) height of masonry wall
h'	clear height between floor slab and roof slab
H	(1) span height (in.) (2) distance between reflecting surface(s) and/or free edge(s) in vertical direction (ft)
H_c	height of charge above ground (ft)
H_c	scaled height of charge above ground (ft/lb$^{1/3}$)
H_s	height of structure (ft)
H_T	scaled height of triple point (ft/lb$^{1/3}$)
i	unit positive impulse (psi-ms)
i^-	unit negative impulse (psi-ms)
$\bar{i}_a$	sum of scaled unit blast impulse capacity of receiver panel and scaled unit blast impulse attenuated through concrete and sand in a composite element (psi-ms/lb$^{1/3}$)
i_b	unit blast impulse (psi-ms)
i_b	scaled unit blast impulse (psi-ms/lb$^{1/3}$)
i_{bt}	total scaled unit blast impulse capacity of composite element (psi-ms/lb$^{1/3}$)
$\bar{i}_{ba}$	scaled unit blast impulse capacity of receiver panel of composite element (psi-ms/lb$^{1/3}$)
$\bar{i}_{bd}$	scaled unit blast impulse capacity of donor panel of composite element (psi-ms/lb$^{1/3}$)
i_e	unit excess blast impulse (psi-ms)
i_r	unit positive normal reflected impulse (psi-ms)
i_r^-	unit negative normal reflected impulse (psi-ms)
i_s	unit positive incident impulse (psi-ms)

i_s^-	unit negative incident impulse (psi-ms)
I	moment of inertia (in.^{4})
I_a	average of gross and cracked moments of inertia of width b (in.^{4})
I_c	moment of inertia of cracked concrete section of width b (in.^{4})
I_g	moment of inertia of gross concrete section of width b (in.^{4})
I_m	mass moment of inertia (lb-ms^2-in.)
I_n	moment of inertia of net section of masonry unit (in.^{4}
j	ratio of distance between centroids of compression and tension forces to the depth d
k	constant defined in paragraph
K	(1) unit stiffness (psi-in for slabs) (lb/in/in for beams)(lb/in for springs) (2) constant defined in paragraph
K_e	elastic unit stiffness (psi/in for slabs) (lb/in/in for beams)
K_{ep}	elasto-plastic unit stiffness (psi-in for slabs) (psi for beams)
K_E	equivalent elastic unit stiffness (psi-in for slabs) (psi for beams) equivalent spring constant
K_L	load factor
K_{LM}	load-mass factor
$(K_{LM})_u$	load-mass factor in the ultimate range
$(K_{LM})_{up}$	load-mass factor in the post-ultimate range
K_M	mass factor
K_R	resistance factor
K_1	factor defined in paragraph
KE	kinetic energy
l	charge location parameter (ft)
l_p	spacing of same type of lacing bar (in.)

L	(1) span length (in.)* (2) distance between reflecting surface(s) and/or free edge(s) in horizontal direction (ft)
L_1	length of lacing bar required in distance s_1 (in.)
L_o	embedment length of reinforcing bars (in.)
L_s	length of shaft (in.)
L_w	wave length of positive pressure phase (ft)
L_w^-	wave length of negative pressure phase (ft)
L_{wb}, L_{wd}	wave length of positive pressure phase at points b and d, respectively (ft)
L_1	total length of sector of element normal to axis of rotation (in.)
m	unit mass (psi-ms^2/in.)
m_a	average of the effective elastic and plastic unit masses (psi-ms^2/in.)
m_e	effective unit mass (psi-ms^2/in.)
m_u	effective unit mass in the ultimate range (psi-ms^2/in.)
m_{up}	effective unit mass in the post-ultimate range (psi-ms^2/in.)
M	(1) unit bending moment (in-lbs/in.) (2) total mass (lb-ms^2/in.)
M_e	effective total mass (lb-ms^2/in.)
M_u	ultimate unit resisting moment (in-lbs/in.)
M_u^-	ultimate unit rebound moment (in-lbs/in.)
M_c	moment of concentrated loads about line of rotation of sector (in.-lbs)
M_A	fragment distribution parameter
M_E	equivalent total mass (lb-ms^2/in.)
M_{HN}	ultimate unit negative moment capacity in horizontal direction (in.-lbs/in.)*
M_{HP}	ultimate unit positive moment capacity in horizontal direction (in.-lbs/in.)*

* See note at end of symnbols

M_N	ultimate unit negative moment capacity at supports (in.-lbs/in.)
M_P	ultimate unit positive moment capacity at midspan (in.-lbs/in.)
M_{VN}	ultimate unit negative moment capacity in vertical direction (in.-lbs/in.)*
M_{VP}	ultimate unit positive moment capacity in vertical direction (in.-lbs/in.)*
n	(1) modular ratio (2) number of time intervals (3) number of glass pane tests
N	number of adjacent reflecting surfaces
N_f	number of primary fragments larger than W_f
p	reinforcement ratio equal to (A_s/bd) or (A_s/bd_c)
p'	reinforcement ratio equal to (A'_s/bd) or (A'_s/bd_c)
p_b	reinforcement ratio producing balanced conditions at ultimate strength
p_p	prestressed reinforcement ratio equal to A_{ps}/bd_p
p_m	mean pressure in a partially vented chamber (psi)
p_{mo}	peak mean pressure in a partially vented chamber (psi)
p_H	reinforcement ratio in horizontal direction on each face*
p_T	reinforcement ratio equal to $p_H + p_V$
p_V	reinforcement ratio in vertical direction on each face*
p(x)	distributed load per unit length
P	(1) pressure (psi) (2) concentrated load (lbs)
P^-	negative pressure (psi)
P_i	interior pressure within structure (psi)
P_i	interior pressure increment (psi)
P_f	fictitious peak pressure (psi)
P_o	peak pressure (psi)

* See note at end of symnbols

P_r	peak positive normal reflected pressure (psi)
P_r^-	peak negative normal reflected pressure (psi)
P_{ra}	peak reflected pressure at angle of incidence a (psi)
P_s	positive incident pressure (psi)
P_{sb}, P_{se}	positive incident pressure at points b and e, respectively (psi)
P_{so}	peak positive incident pressure (psi)
P_{so}^-	peak negative incident pressure
P_{sob},P_{sod},P_{soe}	peak positive incident pressure at points b, d, and e, respectively (psi)
P(F)	probability of failure of glass pane
q	dynamic pressure (psi)
q_b, q_e	dynamic pressure at points b and e, respectively (psi)
q_o	peak dynamic pressure (psi)
q_{ob},q_{oe}	peak dynamic pressure at points b and e, respectively (psi)
r	(1) unit resistance (psi) (2) radius of spherical TNT (density equals 95 lb/ft^3 charge (ft))
r^-	unit rebound resistance (psi, for panels) (lb/in for beams)
Δr	change in unit resistance (psi, for panels) (lb/in for beams)
r_d	radius from center of impulse load to center of door rotation (in.)
r_e	elastic unit resistance (psi, for panels) (lb/in for beams)
r_{ep}	elasto-plastic unit resistance (psi, for panels) (lb/in for beams)
r_s	radius of shaft (in.)
r_u	ultimate unit resistance (psi, for panels) (lb/in for beams)
r_{up}	post-ultimate unit resistant (psi)
r_1	radius of hemispherical portion of primary fragment (in.)
R	(1) total internal resistance (lbs) (2) slant distance (ft)

R_f	distance traveled by primary fragment (ft)
R_g	uplift force at corners of window frame (lbs)
R_l	radius of lacing bend (in.)
R_A	normal distance (ft)
R_E	equivalent total internal resistance (lbs)
R_G	ground distance (ft)
R_u	total ultimate resistance
R_I, R_{II}	total internal resistance of sectors I and II, respectively (lbs)
s	sample standard deviation
s_s	spacing of stirrups in the direction parallel to the longitudinal reinforcement (in.)
s_l	spacing of lacing in the direction parallel to the longitudinal reinforcement (in.)
S	height of front wall or one-half its width, whichever is smaller (ft)
S_E	strain energy
t	time (ms)
Δt	time increment (ms)
t_a	any time (ms)
t_b, t_e, t_f	time of arrival of blast wave at points b, e, and f, respectively (ms)
t_c	(1) clearing time for reflected pressures (ms) (2) container thickness of explosive charges (in.)
t_d	rise time (ms)
t_E	time to reach maximum elastic deflection
t_m	time at which maximum deflection occurs (ms)
t_o	duration of positive phase of blast pressure (ms)
t_o^-	duration of negative phase of blast pressure (ms)
t_{of}	fictitious positive phase pressure duration (ms)
t_{of}^-	fictitious negative phase pressure duration (ms)

t_r	fictitious reflected pressure duration (ms)
t_u	time at which ultimate deflection occurs (ms)
t_y	time to reach yield (ms)
t_A	time of arrival of blast wave (ms)
t_1	time at which partial failure occurs (ms)
T	(1) duration of equivalent triangular loading function (ms) (2) thickness of masonry wall
T_c	thickness of concrete section (in.)
T_c	scaled thickness of concrete section (ft/lb$^{1/3}$)
T_g	thickness of glass (in.)
T_i	angular impulse load (lb-ms-in.)
T_N	effective natural period of vibration (ms)
T_r	rise time (ms)
T_s	thickness of sand fill (in.)
$\bar{T}_s$	scaled thickness of sand fill (ft/lb$^{1/3}$)
u	particle velocity (ft/ms)
u_u	ultimate flexural or anchorage bond stress (psi)
U	shock front velocity (ft/ms)
U_s	strain energy
v	velocity (in./ms)
v_a	instantaneous velocity at any time (in./ms)
v_b	boundary velocity for primary fragments (ft/sec)
v_c	ultimate shear stress permitted on an unreinforced web (psi)
v_f	maximum post-failure fragment velocity (in./ms)
v_f(avg.)	average post-failure fragment velocity (in./ms)
v_i	velocity at incipient failure deflection (in./ms)
v_o	initial velocity of primary fragment (ft/sec)
v_r	residual velocity of primary fragment after perforation (ft/sec)

v_s striking velocity of primary fragment (ft/sec)

v_u ultimate shear stress (psi)

v_{uH} ultimate shear stress at distance d_e from the horizontal support(psi)*

v_{uV} ultimate shear stress at distance de from the vertical support (psi)*

V volume of partially vented chamber (ft^3)

V_d ultimate direct shear capacity of the concrete of width b (lbs)

V_{dH} shear at distance d_e from the vertical support on a unit width (lbs./in.)*

V_{dV} shear at distance d_e from the horizontal support on a unit width (lbs/in.)*

V_o volume of structure (ft^3)

V_s shear at the support (lb/in, for panels) (lbs for beam)

V_{sH} shear at the vertical support on a unit width (lbs/in.)*

V_{sV} shear at the horizontal support on a unit width (lbs/in.)*

V_u total shear on a width b (lbs)

V_x unit shear along the long side of window frame (lb/in.)

V_y unit shear along the short side of window frame, (lbs/in.)

w unit weight (psi, for panels) (lb/in for beam)

w_c weight density of concrete (lbs/ft^3)

w_s weight density of sand (lbs/ft^3)

W (1) charge weight (lbs)
(2) weight (lbs)

W_c total weight of explosive containers (lbs)

W_f weight of primary fragment (oz)

W_{co} total weight of steel core (lbs)

W_{c1}, W_{c2} total weight of plates 1 and 2, respectively (lbs)

W_s width of structure (ft)

* See note at end of symbols

WD	work done
x	yield line location in horizontal direction (in.)*
X	deflection (in.)
X_a	any deflection (in.)
X_c	lateral deflection to which a masonry wall develops no resistance (in.)
X_e	elastic deflection (in.)
X_{ep}	elasto-plastic deflection (in.)
X_f	maximum penetration into concrete of armor-piercing fragments (in.)
X'_f	maximum penetration into concrete of fragments other than armor-piercing (in.)
X_m	maximum transient deflection (in.)
X_p	plastic deflection (in.)
X_s	(1) maximum penetration into sand of armor-piercing fragments (in.) (2) static deflection
X_u	ultimate deflection (in.)
X_E	equivalent elastic deflection (in.)
X_1	(1) partial failure deflection (in.) (2) deflection at maximum ultimate resistance of masonry wall (in.)
y	yield line location in vertical direction (in.)*
y_t	distance from the top of section to centroid (in.)
Z	scaled slant distance ($ft/lb^{1/3}$)
Z_A	scaled normal distance ($ft/lb^{1/3}$)
Z_G	scaled ground distance ($ft/lb^{1/3}$)
α	(1) angle formed by the plane of stirrups, lacing, or diagonal reinforcement and the plane of the longitudinal reinforcement (deg) (2) angle of incidence of the pressure front (deg) (3) acceptance coefficient

* See note at end of symbols

β (1) coefficient for determining elastic and elasto-plastic resistances
(2) particular support rotation angle (deg)
(3) rejection coefficient

β_1 factor equal to 0.85 for concrete strengths up to 4000 psi and is reduced by 0.05 for each 1,000 psi in excess of 4,000 psi

γ coefficient for determining elastic and elasto-plastic deflections

γ_p factor for type of prestressing tendon

ϵ_m unit strain in mortar (in./in.)

θ support rotation angle (deg)

θ angular acceleration (rad/ms^2)

θ_{max} maximum support rotation angle (deg)

θ_H horizontal rotation angle (deg)*

θ_V vertical rotation angle (deg)*

λ increase in support rotation angle after partial failure (deg)

μ ductility factor

ν Poisson's ratio

Σ_o effective perimeter of reinforcing bars (in.)

ΣM summation of moments (in.-lbs)

ΣM_N sum of the ultimate unit resisting moments acting along the negative yield lines (in.-lbs)

ΣM_p sum of the ultimate unit resisting moments acting along the positive yield lines (in.-lbs)

τ_s maximum shear stress in the shaft (psi)

ϕ (1) capacity reduction factor
(2) bar diameter (in.)

Φ_r assumed shape function for concentrated loads

$\phi(x)$ assumed shape function for distributed loads free edge

ω angular velocity (rad./ms)

simple support

* See note at end of symbols

$\overline{//////}$ fixed support

$\overline{XXXXXX}$ either fixed, restrained, or simple support

* Note. This symbol was developed for two-way elements which are used as walls. When roof slabs or other horizontal elements are under consideration, this symbol will also be applicable if the element is treated as being rotated into a vertical position.

APPENDIX 6C

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