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The emphasis of this report will be on the methods available to design and analyze aboveground arches for blast loads and also on the final design results and comparisons for each case examined. It is not intended to be an in-depth study on structural dynamics and how it is applied to blast resistant design, but, will include enough background information on the procedures used herein to provide a basis for those procedures.

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> Page 4

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A COMPARISON STUDY OF ABOVEGROUND REINFORCED CONCRETE CYLINDRICAL ARCHES DESIGNED TO RESIST BLAST LOADS

BY

TIMOTHY LLOYD BOONE

A REPORT PRESENTED TO THE GRADUATE COMMITTEE
OF THE DEPARTMENT OF CIVIL ENGINEERING IN
PARTIAL FULFILLMENT FOR THE DEGREE OF
MASTER OF CIVIL ENGINEERING

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The author wishes to express his sincere thanks and gratitude to all those who assisted in the completion of this report. Special thanks goes to Dr. Fernando E. Fagundo for serving as the author's committee chairman and advisor and also for his outstanding and continuous guidance, advice, and support throughout the completion of this report. The author would also like to thank Dr. Clifford O. Hays and Dr. John M. Lybas for serving on the supervisory committee. Thanks also goes to Dr. Lybas for serving as the author's advisor prior to the initiation of this project.

The author must also thank his wife, Patty, for her understanding, patience, and encouragement. Her tolerance of the many hours and evenings at home alone with the children throughout the pursuit of this degree and especially during the completion of this report have made it all possible.

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a = height of concrete compression block, in

A_b = cross-sectional area of one steel reinforcing bar, sq in

A_c = cross-sectional area of concrete section, sq in

A_s = cross-sectional area of total steel in tension zone, sq in, (primary direction)

A_l = cross-sectional area of total steel in longitudinal direction, sq in

A_t = total cross-sectional area of steel in primary direction, both faces, sq in

b = width of concrete section, in

B = explosive and case material constant

C_d = drag coefficient

C_r = reflected pressure coefficient

d = distance from centroid of tensile reinforcement to compression face of concrete member, in

D = width of arch bay, ft

D_i = inside diameter of bomb casing, in

DL = total dead load, lbs

E_c = concrete modulus of elasticity, psi

f'_c = compressive strength of concrete, psi

f'_{dc} = dynamic compressive strength of concrete, psi

f_{dy} = dynamic yield strength of steel reinforcement, psi

f_y = yield strength of steel reinforcement, psi

F_e = equivalent force acting on member, lbs

F_t = static distribution of dynamic load on member, lb/ft

g = acceleration force due to gravity, ft/sq sec or in/sq sec

h	=	total thickness or height of concrete section, in
H	=	distance from ground level to crown of arch, ft
I_a	=	effective moment of inertia, in^4/in
I_e	=	equivalent impulse
I_d	=	approximated dynamic impulse
I_r	=	approximated reflected impulse
I_s	=	positive incident impulse
I_{so}	=	approximated incident impulse
k	=	metal constant for fragment penetration
K	=	stiffness of structural element, lb/in
L	=	length of cylindrical arch, ft
L_a	=	effective arc length of one half of the arch, in
L_w	=	length of blast wave, ft
m	=	mass per unit arch area, lb-sq in/sq sec
M_a	=	primary fragment distribution parameter
M_e	=	equivalent mass of the system, lb-cu in/sq sec
M_t	=	mass per unit arch length, lb-in/sq sec
M_u	=	ultimate moment capacity of reinforced concrete member, lb-in
P_c	=	uniform radial load on arch or ring, psi
P_{cr}	=	critical buckling pressure on an arch, psi
P_e	=	total equivalent pressure, psi
P_r	=	peak reflected pressure, psi
P_{so}	=	peak positive incident pressure, (overpressure), psi
P_u	=	ultimate compressive capacity in arch, psi
Q_o	=	peak dynamic pressure, psi
r	=	radius of arch, ft or in
R_c	=	required compressive capacity of the arch, psi

R_f = required flexural capacity of the arch, lbs

R_{fr} = distance travelled by primary fragment, ft

R_G = distance from point of detonation or explosion to nearest point on structure, ft

R_{mc} = actual compressive capacity of reinforced concrete arch, psi

R_{mf} = actual flexural capacity of reinforced concrete arch, psi

s = spacirg of steel reinforcing bars, in

S = thrust in arch or ring, lbs

S' = critical thrust or buckling load in an equivalent beam, lbs

S_{cr} = critical buckling thrust in an arch, lbs

T_a = time ellapsed from detonation to arrival of wave front at structure, sec or ms

T_{app} = approximated time for blast wave to completely pass over the structure, sec or ms

T_c = thickness of bomb casing, in

T_{nc} = natural period of vibration of a reinforced concrete arch in the compressive mode, sec or ms

T_{nf} = natural period of vibration of a reinforced concrete arch in the flexural mode, sec or ms

T_o = duration of positive phase of blast wave, sec or ms

T_{of} = fictitious duration of blast wave, sec or ms

TOTWT = total structural weight, tons

TOT$ = total estimated structural cost of structure, $

u = ductility ratio

U = velocity of blast wave, fps or ft/ms

V = dynamic reaction, lbs

V_{act} = actual shear load, lbs

V_{all} = allowable vertical shear, lbs

V_c = total volume of concrete, cy

V_o = initial velocity of primary fragments, fps

V_s = impact or final velocity of primary fragments, fps

VT_{all} = allowable diagonal shear, lbs

W = charge weight of explosive, lbs

W_c = metal casing weight of bomb, lbs

W_f = weight of heaviest fragment, oz

W_i = equivalent bare charge weight for impulse, lbs

W_p = equivalent bare charge weight for peak pressure, lbs

WT_c = total weight of concrete, tons

WT_s = total weight of steel, tons

$WT_s(p)$ = total weight of steel, (primary direction), lbs

$WT_s(l)$ = total weight of steel, (longitudinal direction), lbs

XF = maximum penetration of primary fragment, in

XF' = corrected maximum penetration of primary fragment, in

XL = load factor

XLM = load-mass factor

X_m = plastic deflection, in

XM = mass factor

X_y = elastic deflection, in

Z_g = scaled ground distance of explosion from structure.

Z_i = scaled ground distance for impulse

Z_p = scaled ground distance for peak pressure

α = angle of incidence or slope at base of arch, deg

β = interior angle of arch from crown to base, radians

p = ratio of steel area in tensile zone to total cross-sectional area of concrete section

p_t = total steel ratio, both faces

p_l = steel ratio of longitudinal reinforcement

p_v = steel ratio of diagonal reinforcement

σ = compressive stress capacity of reinforced concrete, psi

μ_f = curvature correction factor, flexural mode, for hinged arches

μ = curvature correction for natural period of flexural vibration on a hinged arch

γ_c = weight of concrete, lb/cu ft

γ_s = weight of steel, lb/cu in

$\$_c$ = estimated cost of concrete in the structure, \$

$\$_s$ = estimated cost of steel in the structure, \$

CHAPTER ONE
INTRODUCTION

1.1 Background

The world today contains many threats and dangers; some obvious and some not so obvious. The United States military services are responsible for protecting a large part of the world from these dangers. One of the primary areas of concern included in this responsibility is the design and construction of protective facilities on military installations; especially on those perceived to be in hostile areas.

The major threats for consideration generally fall into two categories; nuclear weapons and non-nuclear or conventional weapons. Based on these threats, the protective facilities required normally fall into two categories, as well; underground and aboveground. As a rule, underground facilities are more costly to construct while being able to withstand larger blast loads than are aboveground facilities. Hence, underground facilities are usually most feasible and effective against nuclear attacks while aboveground facilities are most feasible and effective for conventional attacks.

1.2 Statement of Purpose

Given a known non-nuclear threat and also pre-established minimum dimensional requirements for a facility that is vulnerable to attack, the structural engineer may be faced with a number of options when selecting a final design for this protective facility. Assuming that, due to operational requirements, the options have been narrowed down to an aboveground cylindrical arch constructed of reinforced concrete, (Figure 1-1), there are still a variety of configurations that might be considered before making the final selection. The purpose of this report is to evaluate a number of designs for these types of facilities. The results will be compared in an attempt to develop correlations between different geometric configurations of such arches and between the design parameters relevant to weapons and blast effects.

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The design method to be used is that curently in use by the United States military services as developed at the Air Weapons Laboratory, Kirtland Air Force Base, New Mexico, (Reference 1). This design procedure is quite simplified, as

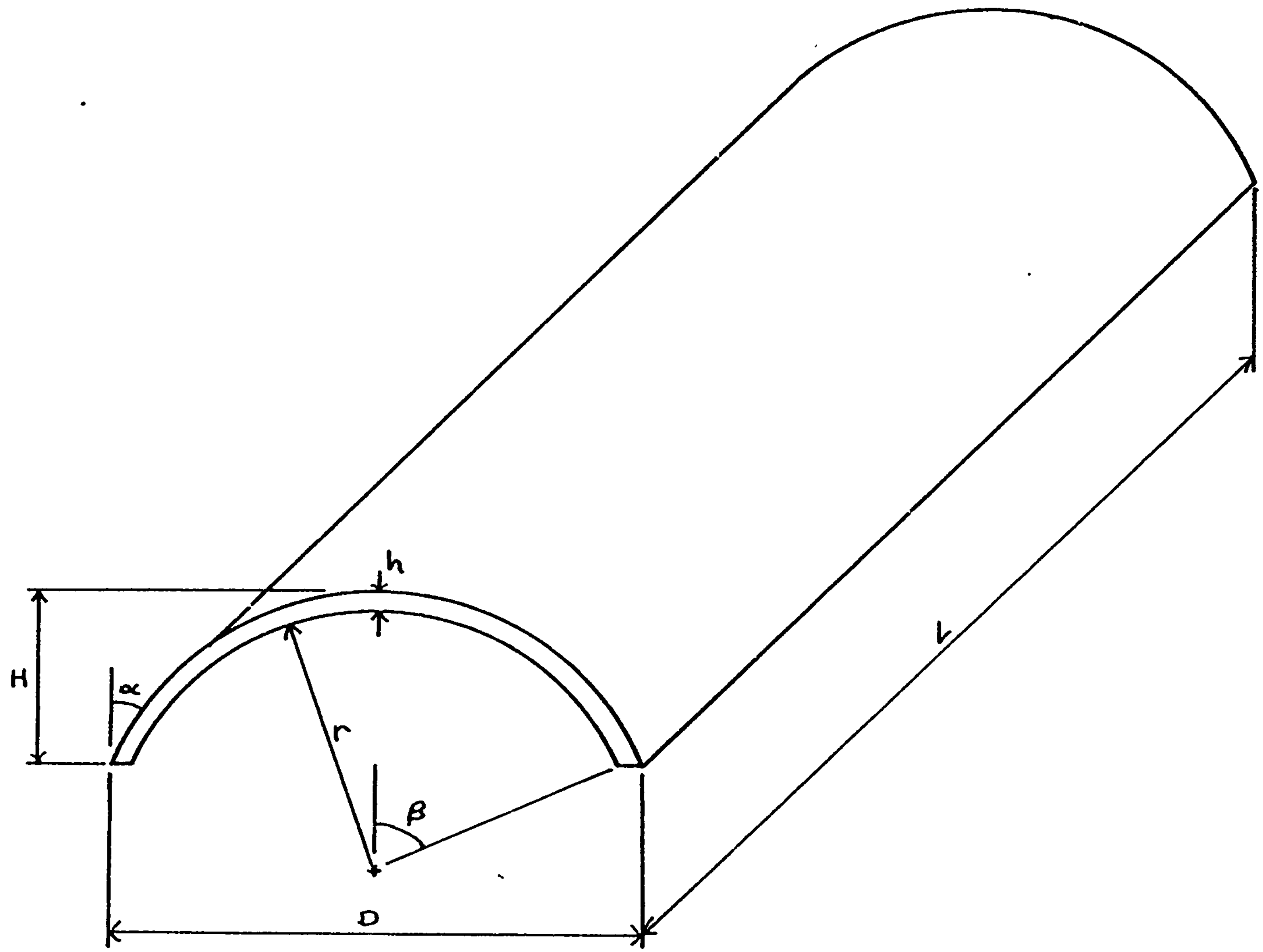


Figure 1-1. Aboveground Reinforced Concrete Cylindrical Arch

will be shown, and would be best suited for an initial or trial design leading to the more thorough and precise methods available to perform the final designs. Only the aboveground arch structure will be studied. The blast effects on foundations, openings, end walls, and mechanical and electrical systems will not be considered. The aboveground portion of the structure is most affected by the various blast design loads and changes in configuration and, as such, will have the greatest impact on the data used for the final comparisons. Airblast and fragmentation will be the only loads resulting from explosions considered in this report, since these are the primary components acting on an aboveground structure. Load components such as ground shock and cratering will be neglected.

CHAPTER TWO
AIRBLAST AND AIRBLAST EFFECTS

2.1. Components of an Airblast

Airblast and shockwave phenomena resulting from an explosion impart a total load on a structure comprised of primarily three components. The first is the peak incident pressure, (P_{so}), or overpressure, that results from the instantaneous pressure rise above ambient pressure upon detonation of the explosive. This is the actual blast wave and is at it's peak immediately after the explosion, expanding radially outward and decaying in intensity from the center of detonation. The blast wave travels at a diminishing velocity, (U), and is always in excess of the speed of sound. The overpressure, as well as the other two components to be identified, are generally depicted by a pressure vs. time load function as shown in Figure 2-1.

The second phenomena producing loads on the structure arrises when the wavefront, (Figure 2-3), makes contact with the structure. This results in a reflected wave as the initial wave 'bounces off' the structure and is overlapped and magnified by itself. As implied, the reflected pressure, (P_r), is actually greater and produces a larger load on the structure than does the peak overpressure, (Figure 2-2).

The magnitude of the reflected wave is largely dependent on the angle at which it impinges on the structure. In the case of a cylindrical arch, the reflected pressure becomes a function of the angle of incidence (α), or slope of

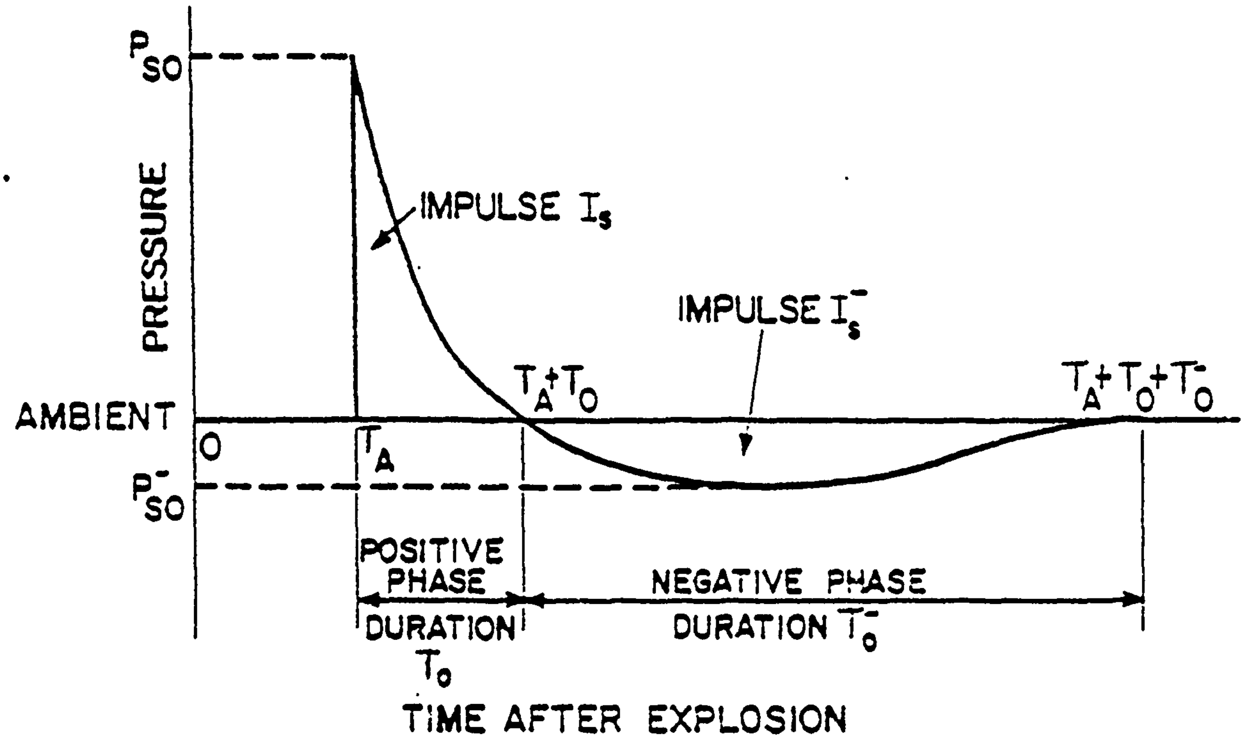


Figure 2-1. Free Field Pressure-Time Variation (Ref. 1)

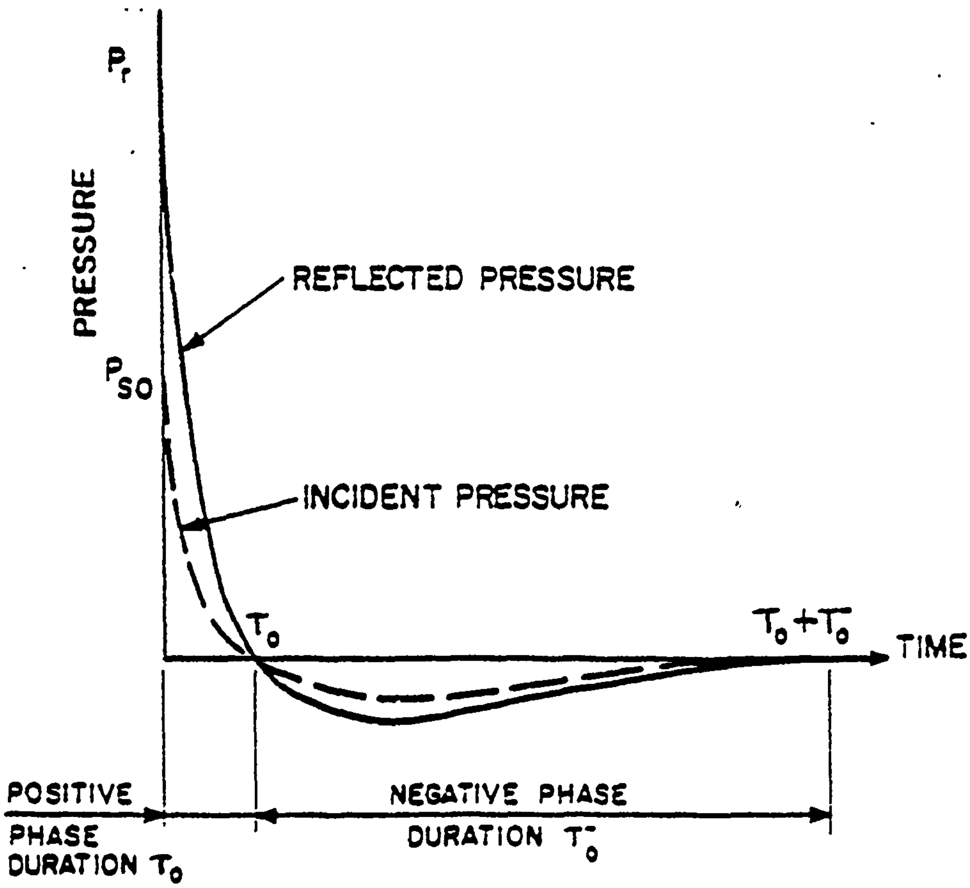


Figure 2-2. Pressure-Time Variation for Free Air Burst (Ref. 1)

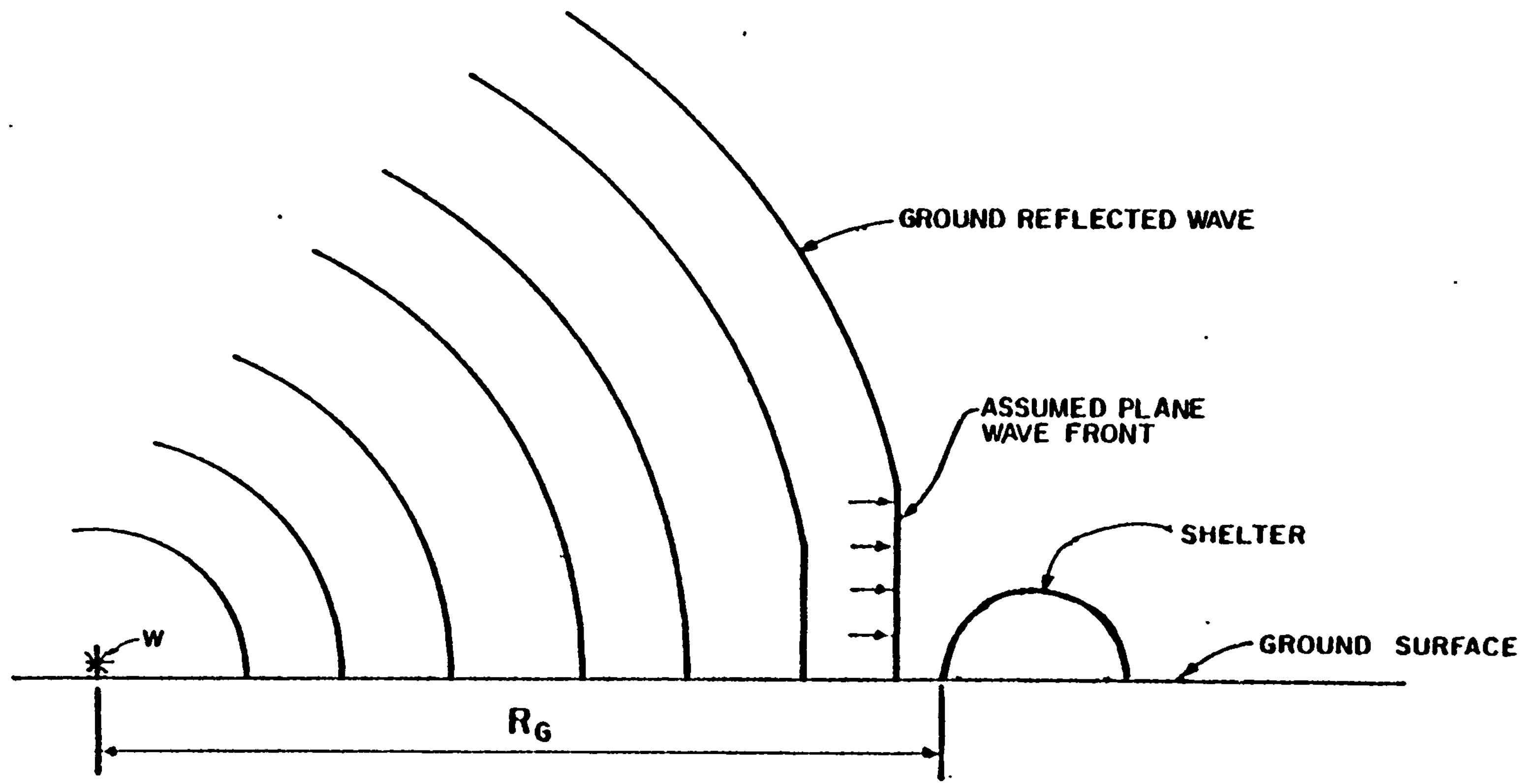


Figure 2-3. Surface Burst Blast Environment (Ref. 1)

the arch wall at its' base. It follows that the largest reflected pressure generally occurs on a surface perpendicular to the ground or to the direction that the blast wave is travelling, ($\alpha = 0$), and decreases in magnitude as the angle of incidence increases, (up to $\alpha = 30°$, Figure 2-4).

The third and final component of the total blast load is the dynamic pressure, (Q_o). A blast wave most closely resembles and is accompanied by an extremely strong wind. As is the case in wind load design, this results in an inward pressure on the windward side of a structure and an outward pressure on the leeward side of a structure. The dynamic pressure is dependent on the peak incident pressure, (Figure 2-5), and is used in conjunction with a drag coefficient, (C_d), as in wind design, (total dynamic pressure $= Q_o C_d$). Obviously, all three of these airblast components will be acting on an aboveground structure, and it must be designed accordingly.

2.2 Effects of Blast Loads on Structures

The resulting magnitudes and distributions of loads on a structure due to the combined effects of these three load components are largely dependent on four factors; they are: (1) the type and size of weapon or explosive that is used; (2) the location of the weapon or explosive relative to the structure at detonation; (3) the amount of reflection and reinforecment of the blast wave as it reacts with other

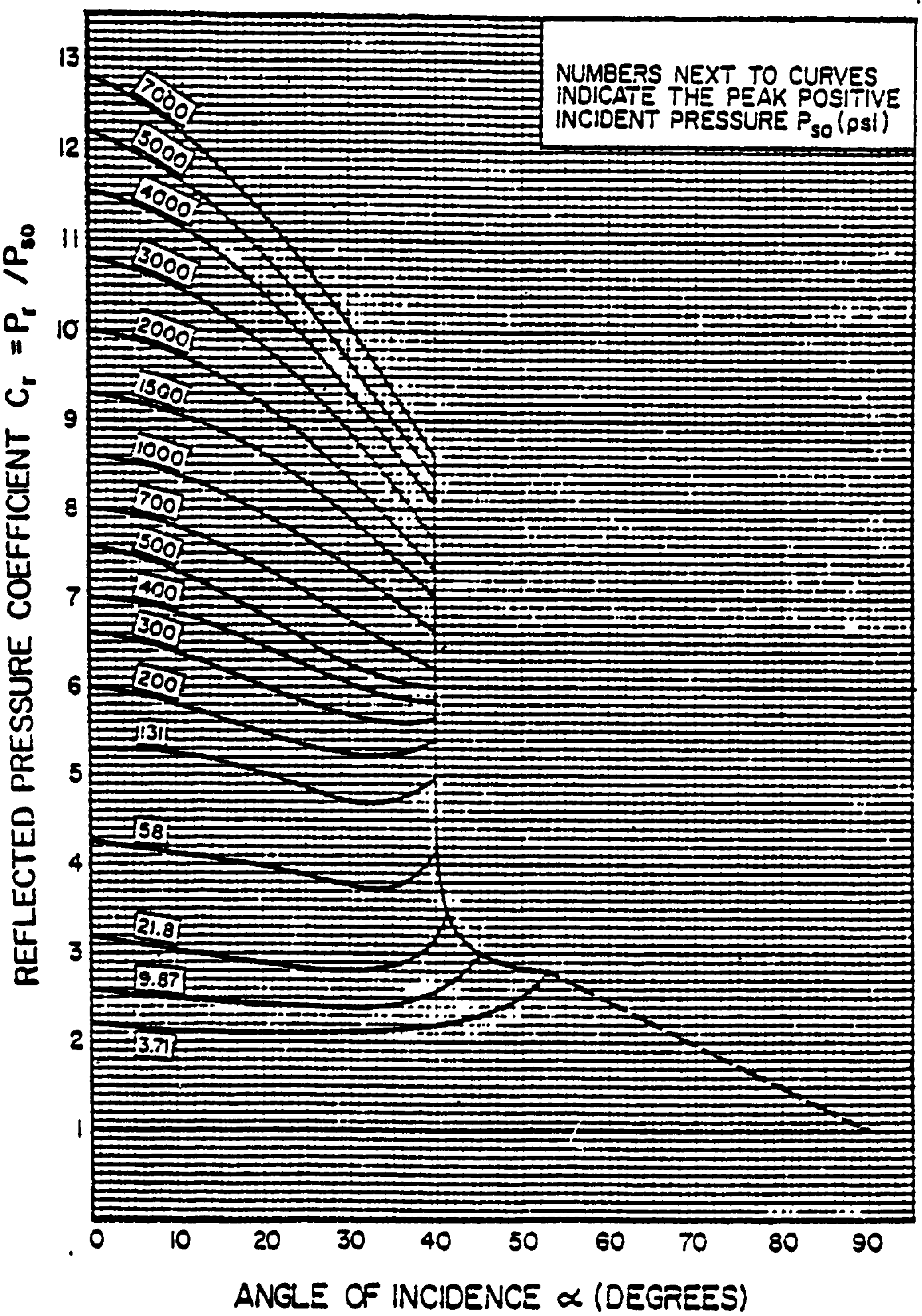


Figure 2-4. Reflected Pressure Coefficient vs. Angle of Incidence (Ref. 1)

surfaces, also a function of the weapon's location at detonation; and (4) the geometric configuration of the structure. The primary purpose of this report is to study the effects of varying this fourth and final factor.

Structures subjected to these blast loads, or any dynamic loads, have a much different response than those subjected to static loads. Whereas, a static load produces a constant deflection in the structural element, a dynamic load produces deflections that vary over time, (vibrations). If this deflection vs. time function can be described by one coordinate, the element is said to vibrate in only one mode and it is a one degree-of-freedom system. It follows, that if two coordinates or variables are required to describe the motion, it is a two degree-of-freedom system, and so on. Vibrations may actually occur in an infinite number of modes resulting in infinite numbers of deflected shapes, however, only a few of the lower modes normally have responses that are of any significance for practical purposes.

In the case of an aboveground arch, only the first two modes are usually considered to be significant. These two modes are defined as the compressive or 'breathing' mode, which is symmetrical, and the flexural or bending mode, which is asymmetrical, (Figure 2-6). The peak overpressure is assumed to act uniformly across the entire arch creating a radially uniform compressive stress in the arch. The reflected pressure, being the largest at the windward base of the arch, and the dynamic pressure, being positive on the

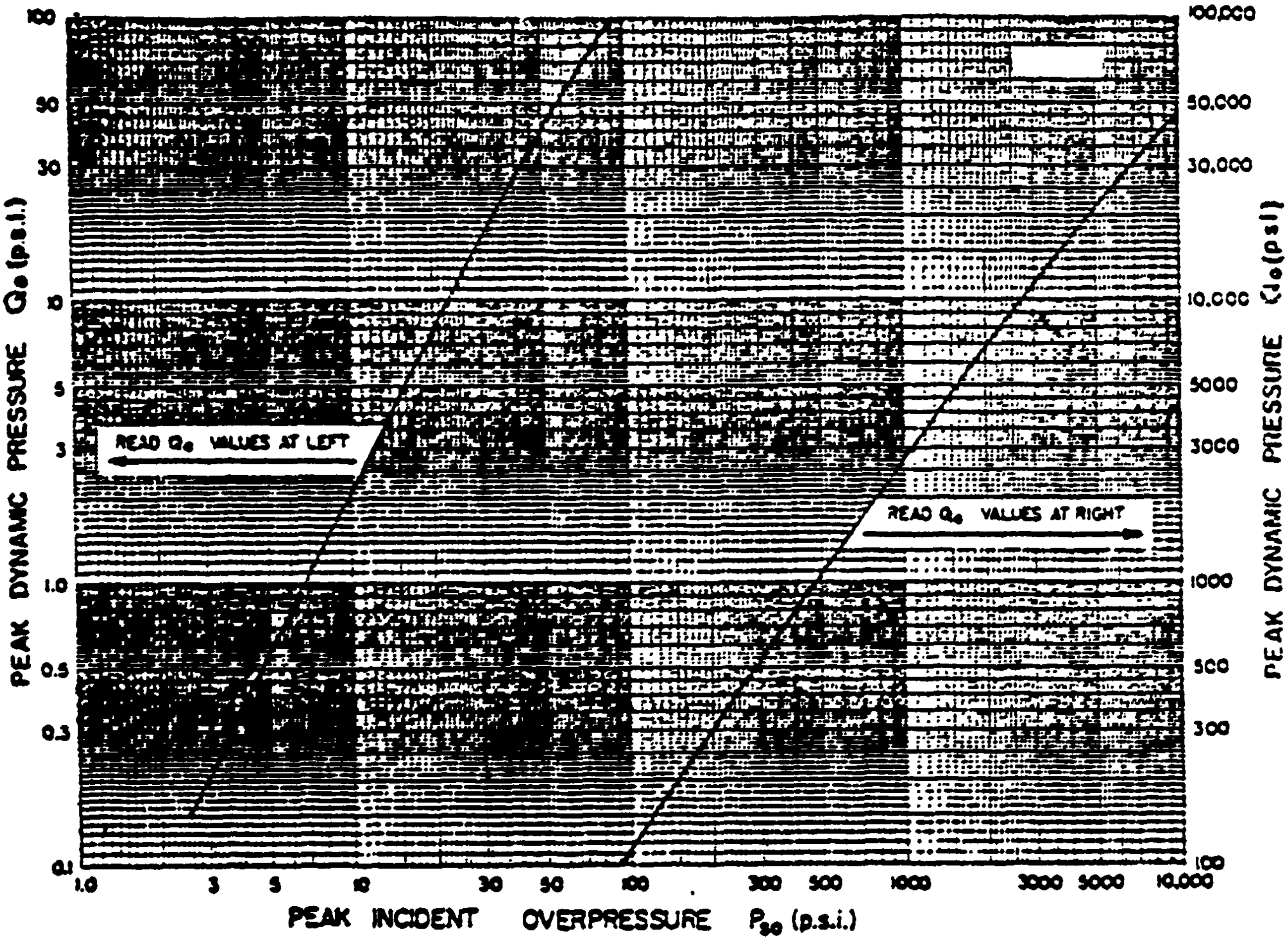


Figure 2-5. Peak Incident Pressure vs. Peak Dynamic Pressure (Ref. 1)

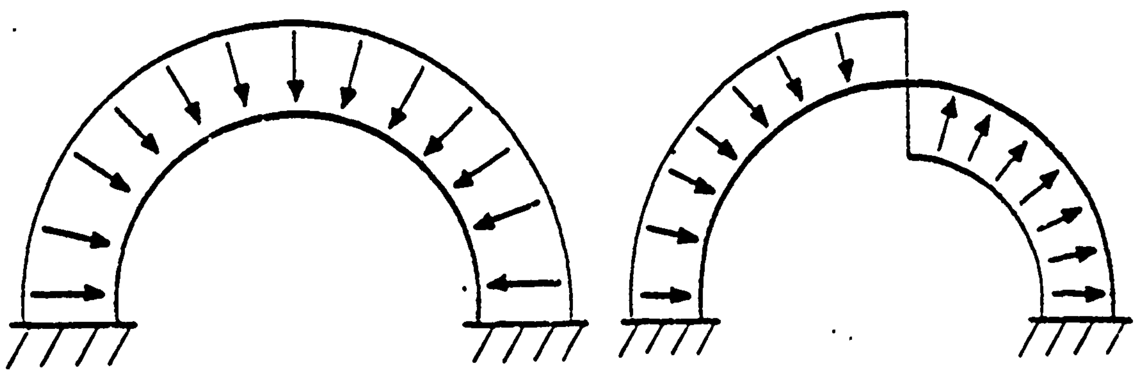


Figure 2-6. Reaction of Arch to Blast Loads (Ref. 1)

windward side and negative on the leeward side, are assumed to work in conjunction with one another to produce an asymmetrical flexural vibration in the arch, resulting in bending stresses. Again, this implies a dynamic system with two degrees-of-freedom or two primary modes of vibration.

Another important parameter when considering dynamic systems and vibrations is the frequency or period of the respective vibrations. A structural element placed in some initial deflection and then released, without being affected by any external forces, exhibits a motion known as free vibration. If this motion is sinusoidal with respect to time it is said to be 'harmonic'. The natural period of this vibration is the time it takes for the element to return to its' initial position or to go through one complete cycle. It follows that the natural frequency is the number of these cycles completed in one second. Each structural element and, hence, each structure has its' own inherent natural period or frequency of vibration. Each mode of vibration also has a natural period or frequency. Natural periods of vibration are largely dependent on the structure's <u>mass</u> and <u>stiffness</u> properties as will be shown later.

If the load applied to an element or structure has the same or nearly the same frequency or duration as the natural frequency of the structure itself, a phenomenon known as 'resonance' results. Obviously, this is an undesireable situation and must be avoided to prevent excessive motions from occuring within the structure. A specific step is included in most dynamic design and analysis methods to insure that 'resonance' does not occur, as will be shown in Section 3.4.4.

CHAPTER THREE
METHODS USED

3.1 Introduction

There are three general categories of airblasts, as defined by their relative position to the protective structure, that are of major concern when designing for blast loads. They are as follows:

(1). Free Air-Burst: The explosion takes place above or adjacent to the struture in such a manner that the wavefront impinges directly on the structure with no reflection or amplification occurring in between.

(2). Air-Burst: The explosion occurs above the ground surface, but, at such a distance from the structure that the blast wave is reflected off the ground prior to arriving at the structure itself. This sets up a reflected wave front that is used for the respective load calculations.

(3). Surface Burst: The explosion occurs at or so close to the ground surface that the blast wave is immediately reflected off the ground surface setting up a reflected/reinforced wave front.

The surface burst will be the only category evaluated in this report since it normally creates the most critical pressure loads. The reflected wave front produced by a surface burst is assumed to be essentially hemispherical in shape and is treated as a vertical plane at the point of contact with the structure, (Figure 2-3).

3.2 Dynamic Design Process - General

As is the case in most structural design, the process used here is a trial and error method and iterative in nature. An initial assumption normally made is that the blastwave moves across the structure in a direction perpendicular to the longitudinal axis of the cylinder, (Figure 2-3), producing the worst load case for the ensuing designs. The engineer's goal is to design a structure having an adequate ultimate strength to survive a transient dynamic load with properties as described in Section 2.1. A brief overview of the iterative process employed in the design method used in this report follows:

(1). Having defined the threat, estimate appropriate pressure vs. time load functions.

(2). Select a trial section, based upon some static design principle or criteria.

(3). Determine the actual compressive and flexural capacities of the section using conventional static design and analysis methods.

(4). Determine the two natural periods of vibration (compressive and flexural), of the section.

(5). Using these natural periods of vibration, determine the ultimate dynamic strength resistance that is required in the two modes, or the required capacities, based on the estimated load function.

(6). Depending on the outcome, revise the section as necessary to develop the ultimate strength.

(7). Having revised the section, reiterate steps (3) through (6) until a final satisfactory design has been achieved.

3.3 Load Calculation Method

The parameters required for establishing the actual loads and resulting stresses on a structure can normally be obtained from a graph similar to Figure 3-1. This particular plot uses a scaled ground distance, (Z_g), based on the actual ground distance, (R_g), and the explosive charge weight, (W).

Another important load parameter that must be established is the rate of decay of each blast pressure component. This is a function of the peak incident pressure and the magnitude of the detonation. In the simplified method being used, the actual incident pressure is approximated by an equivalent triangular pressure time pulse, (Figure 3-2). The actual time duration of the positive portion of the blast wave, (T_o from Figure 2-1), is replaced by a fictitious duration, (T_{of}). The impulse, (I_{so}), of this fictitious wave is defined as the integrated area under the positive portion of the triangular curve. If the curve has been approximated by the triangular function shown in Figure

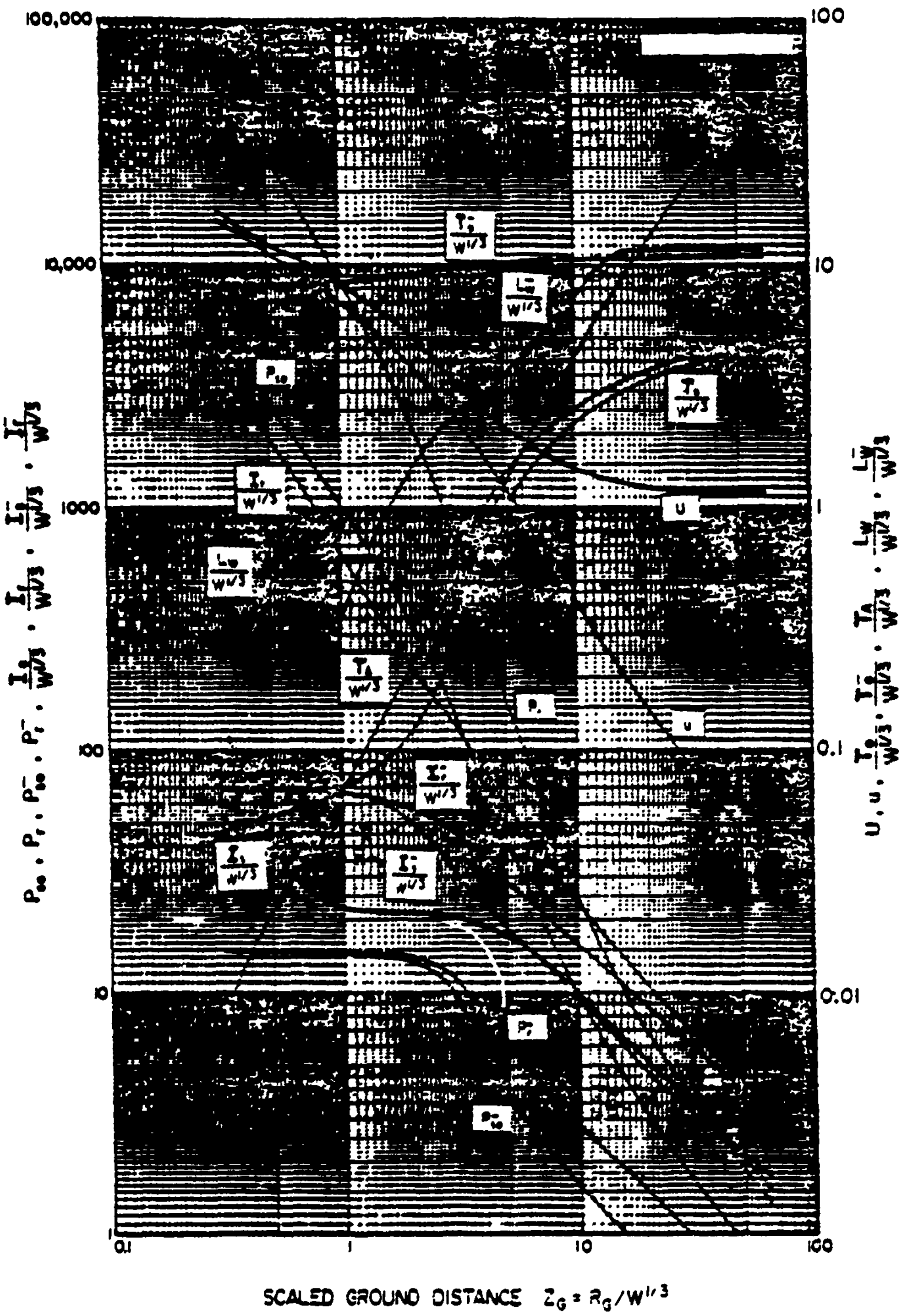


Figure 3-1. Shock Wave Parameters for Hemispherical TNT Surface Explosion at Sea Level (Ref. 1)

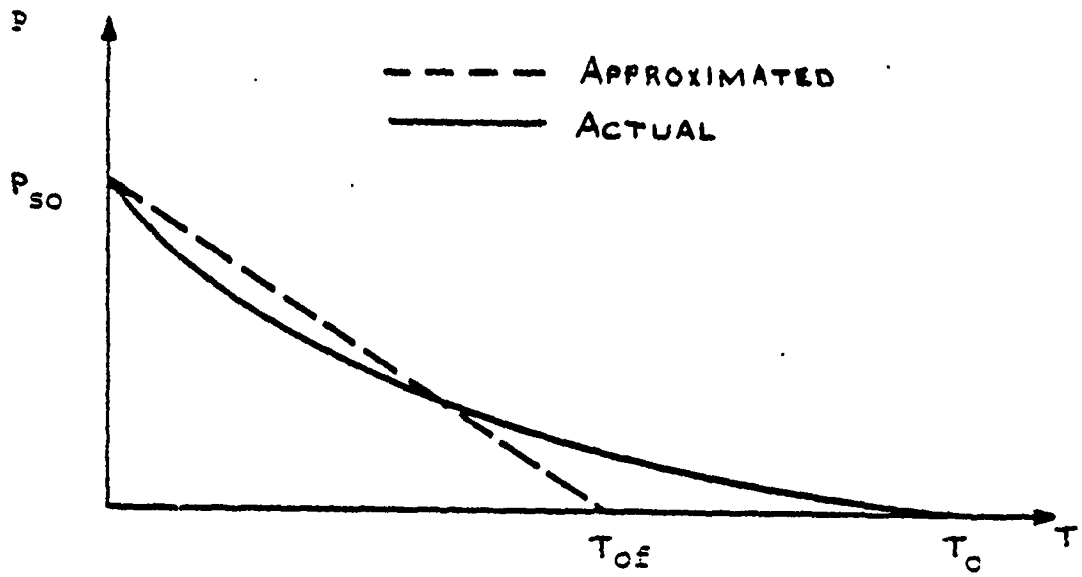


Figure 3-2. Single-Triangle Approximation for Overpressure (Ref. 1)

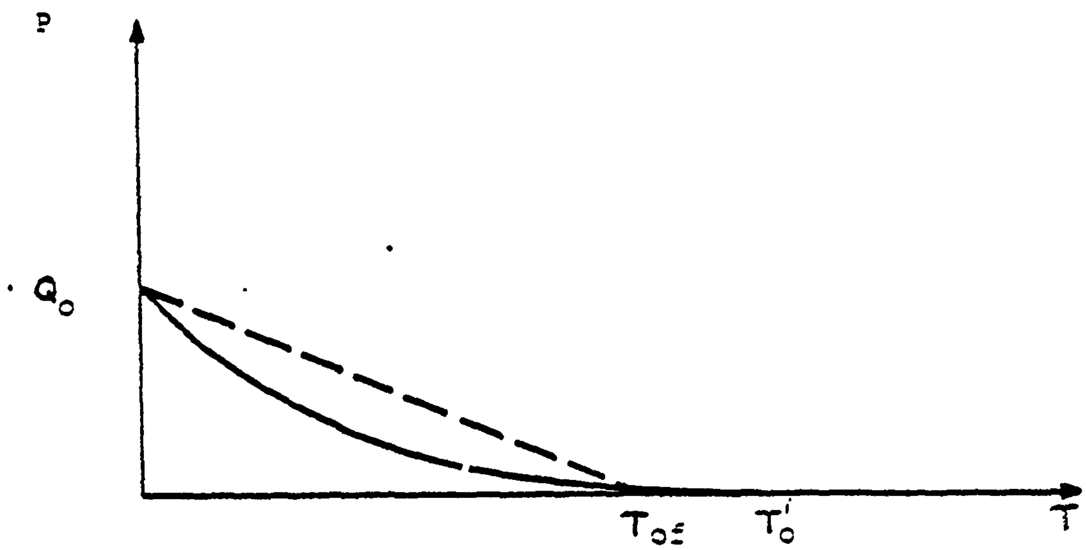


Figure 3-3. Single-Triangle Approximation for Dynamic Pressure (Ref. 1)

3-2, the expression for I_w becomes:

$$I_w = \tfrac{1}{2}(P_{so})(T_{of}) \qquad (3.1)$$

and rearranging:

$$T_{of} = 2I_w/P_{so} \qquad (3.2)$$

A simplified loading function, such as this one, is then substituted for each of the three pressure components presented in Section 2.1, (P_{so}, P_r, and Q_o), and are used for the ensuing designs. Their respective theoretical curves are replaced with triangular functions, as described in the previous paragraph, having initially peaked values which decay to zero in time T_{of}, (Figures 3-2, 3-3, and 3-4).

Another important parameter used in this design method is the time that it takes for the blast wave to completely pass over and clear itself from the arch, (Figure 3-5). Defined as the approximate transit time, (T_{app}), it follows that this equals the time required to travel over the span width of the arch plus the time it takes to travel one of its' own wavelengths, (L_w), or:

$$T_{app} = T_{of} + D/U\text{, sec or ms} \qquad (3.3)$$

Given the simplified load vs. time functions just established for each pressure component and their corresponding values, the next step would be to evaluate how

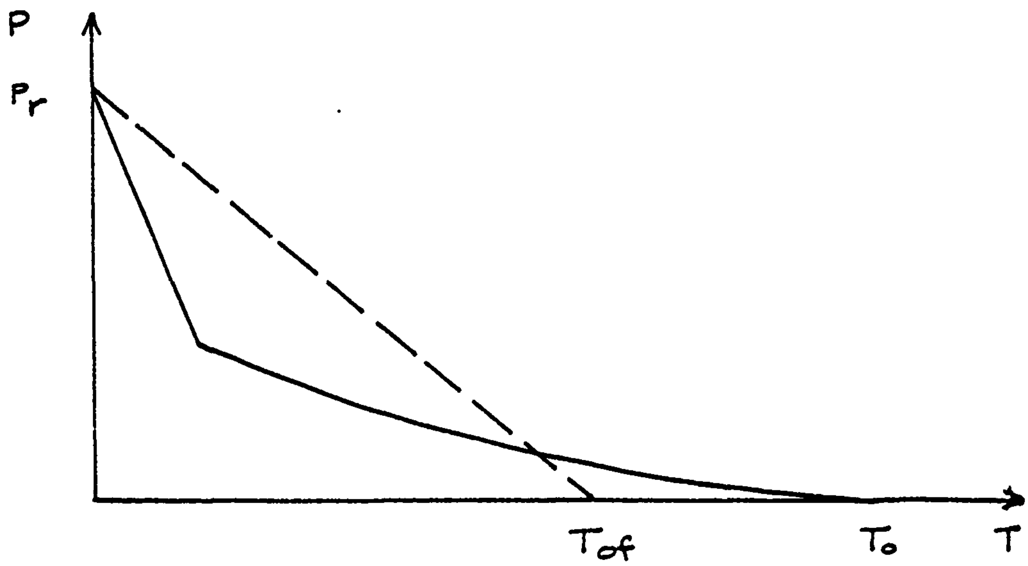


Figure 3-4. Single-Triangle Approximation for Reflected Pressure (Ref. 1)

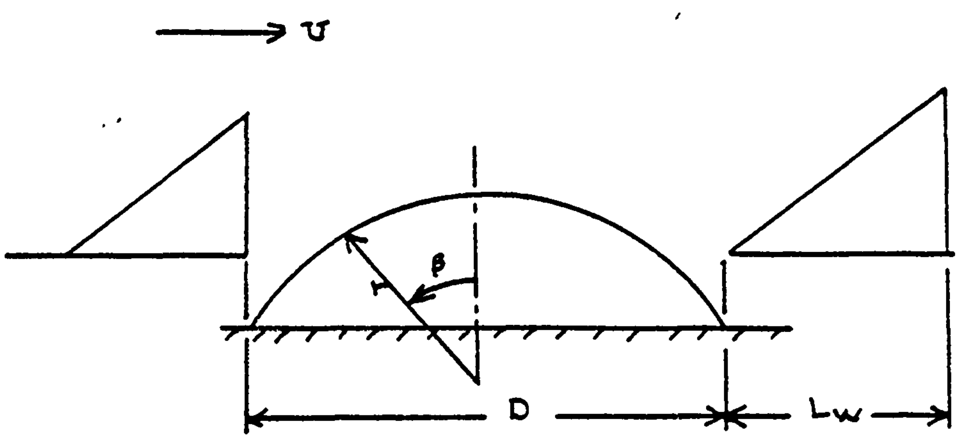


Figure 3-5. Blast Wave Passing Over an Arch (Ref. 1)

they act on the structure. This has also been greatly simplified in the referenced procedure.

The maximum reflected pressure, being dependent on the slope of the arch, is assumed to act uniformly inward on the windward side of the arch. This pressure includes the peak incident and peak dynamic pressures at that point. The peak incident pressure, which was assumed to be a uniform compressive force engulfing the entire facility, would also be included as a uniform inward pressure on the leeward side of the arch. In addition, the dynamic pressure, acting as a wind force, is assumea to contribute a uniform outward pressure on the leeward side of the arch since it's corresponding drag coefficient would be negative over that region, (Figure 3-6). These uniform pressures, as described, are then assummed to be symmetric or assymetric depending on the governing criteria established during the design process.

The approximated load vs. time functions in conjunction with this simplified structure loading are now used to establish one equivalent load or combined pressure, (P_{e}), comprised of all three pressure components. Again, hv defining the impulse as before and by using the previous simplifications, an overall equivalent impulse, (I_{e}), can be calculated. Assuming inward as being positive and outward as being negative leads to:

$$i_r = (½)(P_r \times T_{or}), \text{ (Figure 3-4)} \qquad (3.4)$$

$$I_{so} = (½)(P_{so} \times T_{of}), \text{ (Figure 3-2)} \qquad (3.5)$$

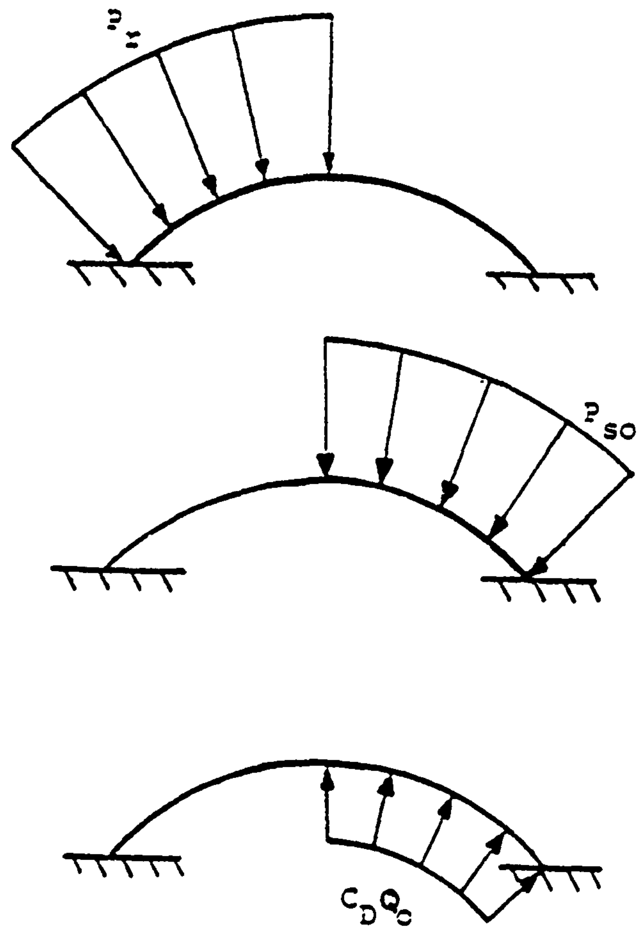


Figure 3-6. Components of Pressure Loading on an Arch (Ref. 1)

$$I_q = -(½)(Q_o C_d \times T_{of}), \text{ (Figure 3-3)} \qquad (3.6)$$

where I_r, I_{so}, and I_q equal the reflected, incident, and dynamic impulses respectively.

Since each of these is assumed to be acting over only one half of the arch, they would again be divided in half and combining Equations 3.4, 3.5, amd 3.6, would give:

$$I_m = (½)(I_r + I_{so} + I_q)$$

$$= (¼)(P_r + P_{so} - Q_o C_d)(T_{of}) \quad (3.7)$$

The actual loads do not act on all points of the structure at once which makes evaluation difficult under those conditions. This is why the actual loads are simplified and converted into one equivalent load. This conversion also allows the use of equations and charts which have been developed specifically for such simplified systems. The equivalent load is then assumed to have the duration, T_{app}, previously introduced, (Figure 3-7). The equivalent impulse based on this assumption, along with the principles established in developing Equation 3.1, would be:

$$I_e = (½)(P_e \times T_{app}), \text{ (Figure 3-7)} \qquad (3.8)$$

Combining Equations 3.7 and 3.8 and solving for P_e, leads to the following expression:

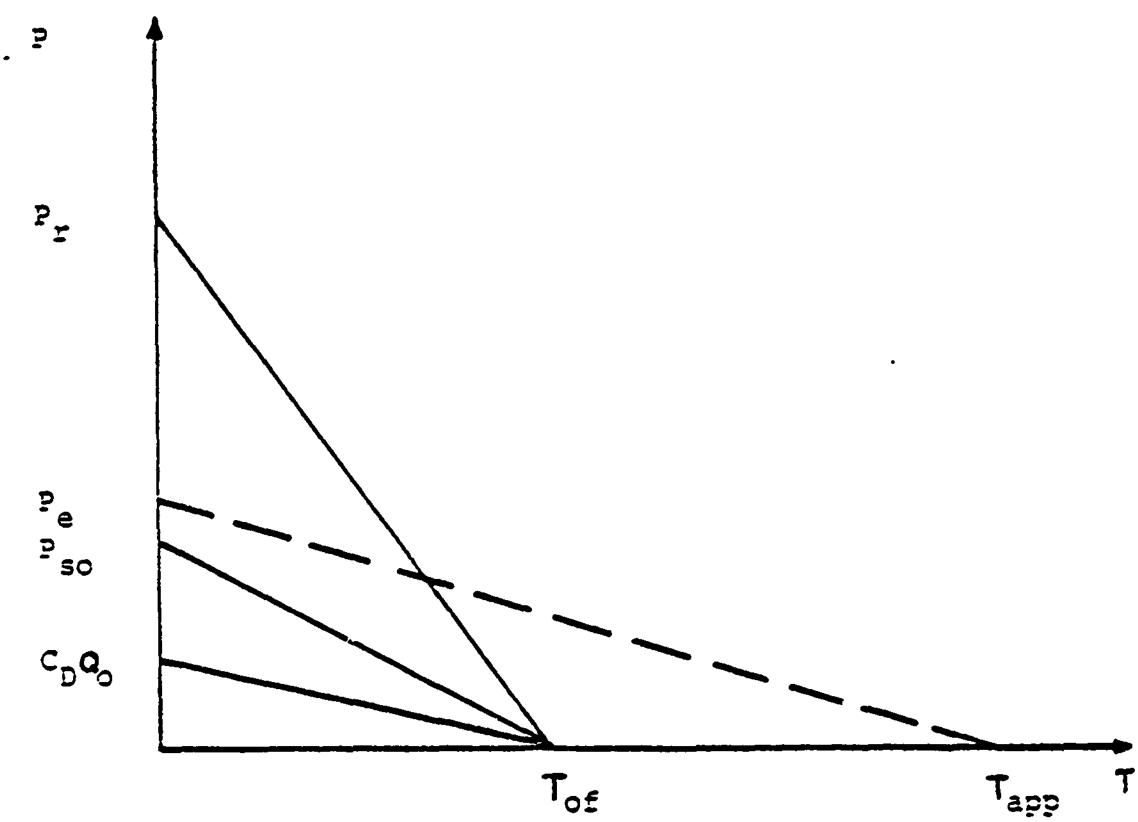


Figure 3-7. Total Approximated Loading Function for an Arch (Ref. 1)

$$P_{a} = (\tfrac{1}{2})(P_{r} + P_{so} - Q_{o}C_{d})(T_{c}f/T_{app}), \text{ psi} \quad (3.9)$$

3.4 Design and Analysis Method

The intent of this section is to review the design method selected for use in this report, step-by-step, primarily as presented in Reference 1. As implied in Section 1.2, it is not intended to derive each and every equation, but, only to present enough background on each step to clarify the reason for its' use.

3.4.1 Trial Section

Assuming that the threat has been adequately defined and the appropriate loading conditions have been established, as outlined in Section 3.3, a trial section may be selected and the design process begun. In the method used, the selection of a trial section includes the choice of the thickness of the concrete, (h), and the amount of steel reinforcement or steel ratio, (p), to be used in the reinforced concrete arch. It is common to select the trial section based on some static loading analysis method which will be explained just prior to beginning the actual designs.

3.4.2 Actual Compressive Capacity

Once a trial section has been chosen, it's capacity in both the compressive and flexural modes may be determined using well-established engineering principles in the design and analysis of reinforced concrete structures. Many of these methods are as outlined in the ACI Code, (Reference 6).

The compressive capacity is defined as the arch's ability to adequately resist the axial thrust that is produced by the uniform inward pressures acting on the structure. The basis for the static analysis in the compressive mode is the equation used for determining hoop stresses in circular rings. This equation states that the allowable or ultimate compressive capacity in a circular ring, (P_u), is the useable compressive strength capacity of the material being used in the ring, (σ), over the ring's cross-sectional area, (A_c), or:

$$P_u = \sigma \times A_c, \text{ lbs} \tag{3.10}$$

For reinforced concrete the useable compressive strength is comprised of both that contributed by the concrete and that contributed by the total reinforcing steel in the concrete. From reinforced concrete design, the concrete contribution is based on it's compression block, ($0.85f'_c$), and the reinforcement's contribution equals (p_t)(f_y), where:

f'_c = compressive strength of concrete, psi

f_y = yield strength of steel reinforcement, psi

p_t = total steel ratio in primary direction, both faces

so:

$$\sigma = (.85)(f'_c) + (p_t)(f_y), \text{ psi} \tag{3.11}$$

Throughout this report a 1" wide strip of concrete is assumed for evaluation purposes, (b = 1"), unless otherwise noted, hence, substituting Equation 3.11 into Equation 3.6 for this circular ring, gives:

$$P_u = (0.85f'_c + (p_b)(f_y))A_c\text{, lbs/in of width} \quad (3.12)$$

where:

$$A_c = (h)(b) = h\text{, in}^2$$

In order to convert this to a uniform radial pressure, Equation 3.12 is divided by the radius of the ring or arch, (r), resulting in the actual compressive capacity or resistance provided by the given structure, (R_{mc}), which is:

$$R_{mc} = (0.85f'_c + (p_b)(f_y))h/r\text{, psi} \quad (3.13)$$

3.4.3 <u>Actual Flexural Capacity</u>

In determining the flexural or bending capacity of an arch, (with hinged supports), the member is normally treated as a simply supported reinforced concrete beam and the results obtained from that evaluation are then corrected for the curvature of the arch. The length of this equivalent beam is assumed to be one half the developed arc length of the arch, (L_a), or:

$$L_m = r \times \beta, \text{ in} \qquad (3.14)$$

where:

β = interior angle of arch from crown to support in radians, (Figure 1-1).

The equivalent beam will have the same cross-sectional properties of the arch member where h is the thickness, b is the width, (1"), and p_t is the total steel ratio. The correction for curvature factor used for hinged arches, (μ_r), is as follows:

$$\mu_r = 1 - (1/n^2) \qquad (3.15)$$

where:

$$n = \pi/\beta$$

From basic flexural design for reinforced concrete, the ultimate moment, (M_u), that a rectangular reinforced concrete beam can resist is based on two criteria. At this ultimate moment the steel reinforcement in tension reaches it's yield strength and the compression block, ($0.85f'_c$), in the concrete has a certain height, (a), where:

$$a = (A_s f_y)/(0.85f'_c b), \text{ in} \qquad (3.16)$$

and:

$$A_{s} = pbd, \text{ in}^2 \qquad (3.17)$$

so:

$$M_u = (A_s f_y)(d - \tfrac{1}{2}a), \text{ in-lb} \qquad (3.18)$$

where:

d = distance from compression face of beam to centroid of tensile reinforcement, in

and:

d − ½a = the moment arm, in

Equating the internal and external work on this beam, loaded to M_u, will yield an expression for the flexural resistance of the beam, $(8M_u/L_m)$. Correcting for the curvature, (Eq. 3.15), gives the following expression for the actual flexural capacity of the arch, (R_{mf}):

$$R_{mf} = (8M_u/L_m)\mu_f, \text{ lbs} \qquad (3.19)$$

It must now be determined if the capacities in each of the two modes will be adequate to resist the applied loads and their resulting stresses in each mode. There are two major considerations in determining the required resistances. The first is the actual dynamic load applied as converted to the effective load, (Eq. 3.9). The second is the natural frequency or period of vibration of the arch as designed, and as discussed in Section 2.2.

3.4.4 Required Compressive Capacity

In the compression mode, the actual dynamic compressive thrust applied and the natural period of vibration are used to determine the required compressive resistance. A common expression used to determine the natural period of vibration, (T_{nc}), in the uniform compressive mode is:

$$T_{nc} = 2\pi r(m/E_c A_c)^{1/2} \text{ , sec or ms} \quad (3.20)$$

In this expression, 2π establishes one cycle, (2π radians = 360 degrees), and r establishes a radial mode of vibration. The modulus of elasticity of concrete, (E_c), and the concrete cross-sectional area, (A_c), establish the stiffness of the arch. The arch mass contribution, m, is the mass per unit of arch area and is determined as follows:

$$m = (bh)\tau_c/g, \text{ lb-in}^2/\text{sec}^2 \quad (3.21)$$

where:

τ_c = weight per cubic foot of concrete, lb/cf

g = acceleration due to gravity

The natural period of the compressive vibration is now compared to the duration of the load, ($T_c = T_{app}$), using the ratio T_c/T_nc. Obviously, if this ratio is close to 1, meaning the two time periods are nearly equal, 'resonance' will occur and structural changes will be necessary.

It is now possible to use a response chart to obtain the remaining information required in this part of the analysis. A response chart is a chart showing many important response variables which characterize a dynamic system and loads. It provides a very rapid means for determining those variables. This is one of the benefits of having converted to a simplified system. Response charts have been compiled for many different systems and loading functions. Figure 3-8 is one that may be used for triangular load functions such as the one approximated in Section 3.3 and used in this application.

A variable that requires introduction at this point is the ductility ratio, (u), which is expressed as X_m/X_y on the response chart. X_m is the deflection at a point when the moment applied equals the ultimate moment, (M_u), resulting in the development of and full rotation of a plastic hinge. X_y is the deflection at this same point under fully elastic conditions. The subject of ductility ratios will be addressed more in the flexural portion of the analysis. For now, full elastic design, (u = 1), will be assumed in the compression mode. This is because the entire arch section is considered to be under a uniform stress intensity equal to P/A. When the material being used is reinforced concrete, these stresses approach the ultimate capacity of the concrete and the possibility of a sudden failure over large areas of the arch is quite great.

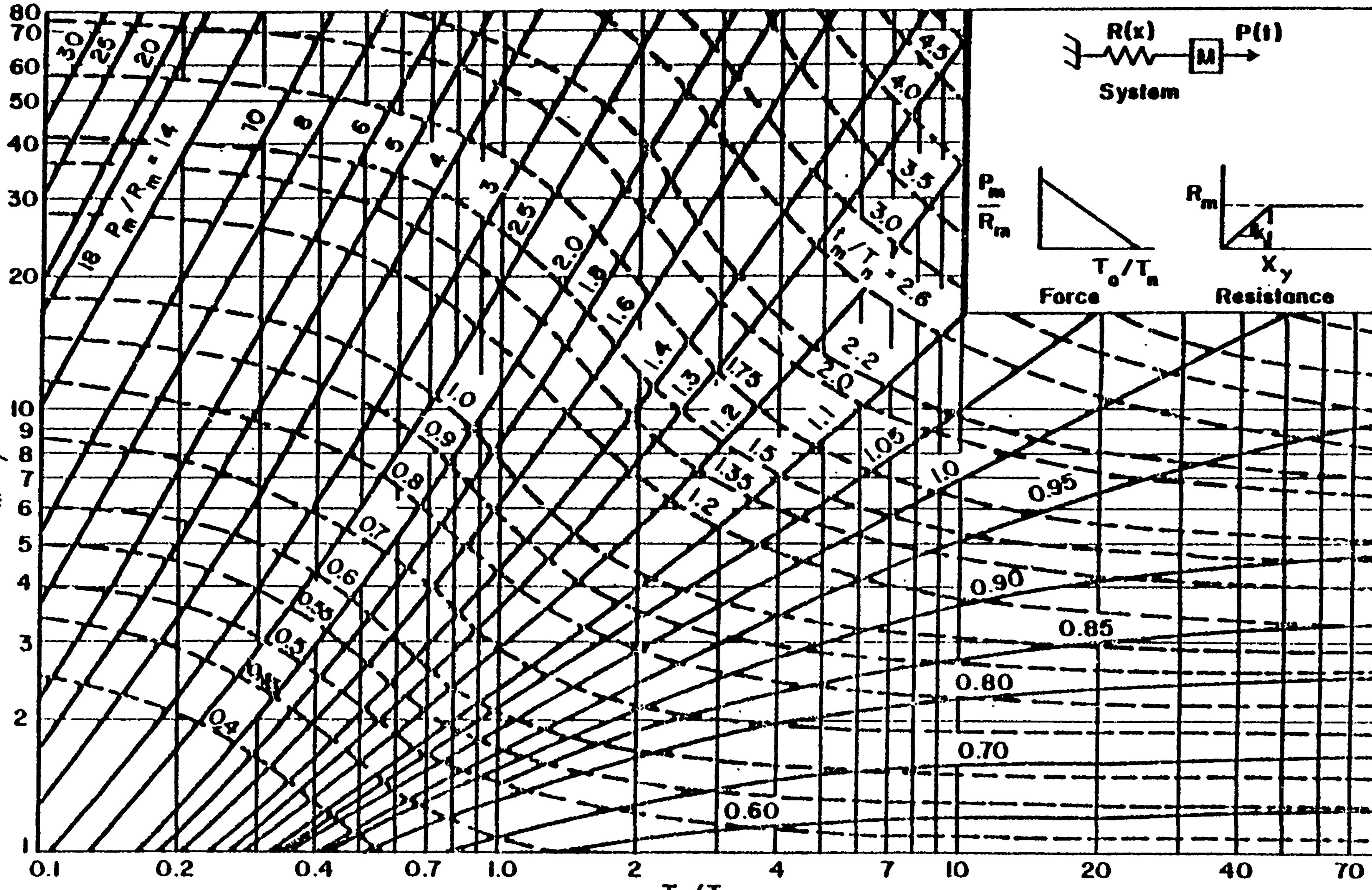


Figure 3-8. Maximum Response of Simple Spring-Mass Systems to Initially Peaked Triangular Force Pulses (Ref. 1)

Having now established values for u and T_o/T_{nc}, a value of P_m/R_m is determined using the response chart where:

$P_m = P_a$ = the actual thrust applied to the arch, psi

$R_m = R_c$ = the required compressive capacity of the arch, psi

The required compressive capacity of the arch as designed can now be solved for using the following expression:

$$R_c = P_a/(P_m/R_m), \text{ psi} \qquad (3.22)$$

A quick check of the actual compressive capacity is now possible by the following:

$$R_{mc}/R_m \leq 1 \qquad (3.23)$$

3.4.5 <u>Required Flexural Capacity</u>

In determining the resistance required in the flexural mode, the arch is again treated as a simply supported beam with length L_a, (Eq. 3.14), as before. The natural frequency of this equivalent beam, in the bending mode, is determined and is then corrected for curvature using the following correction factor, (μ):

$$\mu = (n^2 + 1.5)/(n^2 - 1) \qquad (3.24)$$

An equation that has been derived for the natural flexural period, (T_{nf}), of a simply supported beam is:

$$T_{nf} = 2\pi((XLM \times M_t)/K)^{1/2} \times \mu, \text{ sec or ms} \quad (3.25)$$

where 2π again establishes one complete cycle, K is the <u>stiffness</u> of a simply supported beam, and M_t is the <u>mass</u> per unit arch length, (mL_a). XLM is a transformation factor that requires further discussion at a later point. These components are determined as follows:

$$K = 384E_c I_a/5L^3, \text{ lb/in} \quad (3.26)$$

where:

I_a = the effective moment of inertia, using the average of the uncracked and cracked transformed sections

and:

$$I_a = (bd^3/2)(5.5p + 0.083), \text{ in}^4\text{/in} \quad (3.27)$$

XLM is a load-mass factor originating from the concept of transformation factors. Transformation factors are used to convert systems of two or more degrees of freedom into equivalent one degree of freedom systems, (another simplification). They are obtained from an assumed deformed shape of the member that results when the dynamic load is statically applied. From this statically deflected shape of the beam comes an equivalent mass, (M_e), and an equivalent force, (F_e). These are used to determine the mass factor, (XM), and the load factor, (XL), where:

$$XM = M_e/M_t \tag{3.28}$$

$$XL = F_e/F_t \tag{3.29}$$

and:

F_t = the static distribution of the dynamic load over the length of the beam, lb/ft

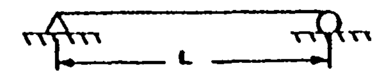
The load-mass factor can then be expressed as:

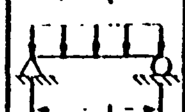
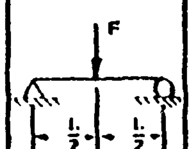
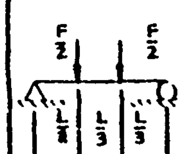
$$XLM = XM/XL \tag{3.30}$$

Tables containing these transformation factors have been produced for beams and slabs having different boundary and loading conditions. Table 3-1 shows the transformation factors for a simply supported beam with a uniformly distributed load. These tables only include values for the fully elastic condition, (u = 1), and for the fully plastic condition, (u = 20). Values corresponding to ductility ratios between one and twenty may be linearly interpolated.

The subject of the ductility ratio and a short discussion on elasto-plastic design in the flexural mode is now necessary. Plastic behavior is not generally permissible under continuous operating conditions, but, is quite

Table 3-1. Transformation Factors for Beams and One-Way Slabs, Simply Supported (Ref. 1)



Loading diagram	Strain range	Load factor K_L	Mass factor K_M Concentrated mass*	Mass factor K_M Uniform mass	Load-mass factor K_{LM} Concentrated mass*	Load-mass factor K_{LM} Uniform mass	Maximum resistance R_m	Spring constant K	Dynamic reaction V
 $F = pL$	Elastic	0.64	----	0.50	----	0.78	$\frac{8M_p}{L}$	$\frac{384EI}{5L^3}$	$0.39R + 0.11F$
	Plastic	0.50	----	0.33	----	0.66	$\frac{8M_p}{L}$	0	$0.38R_m + 0.12F$
	Elastic	1.0	1.0	0.49	1.0	0.49	$\frac{4M_p}{L}$	$\frac{48EI}{L^3}$	$0.78R - 0.28F$
	Plastic	1.0	1.0	0.33	1.0	0.33	$\frac{4M_p}{L}$	0	$0.75R_m - 0.25F$
	Elastic	0.87	0.76	0.52	0.87	0.60	$\frac{6M_p}{L}$	$\frac{56.4EI}{L^3}$	$0.525R - 0.025F$
	Plastic	1.0	1.0	0.56	1.0	0.56	$\frac{6M_p}{L}$	0	$0.52R_m - 0.02F$

appropriate for structures subject to a severe dynamic load only once or twice during it's lifetime. This is normally the case in blast-resistant design. Elasto-plastic design allows a greater portion of the energy absorbing capacity of the structure to be utilized resulting in a more economical design. The ductility ratio selected ultimately depends on the facility's function and the amount of damage that may be tolerated. A ductility ratio of three would allow moderate damage to occur, indicating that the steel would probably yield and concrete probably crack, however, there would be no significant impairment of the structure's resistance to future loading. For the purposes of this report, basic and initial survival will be the assumed criteria and a ductility ratio of ten, (μ = 10), will be used in the flexural mode.

The actual load duration, ($T_o = T_{app}$), is again compared to the natural period of vibration in the flexural mode insuring that 'resonance' will not be a problem. It should be noted that the duration of blast waves resulting from non-nuclear explosions are usually quite short with respect to the natural period of flexural vibrations. As a rule, when the ratio of T_o/T_{nf} is less than or equal to one fifth, the shape of the load function is not an important factor and response charts such as Figure 3-8 may not be used. In that event, P_m/R_m is determined through the following expression:

$$P_m/R_m = (T_{nf}/\pi T_o)(2u - 1)^{1/2} \qquad (3.31)$$

where:

$R_m = R_{fr}$ = the required flexural capacity of the structure, lbs

The required flexural capacity of the arch can now be solved by:

$$R_{fr} = P_{cu}/(P_m/R_m) \times L_{u}b, \text{ lbs} \qquad (3.32)$$

A quick check of the adequacy of the flexural capacity provided by the structure as designed would again be performed using:

$$R_{mfr}/R_{fr} \leq 1 \qquad (3.33)$$

3.4.6 <u>Combined Loading Capacity</u>

The assumption, in Figure 2-6, that the compressive and flexural modes occur simultaneously, (combired axial and bending), dictate that an interaction check must be performed. For reinforced concrete this leads to the use of an interaction diagram, (Figure 3-9).

Figure 3-9 represents two possible modes of failure. A compression or concrete crushing failure is indicated along the straight portion of the curves, whereas, a tension or tensile yielding of steel failure is indicated along the curved portion. The location on the curve reveals whether the flexural or axial loading condition predominates within the structure. Furthermore, if the given design is located on the curved portion, additional compressive axial loads increase the allowable bending capacity of the member or structure.

Again, it is known that non-nuclear explosions usually result in very short and intense peak overpressures so that the shock waves are acting on the structure for extremely short periods of time. With this in mind, the most severe flexural loading will probably occur before the arch is entirely engulfed by the blast and this potential increase in allowable bending capacity cannot be fully taken advantage of.

3.4.7 Dynamic Reactions and Shear

Of great importance to the loading conditions that will result on supports and foundations and to the design of these parts of the structure, is the dynamic reaction. This is the downward force produced at the base due to the dynamic loads applied. Since the design of the foundation has not been included in the scope of this project, these reactions will only be used to check for shear problems.

The equations for dynamic reactions, (V), included in Table 3-1 were arrived at by the following method. The arch is again assumed to be acting as a simply supported beam. An equilibrium analysis is performed on this beam using the applied dynamic loads in addition to an inertia force acting upward on the beam. This inertia force has a distribution equal to the previously assumed deflected shape. Any static loads are then added, as appropriate, to arrive at the total

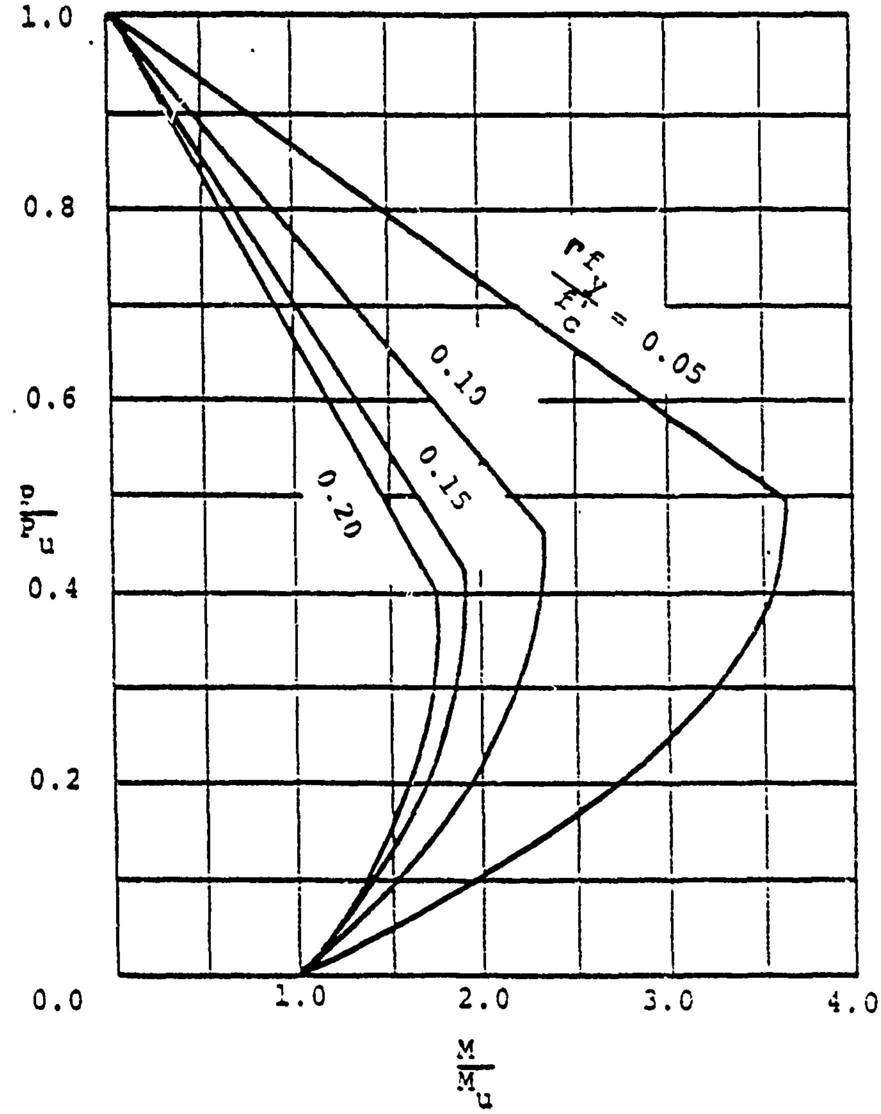


Figure 3-9. Interaction Diagram for Reinforced Concrete Beam-Columns (Ref. 1)

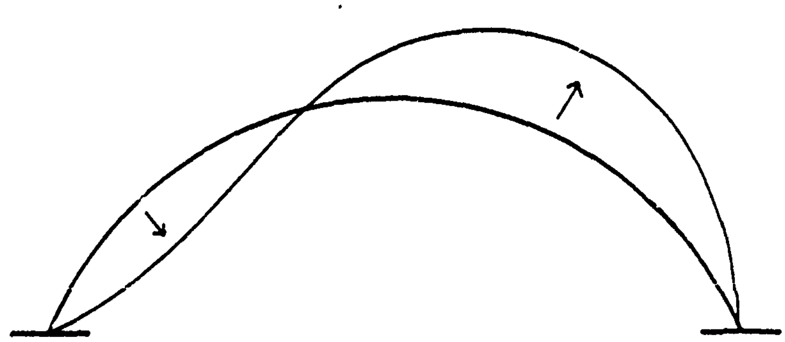


Figure 3-10. Buckling in Aboveground Arches

reaction at the supports. The two variables in this dynamic reaction equation are:

R = the maximum required dynamic resistance of the structure

F = the load acting on the structure when the maximum deflection occurs or when the system goes plastic.

In short duration loads created by non-nuclear explosions, it can generally be assumed that the load has already passed, (F = 0), by the time plasticity or maximum deflection conditions occur.

This dynamic reaction will then be compared to the allowable vertical shear, (V_{all}), and the allowable diagonal shear, (VT_{all}), values; they are:

$$V_{all} = (d/h)0.2f'_c bd, \text{ lbs} \qquad (3.34)$$

$$VT_{all} = bd(2\sqrt{f'_c} + p_v f_y), \text{ lbs} \qquad (3.35)$$

where:

p_v = steel ratio of diagonal reinforcement, (stirrups)

3.4.8 Buckling

Arch facilities such as the ones being considered, normally have relatively high slenderness ratios. This necessitates a stability or buckling check to be performed. Buckling in arches occurs as an inward deflection on one side and an outward deflection on the other side, (Figure 3-10).

Basic relations from static principles are again used in this evaluation. An arch or ring subject to a uniform radial load, (P_c), has a thrust, (S), where:

$$S = P_c \times r, \text{ lbs/in of width} \qquad (3.36)$$

The critical buckling value, (S'), for the equivalent beam that has been used elsewhere in the analysis is as follows:

$$S' = \pi^2 EI/L^2, \text{ lbs/in of width} \qquad (3.37)$$

Using the curvature correction factor from the flexural analysis the following critical value, (S_{cr}), is obtained for the arch:

$$S_{cr} = \mu_r \times S', \text{ lbs/in} \qquad (3.38)$$

Converting this into a critical pressure, (P_{cr}), is accomplished by:

$$P_{cr} = S_{cr}/r, \text{ psi} \qquad (3.39)$$

and combining the last three equations gives:

$$P_{cr} = (EI/r^3)(n^2 - 1), \text{ psi} \qquad (3.40)$$

This value is then compared against the required compressive capacity to check for potential buckling failures as follows:

$$R_c / P_{cr} \leq 1 \tag{3.41}$$

At this point, depending on the outcome of the various checks, the section may need to be revised and analyzed again. It should not require many iterations for the designer to realize which area(s) are going to be governing throughout the process and, knowing this, use them to arrive at a final design much more quickly.

CHAPTER FOUR
LOTUS 1-2-3 STUDENT EDITION

4.1 Brief Description

LOTUS 1-2-3, Student Edition, (SE), is primarily a smaller version of the widely used LOTUS 1-2-3 software package; smaller meaning that the worksheet or spreadsheet is not as large as in the main package. The student edition was introduced as a teaching tool for the larger edition. The three main features of these products are the worksheet, the graphics capabilities, and database management. The student edition worksheet was used for this report and, even with its' reduced capabilities, proved to be more than adequate for the intended purpose.

4.2 Why Used

As discussed and established earlier, these designs are no different than most in that they are performed by trial and error using an iterative process. Even using a simplified method, the process becomes extremely time consuming after only a few iterations. Since a total of eight different arch configurations will eventually be examined in this report, a more expedient method of carrying out these iterations was sought. The LOTUS 1-2-3, SE, software package was the means chosen to accomplish this.

4.3 How Used

The LOTUS SE software turned out to be ideal for the scope of work involved in these designs. The many equations, constants, and variables used in the design process were input onto the LOTUS SE spreadsheet. The variables that changed from iteration to iteration and case to case primarily involved the physical configuration of the structure, (d, h, Γ, r, β, p_t, D, and H), and the loading conditions, (P_w and T_w). It was simply a matter of changing these values, as appropriate, for each design case and revision. Immediate results of the structural analysis could then be obtained, including all steps, evaluations, and checks previously discussed.

The calculations for the information that will be used in the final comparisons were also performed on LOTUS SE. These are such quantities as total volume of concrete required, (V_c), total weight of concrete required, (WT_c), total weight of steel required, (WT_s), total structural weight, (TOTWT), estimated cost of concrete required, ($\$_c$), estimated cost of steel required, ($\$_s$), and total estimated structural cost, (TOT$). These will be explained further in CHAPTER SIX.

CHAPTER FIVE
DESIGN SETUP

5.1 Case 1 - A through D

The scenario for Case 1 is to design a protective structure for an aircraft to be parked on the airfield apron at Air Force Base X which is in a location that may be expected to come under some form of aerial attack. The size of the aircraft is such that the minimum dimensions of the facility will be as follows:

Height(H)--60 Feet

Width(D)---30 Feet

Length(L)--100 Feet

Having already decided to use an aboveground arch, the four configurations selected for evaluation are shown in Figure 5-1. As evidenced by Figure 5-1, the structure becomes larger as the angle of incidence increases. It was also explained in CHAPTER TWO that the applied load decreases, (due to the reflected pressure), as the angle of incidence increases. This raises the question of whether the benefits extending from the smaller applied load are enough to overcome the disadvantages of increasing the size of the facility.

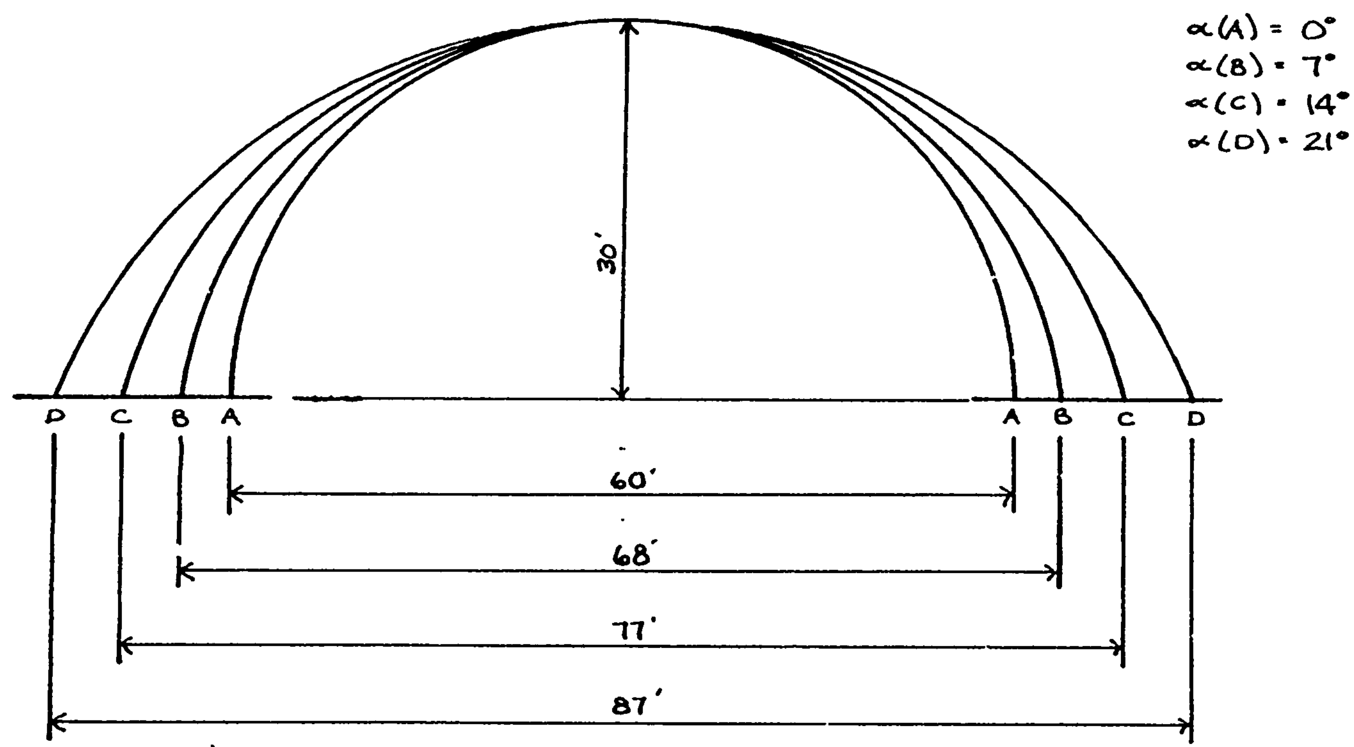


Figure 5-1. Case 1 Arch Configurations

5.2 Case 2 - A through D

For the second set of comparisons, (Case 2), an alternate aircraft required protection which dictated the following minimum required dimensions:

Height(H)--20 Feet

Width(D)---60 Feet

Length(Lt)-100 Feet

These limitations led to the four configurations shown in Figure 5-2.

As is evident from Figure 5-2, this case leads to a different type of comparison. The structures are increasing in size as the angle of incidence is decreasing. In addition, the larger structures are 'growing up' as opposed to 'growing out' as in Case 1. It seems obvious here that Case 2A would be the best configuration to use since it has both the benefit of being the smallest structure and of having the largest angle of incidence. This will lead to a comparison of the resulting equivalent load functions based more on facility dimensons, (between Cases 1 and 2), as opposed to a load vs. angle of incidence comparison.

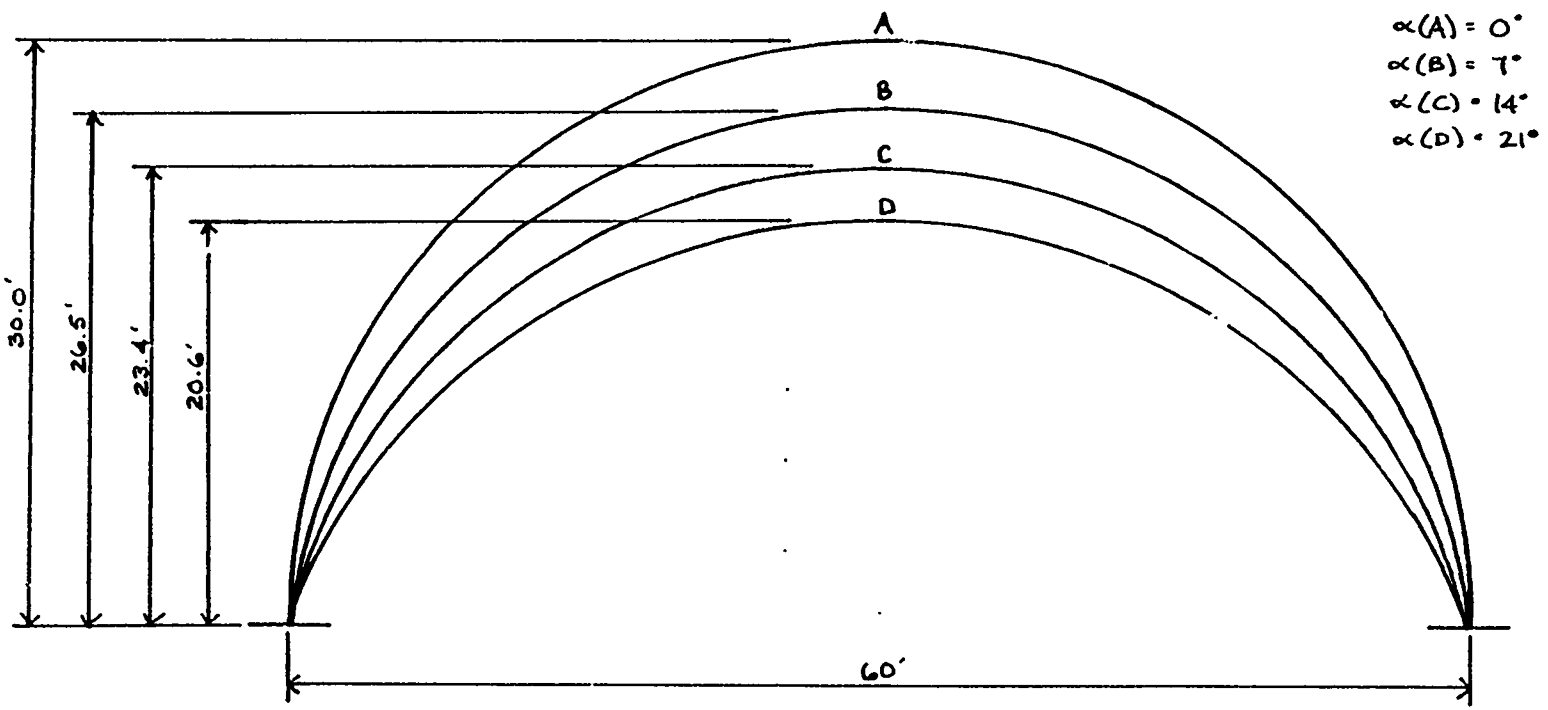


Figure 5-2. Case 2 Arch Configurations

5.3 Actual Threat Defined

Through intelligence procedures it has been determined that the 'worst-case' threat that will be faced by these structures is the detonation of a 1000 lb, General Purpose Bomb, detonated at the ground surface 25 feet from the structure, (Figure 5-3). General Purpose Bombs, (GP), are a high - explosive type or class of bomb, (Figure 5-4). The metal casing on GP bombs is generally thicker than on a Light-Cased Bomb, but, thinner than on armor-piercing type bombs. The casing is strong enough to withstand direct impact on most industrial construction. They are also able to penetrate soil without deformation or rupture, but, will normally break up if dropped on a heavy concrete slab. These bombs can be used with delayed fusing as well as fuses that are set to activate on contact with the air or ground. The explosive charge weight to overall weight ratio is usually around fifty percent. The primary concerns in designing aboveground protective facilities against this weapon are blast effects, fragmentation, and penetration. Only blast effects and fragmentation will be considered in these cases as a direct hit is not assumed to be probable.

In order to determine the eventual loading conditions that will result from this threat, some preliminary calculations are required using the specifications on this particular type of weapon, (Table 5-1). Figure 5-5 is used to evaluate the equivalent bare charge weights for both the

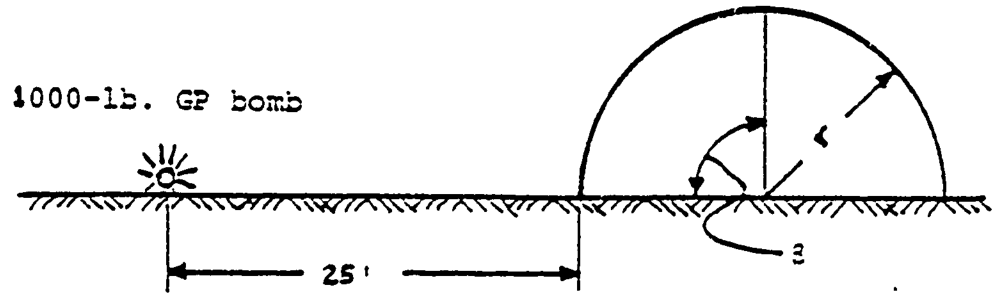


Figure 5-3. Arch Design Conditions (Ref. 1)

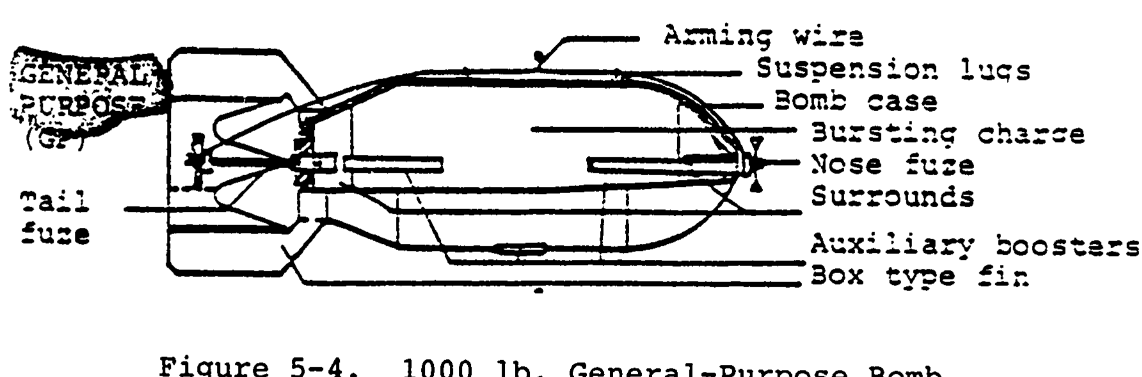


Figure 5-4. 1000 lb. General-Purpose Bomb (Ref. 1)

Table 5-1. Characteristics of Typical Bombs (Ref. 1)

Weapon	Total Wt., lb	Charge Wt., lb	Case Maximum O.D., in	Case Thickness in.
LIGHT-CASE BOMBS				
4000# LC	4531	3690 (Tritonal)	34.0	0.37
GENERAL-PURPOSE BOMBS				
100# GP	119.5	57 (TNT)	8.2	0.16
250# GP	263	127 (TNT)	10.9	0.27
500# GP	549	266 (TNT)	14.2	0.30
1000# GP	1064	555 (TNT)	18.8	0.50
2000# GP	2212	1220 (Tritonal)	23.3	0.50
4000# GP	4229	2002 (Tritonal)	28.0	0.83
SEMI-ARMOR PIERCING BOMBS				
500# SAP	494	162 (TNT)	11.8	0.75
1000# SAP	1040	315 (TNT)	15.1	1.00
2000# SAP	2040	556 (TNT)	19.7	1.5
ARMOR-PIERCING BOMBS				
1000# SAP	1025	140 (Explosive D)	12.0	1.1
1600# SAP	1590	225 (Explosive D)	14.0	1.3
FRAGMENTATION BOMBS				
4# F	3.2	0.5 (TNT)	3.0	0.25
20# F	20.0	2.7 (TNT)	3.6	0.56
90# F	90	11 (Comp B)	6.0	0.94
220# F	216	47 (Comp B)	8.0	1.00
260# F	260	34 (Comp B)	8.0	1.25

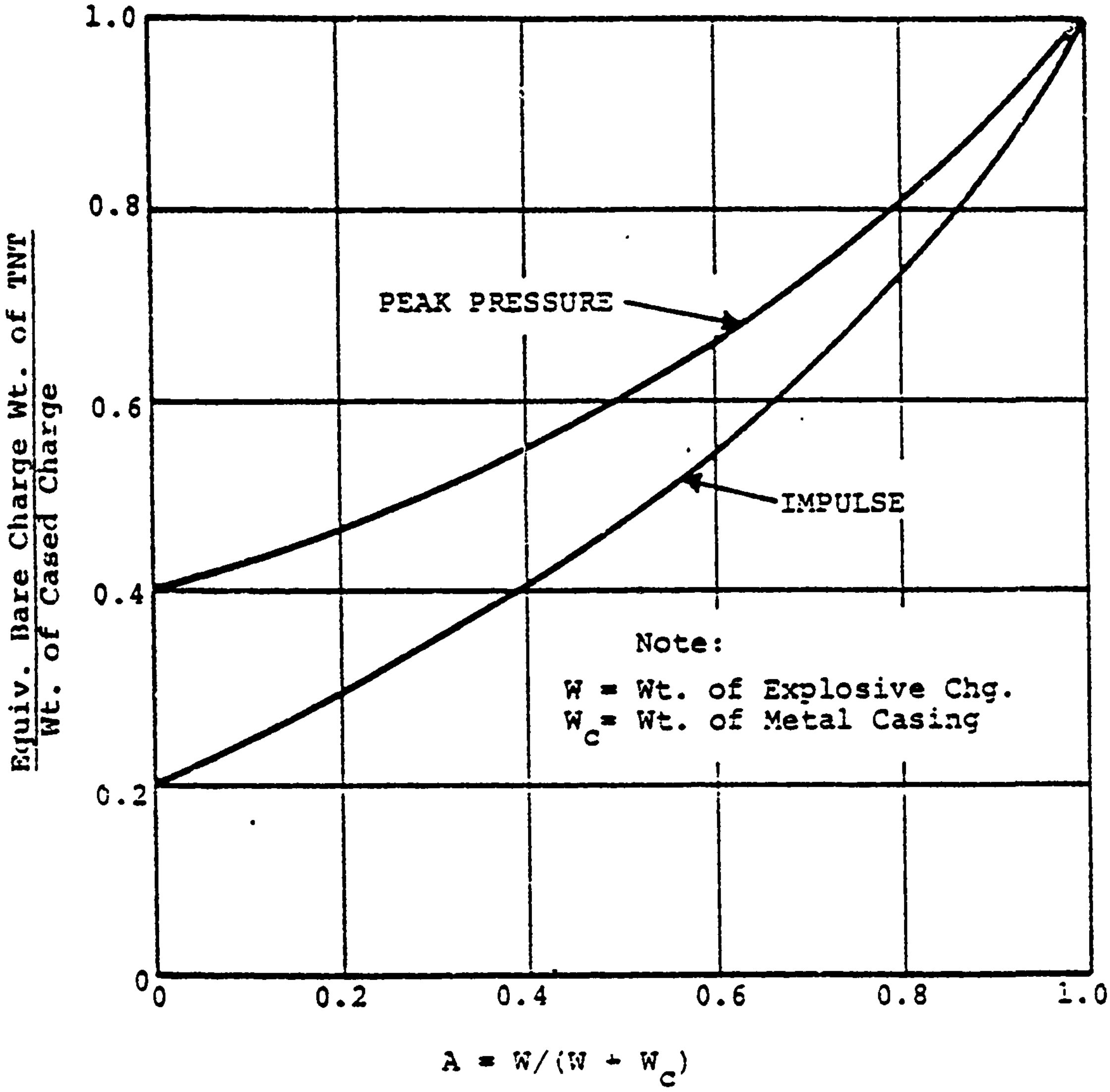


Figure 5-5. Equivalent Bare TNT Weight for Peak Pressure and Impulse for Cased Charges (Ref. 1)

peak incident pressures and impulses, (W_p and W_i), accounting for the effects of the bomb casing and the type of explosive used. Hence, from Figure 5-5 and Table 5-1:

$$W_c = \text{bomb casing weight} = 1064 - 555 = \underline{509 \text{ lbs}}$$

and:

$$A = W/(W + W_c)$$
$$= 555/(555 + 509) = \underline{0.522}$$

and, from Figure 5-5:

$$W_p/W = 0.615 \text{ and } W_p = (W)(W_p/W) = (555)(0.615) = \underline{341 \text{ lbs TNT}}$$
$$W_i/W = 0.486 \text{ and } W_i = (W)(W_i/W) = (555)(0.485) = \underline{269 \text{ lbs TNT}}$$

It is normal to add a 20% safety factor to these charge weights which leads to:

$$W_p = (341)(1.2) = \underline{409 \text{ lbs TNT}}$$
$$W_i = (269)(1.2) = \underline{323 \text{ lbs TNT}}$$

5.4 Actual Loads Established

The weights calculated in Section 5.3 are now used to determine the various loading parameters to be resisted by these structures. Converting the above weights into the scaled distances used in Figure 3-1 gives:

$$Z_p = R_g / W_p^{1/3} = 25/409^{1/3} = \underline{3.37}$$

$$Z_i = R_g / W_i^{1/3} = 25/323^{1/3} = \underline{3.64}$$

The appropriate loading parameters are now determined using Figures 2-4, 2-5, and 3-1, along with the methods presented in Section 3.3. These are then used to establish the equivalent load functions that will be applied in each case.

5.4.1 Case 1 Loads

The load conditions to be resisted by the four arch configurations of Case 1 are as follows:

$$P_{so} = \underline{105 \text{ psi}}$$

$$Q_o = \underline{120 \text{ psi}}$$

$$I_s = \underline{130}$$

$$U = \underline{3000 \text{ fps or } 3 \text{ ft/ms}}$$

$$P_r(A) = \underline{518 \text{ psi}},\quad P_r(B) = \underline{513 \text{ psi}}$$

$$P_r(C) = \underline{503 \text{ psi}},\quad P_r(D) = \underline{485 \text{ psi}}$$

$$T_o f = 2(130)/105 = \underline{2.5 \text{ ms}}$$

$$T_{app}(A) = 2.5 + 60/3 = \underline{22.5 \text{ ms}}$$

$$T_{app}(B) = 2.5 + 68/3 = \underline{25.2 \text{ ms}}$$

$$T_{app}(C) = 2.5 + 77/3 = \underline{28.2 \text{ ms}}$$

$$T_{app}(D) = 2.5 + 87/3 = \underline{31.5 \text{ ms}}$$

so:

$$P_e(A) = \tfrac{1}{2}(105 + 518 - 120)(2.5/22.5) = \underline{27.9 \text{ psi}}$$

$$P_e(B) = \tfrac{1}{2}(105 + 513 - 120)(2.5/25.2) = \underline{24.7 \text{ psi}}$$

$$P_e(C) = \tfrac{1}{2}(105 + 503 - 120)(2.5/28.2) = \underline{21.6 \text{ psi}}$$

$$P_e(D) = \tfrac{1}{2}(105 + 485 - 120)(2.5/31.5) = \underline{18.6 \text{ psi}}$$

where $C_d = 1$, (Reference 4)

5.4.2 Case 2 Loads

The load conditions to be resisted by the four configurations of Case 2 are as follows:

$$P_{so} = \underline{105 \text{ psi}}$$

$$Q_{o} = \underline{120 \text{ psi}}$$

$$I_{w} = \underline{130}$$

$$U = \underline{3000 \text{ fps or } 3 \text{ ft/ms}}$$

$$P_r(A) = \underline{518 \text{ psi}}, \quad P_r(B) = \underline{513 \text{ psi}}$$

$$P_r(C) = \underline{503 \text{ psi}}, \quad P_r(D) = \underline{485 \text{ psi}}$$

$$T_{of} = \underline{2.5 \text{ ms}}$$

$$T_{app}(A, B, C, \text{and } D) = \underline{22.5 \text{ ms}}$$

$$P_{ew}(A) = \tfrac{1}{2}(518 + 105 - 120)(2.5/22.5) = \underline{27.9 \text{ psi}}$$

$$P_{ew}(B) = \tfrac{1}{2}(513 + 105 - 120)(2.5/22.5) = \underline{27.7 \text{ psi}}$$

$$P_{ew}(C) = \tfrac{1}{2}(513 + 105 - 120)(2.5/22.5) = \underline{27.1 \text{ psi}}$$

$$P_{ew}(D) = \tfrac{1}{2}(485 + 105 - 120)(2.5/22.5) = \underline{26.1 \text{ psi}}$$

CHAPTER SIX
DESIGN AND ANALYSIS

6.1 Preliminaries

Prior to beginning the actual design and analysis process, there are certain assumptions to be made and certain criteria to be established. The static material properties are normally revised into their dynamic counterparts as follows:

f'_c = <u>4000 psi</u> (assumed)

$f'_{dc} = 1.25 \times f'_c = 1.25 \times 4000 =$ <u>5000 psi</u> (Table 6-1)

f_y = <u>40000 psi</u> (assumed)

$f_{dy} = 1.1 \times f_y = 1.1 \times 40000 =$ <u>44000 psi</u> (Table 6-2)

$E_c = 33(w_c)^{1.5}\sqrt{f'_c} = 33(145)^{1.5}\sqrt{4000} =$ <u>3.64 x 10 psi</u>

The maximum and minimum steel reinforcement ratios required to insure 'ductile' type failures are computed as they would be for conventional reinforced concrete design and are as follows:

$$p_{max} = \underline{0.03712}$$

$$p_{min} = \underline{0.00500}$$

Another assumption to be made will be that the total thickness of the concrete, (h), equals 2½" + d. This usually insures that the required concrete cover limitations are satisfied.

Table 6-1. Recommended Dynamic Design Concrete Stresses (Ref. 1)

Dynamic Compressive Strength	$f'_{dc} = 1.25f'_c$
Dynamic Bond Stress (ASTM A305)	$u_d = 0.15f'_c$
Pure Shear Stress	$v_{dy} = 0.20f'_c$
Dynamic Tensile Strength	$f_{dt} = 7.5\sqrt{f'_c}$

Table 6-2. Dynamic Yield Stresses, Reinforcing Steel (Ref. 1)

Structural Grade	$f_{dy} = 44$ ksi
Intermediate Grade	$f_{dy} = 52$ ksi

6.2 Long-Hand Iteration - Case 1A

For this case only, one iteration will be performed long-hand to further present and clarify the design procedure being used. All other cases will be performed using the LOTUS 1-2-3 SE Program as described in CHAPTER FOUR. The results for the final designs selected for comparison in each case will be compiled and summarized at the end of this chapter. The actual inputs and outputs, (LOTUS SE printouts), for each case examined are located in APPENDIX A. Some steel reinforcement details and sections will also be included as a part of this iteration to indicate how the final configuration might appear. In addition, a check against the fragmentation that the structure would be subject to will also be shown to indicate how it might affect the final designs.

6.2.1 Design and Analysis

As previously discussed, an initial trial section is often selected on the basis of some static design criteria. Here, the critical static buckling load is going to be used to determine an initial thickness of concrete. From the previous discussion on buckling, the equivalent load, Pe(A), is set equal to the critical buckling stress, which, from Equation 3.40, gives:

$$P_{cr} = 3EI/r^3$$

and rearranging:

$$I = P_m r^3$$

$$= 27.9(360)^3/3(3640000) = \underline{119\ in^4/in\ of\ width}$$

Using an initial steel ratio of $\frac{1}{2}p_{max}$, and rearranging Equation 3.27 to solve for d gives:

$$d = (2I/(b(5.5p + 0.083)))^{1/3}$$

$$= ((2 \times 119)/(1(5.5(0.01856) + 0.083)))^{1/3}$$

$$= \underline{10.87\ in,\ say\ 11\ in}$$

In sum then, the initial trial section for Case 1A has the following characterisitics:

$d = \underline{11\ in}$

$h = \underline{13\frac{1}{2}\ in}$

$p = \underline{0.01856}$

$p_t = 2p = 2(0.01856) = \underline{0.03712}$, (See Section 6.2.2)

$p_1 = \underline{0.0025}$, (See Section 6.2.2)

The actual compressive capacity of this section is:

$$R_{mc} = (0.85f'_{dc} + p_t(f_{dy}))h/r$$

$$= (0.85(5000) + 0.03712(44000) \times (13.5)/360$$

$$= \underline{221\ psi}$$

The required compressive capacity of this section with the calculated loads applied comes from the following:

$$m = bh\gamma_c / 1728g$$

where 1728 is a conversion factor for units, so:

$$m = (1)(13.5)(145)/(1728)(386) = \underline{0.00293\ \text{lb-sec}^2/\text{in}^2}$$

and:

$$T_{nc} = 2\pi r(m/EA_c)^{1/2} = 2\pi(360)(0.00293/3640000 \times 13.5)^{1/2}$$
$$= \underline{0.0175\ \text{sec}}$$

therefore:

$$T_o/T_{nc} = 0.0225/0.0175 = \underline{1.29}$$

and from Figure 3-8:

$$P_m/R_m = \underline{0.62}$$

resulting in:

$$R_c = 27.9/0.62 = \underline{45\ \text{psi}}$$

Obviously, compression is not a problem in this configuration since:

$$R_c/R_{mc} = 45/221 = \underline{0.2036 \lll 1}$$

The actual flexural capacity of this section is as follows, where:

$$\mu_f = 1 - 1/n^2 = 1 - \tfrac{1}{4} = \underline{0.75}$$

$$a = A_s(f_{dy})/0.85f'_{dc}(b)$$

$$= (0.01856)(1)(13.5)(44000)/0.85(5000)(1) = \underline{2.31\ \text{in}}$$

and:

$$M_u = A_s(f_{dy})(d - a/2)$$

$$= (0.1856)(1)(13.5)(44000)(11 - 2.31/2) = \underline{96478\ \text{lb-in}}$$

hence:

$$R_{mf} = 8M_u/L_a \times \mu_f = 8(96478)/565.5 \times 0.75 = \underline{1024\ \text{lbs}}$$

where:

$$L_a = r\beta = 360(\pi/2) = \underline{565.5\ \text{in}}$$

The required flexural capacity of this arch with the dynamic loads applied is as follows, where:

$$M_t = mL_a = 0.0293 \times 565.5 = \underline{1.657\ \text{lb-sec/in}^2}$$

and:

$$K = 384EI/5L^3 = 384(3640000)(119)/5(565.5)^3 = \underline{184\ \text{lb/in}}$$

using:

$$u = \underline{10}\ \text{(Section 3.4.5)}$$

and interpolating from Table 3-1:

$$XLM = \underline{0.70}$$

so:

$$\mu = n^2 + 1.5/n^2 - 1 = \underline{1.833}$$

and:

$$T_{nf} = 2\pi(XLM \times M_b/K)^{1/2} \times \mu$$

$$= 2\pi(0.70(1.657)/184)^{1/2} \times 1.833$$

$$= \underline{0.913}$$

then:

$$T_o/T_{nf} = 0.0225/0.913 = \underline{0.0246} \leq 1/5$$

and does not appear in Figure 3-8 so:

$$P_m/R_m = T_{nf}/\pi T_o(2u-1)^{1/2}$$

$$= 0.913/\pi(0.025)(2(10) - 1)^{1/2} \quad = \underline{56.31}$$

and:

$$R_f = 27.9/56.31 \times 565.5(1) = \underline{280 \text{ lbs}}$$

Again, the section appears to be overdesigned in the flexural, as well, since:

$$R_f/R_{mf} = 280/1024 = \underline{0.2734 <<<< 1}$$

It would now be necesary to conduct an interaction check for the combined axial and bending loading condition. Using Figure 3-9 where:

$$p \times f_{dy}/f'_{dc} = 0.01856(44000)/5000 = 0.163$$

and:

$$P/P_u = R_c/R_{mc} = 0.2036 \text{ (Section 6.2.1)}$$

leads to an allowable M/M_u, (R_f/R_{mf}), of 1.70. This is significantly larger than the actual of 0.2727 calculated in Section 6.2.1. As pointed out in Section 3.4.6, this additional allowance can not be fully taken advantage of in these loading cases, which are of an extremely short duration. Regardless, any reduction in the section attempting to take advantage of this would give rise to a stability problem since buckling was the basis for the trial section. It appears, then, that these facilities must be overdesigned for strength requirements in order to meet the necessary stability requirements. Future iterations for this case as well as all others will attempt to find the most efficient design for strength while at the same time maintaining an adequate buckling capacity.

Going on to determine the dynamic reaction and performing shear checks give the following:

$$V = 0.38R + 0.12F + \tfrac{1}{2}DL \quad \text{(from Table 3-1)}$$

as explained, F can be taken as 0 and hence:

$V = 0.38(280) + (565.5)(1)(13.5)(145/1728) = \underline{641 \text{ lbs}}$

where the second portion of the expression represents the structure's dead load, (DL). This actual shear value is now checked against the allowable shear values as follows:

$$V_{all} = (d/h)0.2(f'_c bd$$

$$=(11/13.5)0.2(4000)(1)(11) = \underline{7170 \text{ lbs} > 641 \text{ lbs}} \quad \text{(OK)}$$

and:

$$VT_{all} = bd(2\sqrt{f'_c} = p_v(f_y)), \quad \text{(no stirrups assumed, } p_v=0)$$

so:

$$VT_{all} = (1)(11)(2\sqrt{4000}) = \underline{1391 \text{ lbs} > 641 \text{ lbs}} \quad \text{(OK)}$$

6.2.2 Placement of Reinforcement - Case 1A

Using the resulting steel ratios, the required areas of steel, (A_s and A_1), can now be calculated leading to the selection and placement of the actual bars into the concrete section. A brief evaluation is carried out here for this iteration only to clarify this point. It is not the intent of this report to go into great detail on steel reinforcing placement.

In dynamic design, another consideration in the placement of the steel reinforcement is rebound. Rebound is the reverse deflection occurring after the initial positive deflection or after one half of the vibratory cycle has occurred. The structural element is designed as a singly reinforced member with steel added to the opposite face as dictated by the amount of rebound that will occur. This subject will not be addressed in any more depth here except to say that charts do exist to assist in determining the extent of rebound that will occur in a given structure, (Figure 6-1).

As shown in Figure 6-1, a full 100% rebound, (R_r/R_{ry} = -1.0) normally occurs in structures where the load duration is short compared to the natural period of flexural vibration which, again, covers all cases in this report. Full rebound dictates that an equal amount of steel reinforcement must be used in each face of the structural element.

The following reinforcement calculations will be based upon a rectangular concrete section with the dimensions h x 12". The amount of primary steel required in each face will then be as follows:

$$A_s = pbd$$

$$= 0.01856 \times 12 \times 11 = \underline{2.45\ in^2/ft\ width}\ \text{(each face)}$$

Selecting #8 bars spaced, (s), at 3.5 inches o.c. gives:

$$A_s = A_b/(s/12)$$

where:

$$A_b = \underline{0.79\ in^2}$$

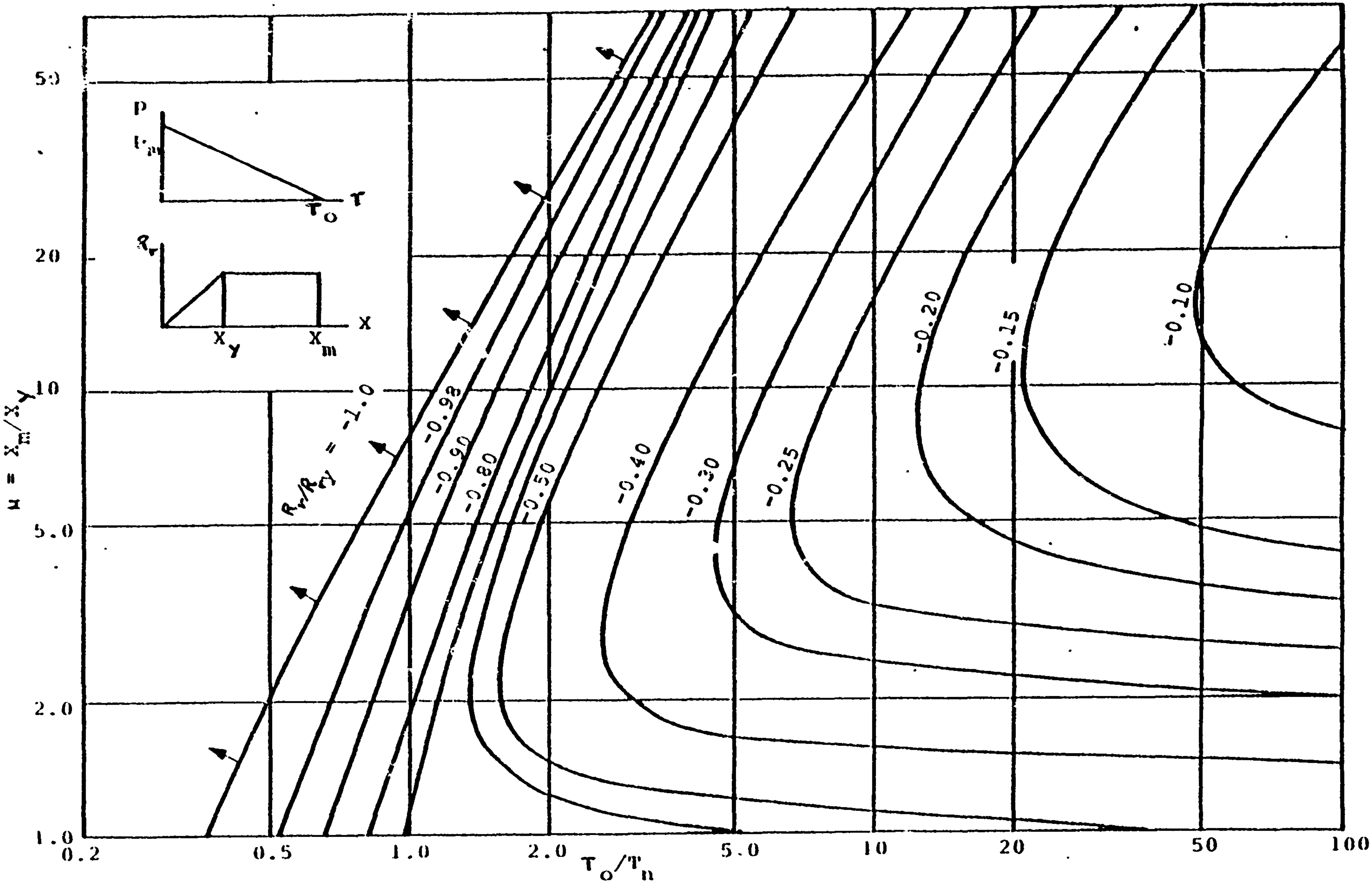


Figure 6-1. Design Chart for Elastic Rebound (Ref. 1)

so:

$$A_s = 0.79/(3\frac{1}{2}/12) = \underline{2.71\ in^2/ft\ width > 2.45} \quad (OK)$$

Reference 1 recommends a minimum longitudinal steel ratio of 0.0025 primarily to control creep and temperature variations, (shrinkage and swelling). Hence, the steel required in the longitudinal direction is as follows:

$$A_l = p_l bd = 0.0025 \times 12 \times 11 = \underline{0.33\ in^2/ft\ width}$$

Selecting #6, (A_b = 0.44 in^2), bars spaced at 16" o.c. gives:

$$A_l = 0.44/(16/12) = \underline{0.33\ in^2/ft\ width = 0.33} \quad (OK)$$

Some general sections and details of the actual placement of these calculated steel requirements are shown in Figure 6-2. These details also consider the clearance and spacing limitations as outlined in the ACI Code, (Reference 6).

It appears that using $\frac{1}{2}p_{max}$ results in quite a large amount of steel reinforcement to be placed in the primary direction which accounts for some of the overdesign for strength that occurred earlier. While this may be necessary to insure 'ductile' type failures, the most economical designs would minimize the amount of steel used. Therefore, for the sake of comparing, all subsequent designs to be performed on the LOTUS SE program will attempt to minimize these steel requirements while at the same time meeting or exceeding all other limiting criteria.

6.2.3 Fragmentation

As mentioned in Section 5.3, another design consideration for a surface explosion of a 1000 lb bomb is the threat of fragmentation. The primary purpose of a fragmentation check is to determine if casing fragments caused by the explosion contain enough velocity and energy at impact with the structure to penetrate the protective envelope. Reference 1 also presents a method to carry out this procedure which will be used here. The general principles and basis for the steps involved in this process will be discussed as they occur.

An important factor in this procedure is the shape of the weapon, (cylindrical or non-cylindrical). From Figure 5.4 it can be seen that the shape of the 1000 lb GP Bomb is not clearly defined so a check will be performed assuming both conditions and the governing results will be used.

Again, using the required weapon specifications from Table 5-1, the W/W_c ratio for the given weapon is calculated. Since the non-cylindrically cased weapons tend to be much more unpredictable, safety factors are applied to the W/Wc ratio for those, giving the following:

$$W/W_c = 555/509 = \underline{1.09} \quad \text{(cylindrical)}$$

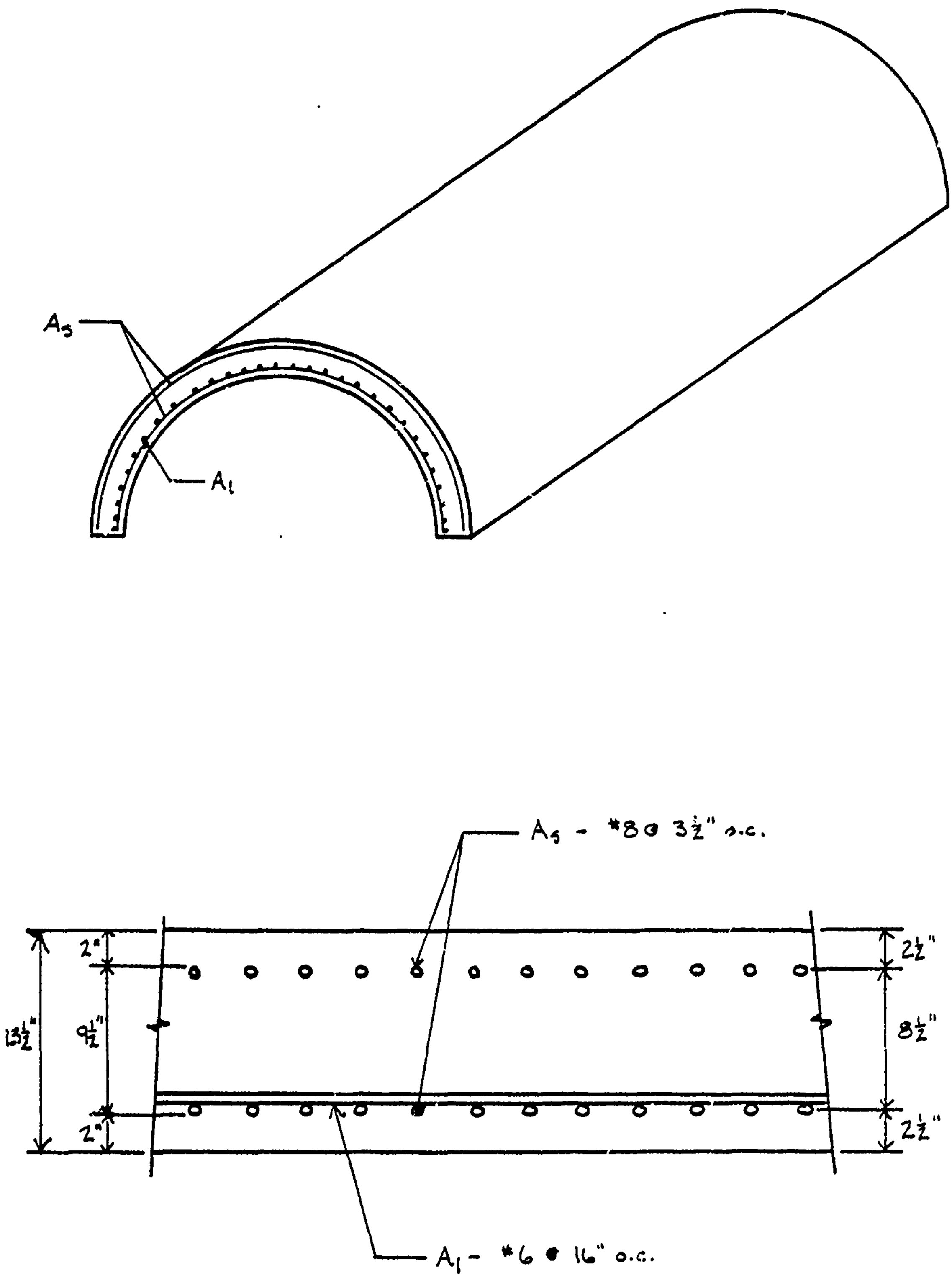


Figure 6-2. Steel Reinforcement Sections and Details

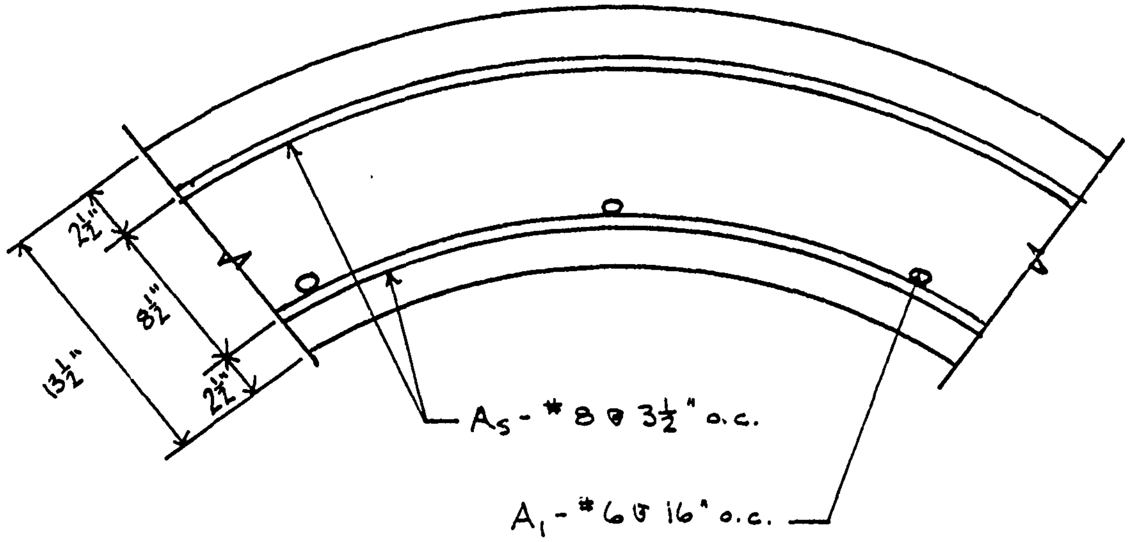


Figure 6-2 . - continued

Table 6-3. Constants for Primary Fragment Calculations (Ref. 1)

Explosive Material	Gurney Energy Constant $(2E')^{1/2}$	B
Amatol		0.35
Comp. B	7880	
H-6	7110	
Hexanite		0.32
Pentolite	7550	
RDX/TNT (75/25)	7850	
RDX/TNT (70/30)	8380	
RDX/TNT (60/40)	7880	0.27
TNT	6940	0.30
Tetryl	7460	0.24
Torpex	7450	

$$(W \times 1.2)/(W_c \times 0.5) = 566/254.5 = \underline{2.62} \text{ (non-cylindrical)}$$

From Table 6-3, (assuming mild steel casing):

$$B = 0.30$$

and:

$$(2E')^{1/2} = 6940, \text{ for TNT in a mild steel case}$$

The initial velocity of the primary fragments immediately after explosion, (V_o), is then determined using Figure 6-3 or:

$$V_o = (2E')^{1/2}((W/W_c)/(1 + W/2W_c))^{1/2}$$

which gives:

$$V_o = 6940(1.09/(1 + 1.09/2))^{1/2}$$

$$= \underline{5829 \text{ ft/sec}} \quad \text{(cylindrical)}$$

and:

$$V_o = 6940(2.62/(1 + 2.62/2))^{1/2}$$

$$= \underline{7391 \text{ ft/sec}} \quad \text{(non-cylindrical)}$$

The next step is to calculate a parameter that predicts the primary fragment distribution, (M_A), using:

$$M_A = B(T_c)^{5/6} \times D_i^{1/3}(1 + T_c/D_i)$$

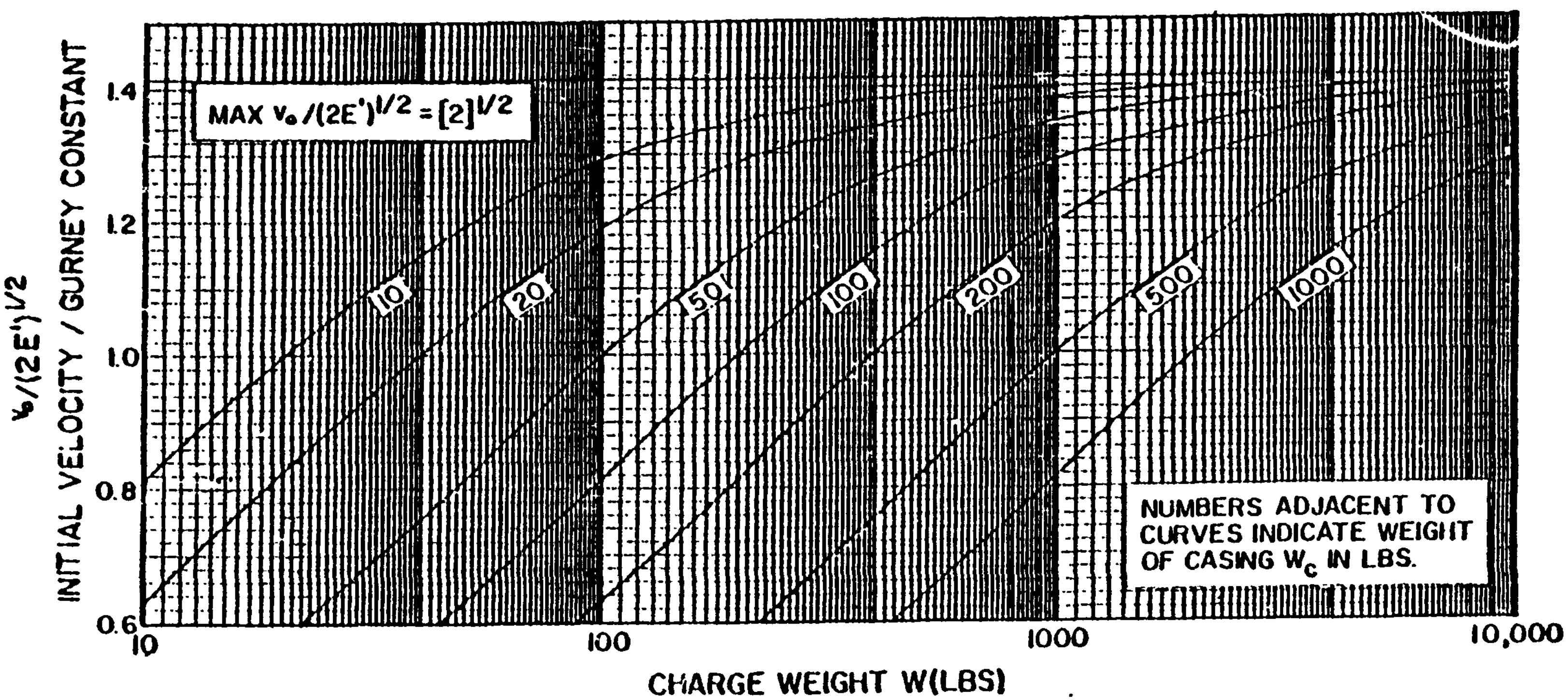


Figure 6-3. Variation of Primary Fragment Velocities with Distance (Ref. 1)

where:

T_c = case thickness, in

D_i = inside diameter of case, in

which, for both cases, gives:

$$M_A = 0.30(0.5)^{5/6} \times 17.8^{1/3}(1 + 0.5/17.8)$$
$$= \underline{0.452}$$

Using this parameter, or Figure 6-4, to determine the weight of the largest fragment, (W_f), where:

$$W_f = (M_A \times \ln(8W / M_A^2))^2$$

gives:

$$W_f = (0.452 \times \ln((8)(509)/0.452^2))^2$$
$$= \underline{2.12 \text{ oz}} \qquad \text{(cylindrical)}$$

and:

$$W_f = (0.452 \times \ln((8)(254.5)/0.452^2))^2$$
$$= \underline{2.04 \text{ oz}} \qquad \text{(non-cylindrical)}$$

The velocity of this largest fragment at impact with the structure, (V_s), is then calculated using:

$$V_s = (V_o)e^{(-0.004R_f / W_f^{1/3})}$$

where:

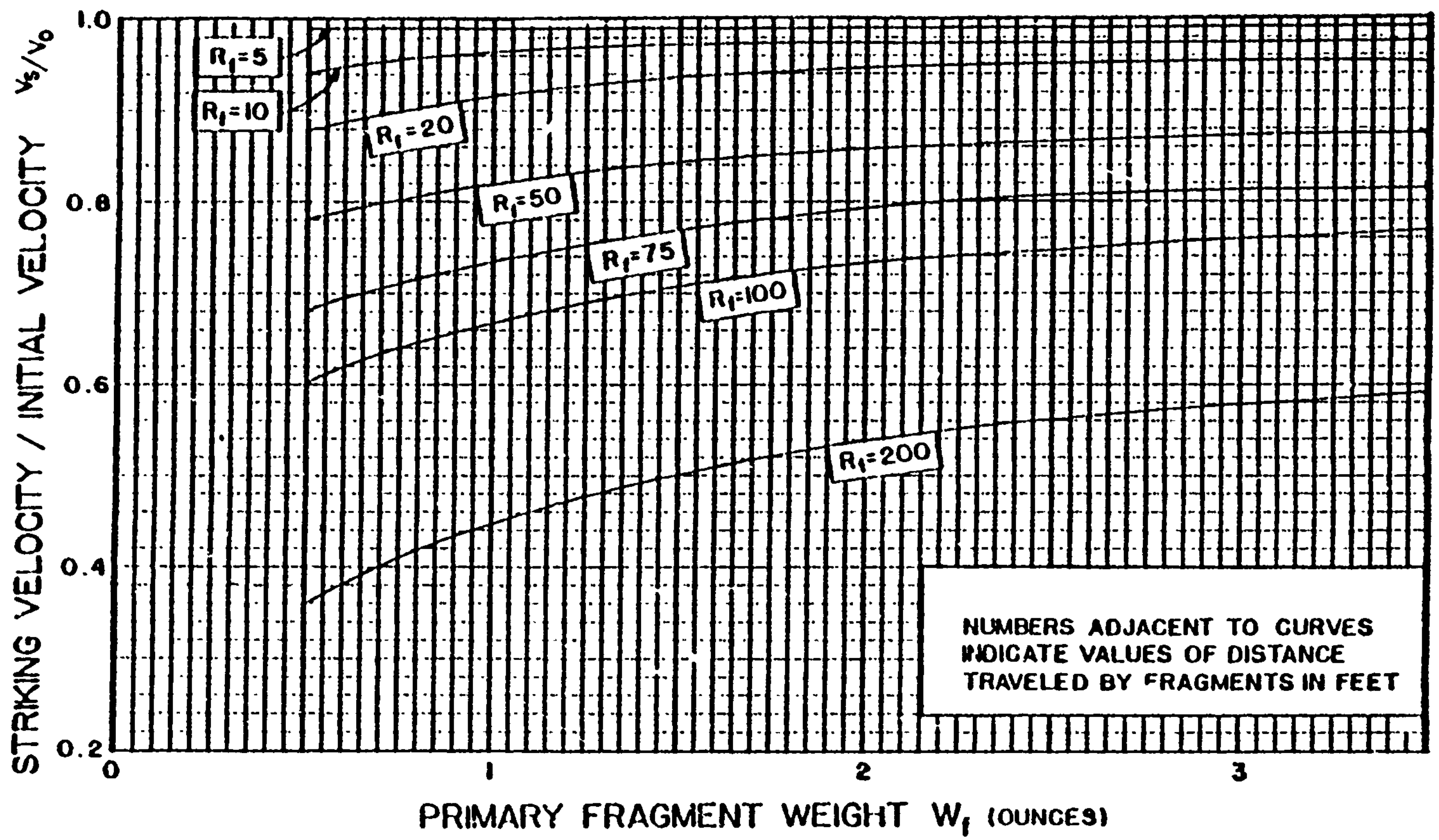


Figure 6-4. Initial Velocity of Primary Fragments vs. Charge Weight (Ref. 1)

$R_f = R_g$

which gives:

$$V_s = (5829)e^{(-0.004 \times 25/2.12^{1/3})}$$

$$= \underline{5329\ \text{ft/sec}} \quad \text{(cylindrical)}$$

and:

$$V_s = (7391)e^{(-0.004 \times 25/2.04^{1/3})}$$

$$= \underline{6831\ \text{ft/sec}} \quad \text{(non-cylindrical)}$$

The maximum penetration, (XF), of this largest fragment based on it's final impact velocity may now be calculated as follows:

$$XF = 0.162(10^{-5})(W_f)^{0.4}(V_s)^{1.8}$$

which is based on f'_c = 5000 psi and armor-piercing steel casing. Correcting for these gives:

$$XF' = XF \times k \times \sqrt{5000}/\sqrt{4000}$$

where:

$$k = 0.70\text{, mild steel}$$

so:

$$XF' = 0.162(10^{-5})(2.12)^{0.4}(5392)^{1.8} \times 0.7 \times \sqrt{5000}/\sqrt{4000}$$

$$= \underline{8.92'', \text{ Say } 9''} \qquad \text{(cylindrical)}$$

and:

$$XF' = 0.162(10^{-5})(2.04)^{0.4}(6831)^{1.8} \times 0.7 \times \sqrt{5000}/\sqrt{4000}$$

$$= \underline{13.5''} \qquad \text{(non-cylindrical)}$$

This indicates then that a reinforced concrete arch of total thickness 13½" or less will be fully penetrated by the primary fragments coming from the detonation of this weapon. The fragments will be embedded in concrete shells that are any thicker.

6.3. Cases 1A - 1D Summarized

The final designs for the four configurations of Case 1 selected for comparison have been summarized and tabulated on the following page. These figures are those obtained from the prinouts in APPENDIX A. As discussed in Section 6.2.2, only those using the minimum allowable amounts of steel are being compared. The individual configurations and load functions used are also included in this section as a reminder and a reference.

Parameter	1A	1B	1C	1D
d, in	15	15	15.5	16
h, in	17.5	17.5	18	18.5
p	0.005685	0.006787	0.006323	0.006504
p_b	0.01137	0.01357	0.01265	0.01301
p_1	0.0025	0.0025	0.0025	0.0025
$R_{mc}(P_u)$, psi	230.9	206.9	182.2	159.0
$R_c(P)$, psi	45.1	39.7	34.4	29.1
$R_{mf}(M_u)$, lbs	579.6	687.7	674.8	715.0
$R_f(M)$, lbs	312.5	331.4	340.2	341.7
P/P_u	0.1954	0.1920	0.1886	0.1830
M/M_u	0.5392	0.4819	0.5042	0.4779
M/M_u(all)	2.544	2.386	2.424	2.364
P_{cr}, psi	45.1	39.7	34.4	29.1
P/P_{cr}	1	1	1	1
V_{act}, lbs	949.1	997.9	1084.6	1178.8
V_{all}, lbs	10285.7	10285.7	10677.8	11070.3
VT_{all}, lbs	1897.4	1897.4	1960.6	2023.8
V_c, cy	1527.2	1603.7	1749.6	1929.2
WT_c, tons	996.5	1046.4	1141.6	1258.8
WT_{st}, tons	39.9	48.6	50.2	56.9
TOTWT, tons	1036.4	1095.0	1191.8	1315.7
$\$_c$, \$	99266	104239	113723	125398
$\$_s$, \$	47894	58285	60194	68257
TOT\$, \$	147159	162524	173918	193655

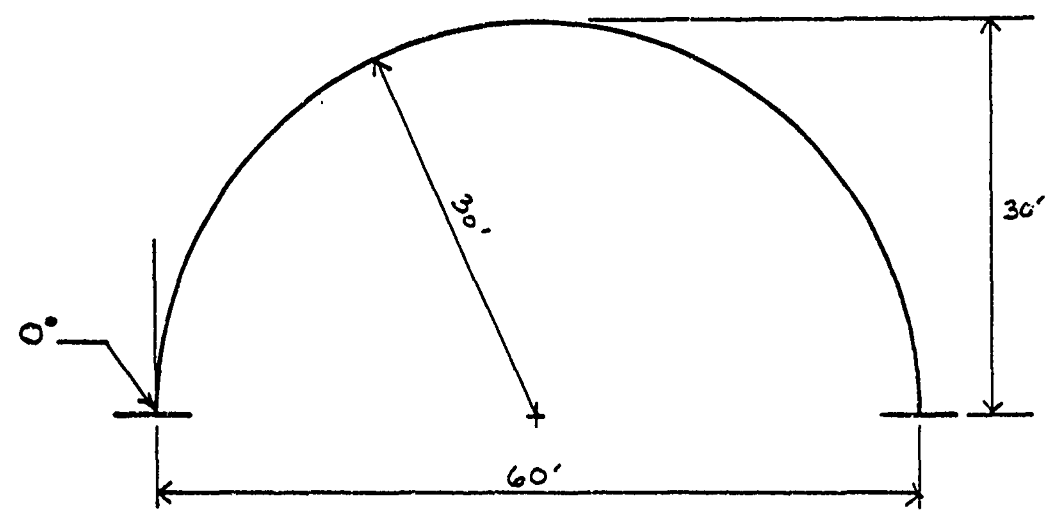


Figure 6-5. Case 1A Arch Configuration

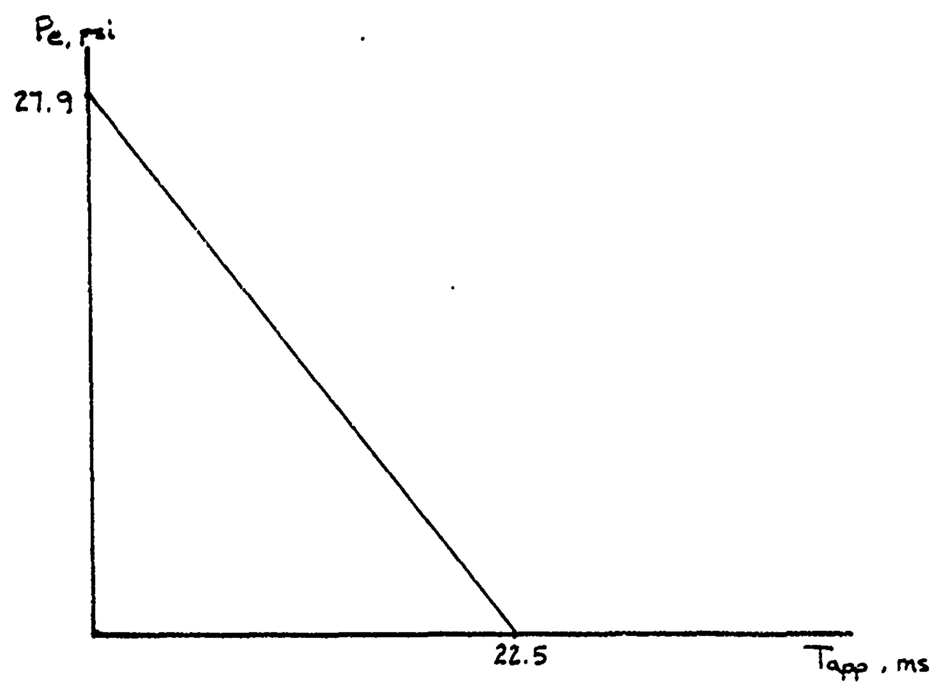


Figure 6-6. Case 1A Approximated Loading Function

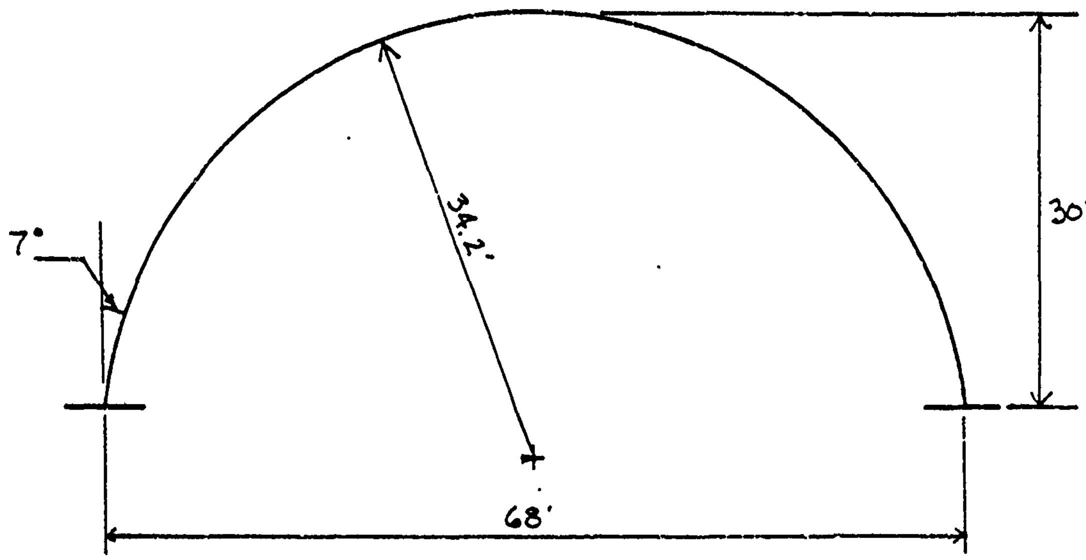


Figure 6-7. Case 1B Arch Configuration

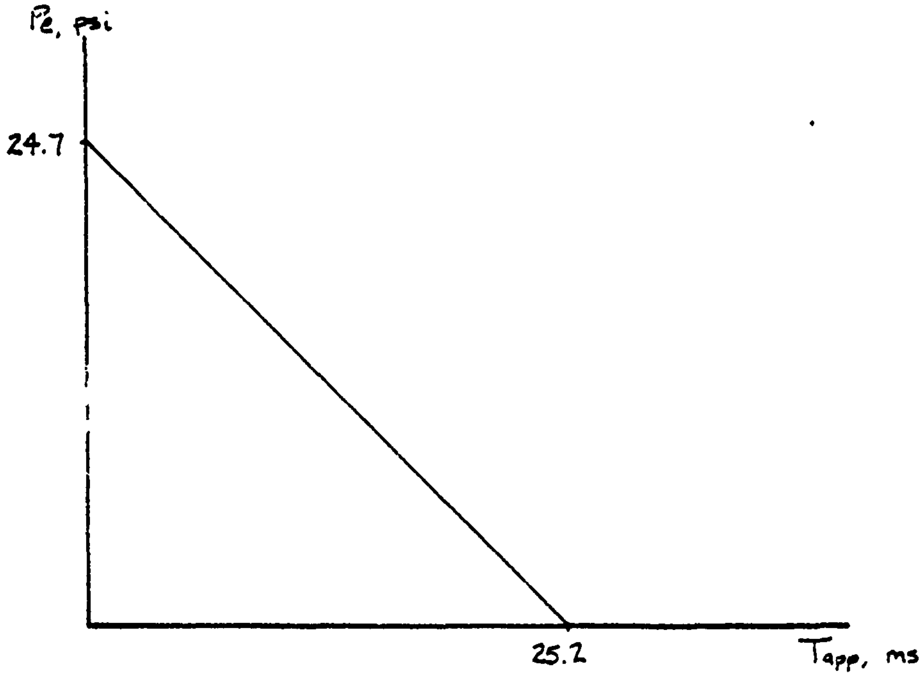


Figure 6-8. Case 1B Approximated Loading Function

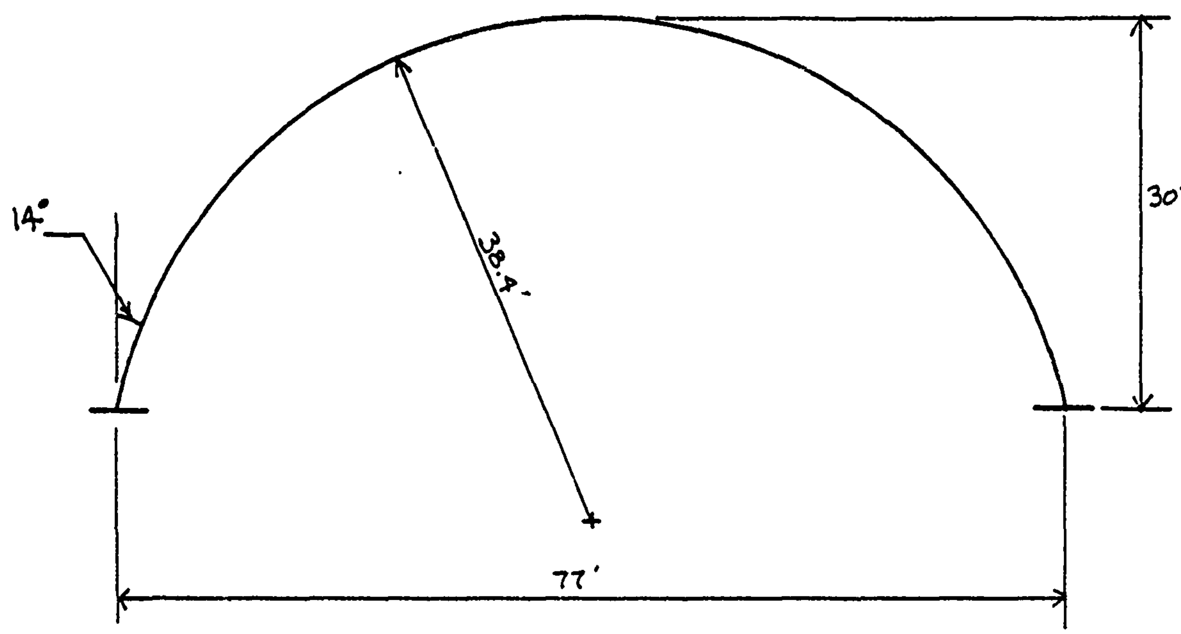


Figure 6-9. Case 1C Arch Configuration

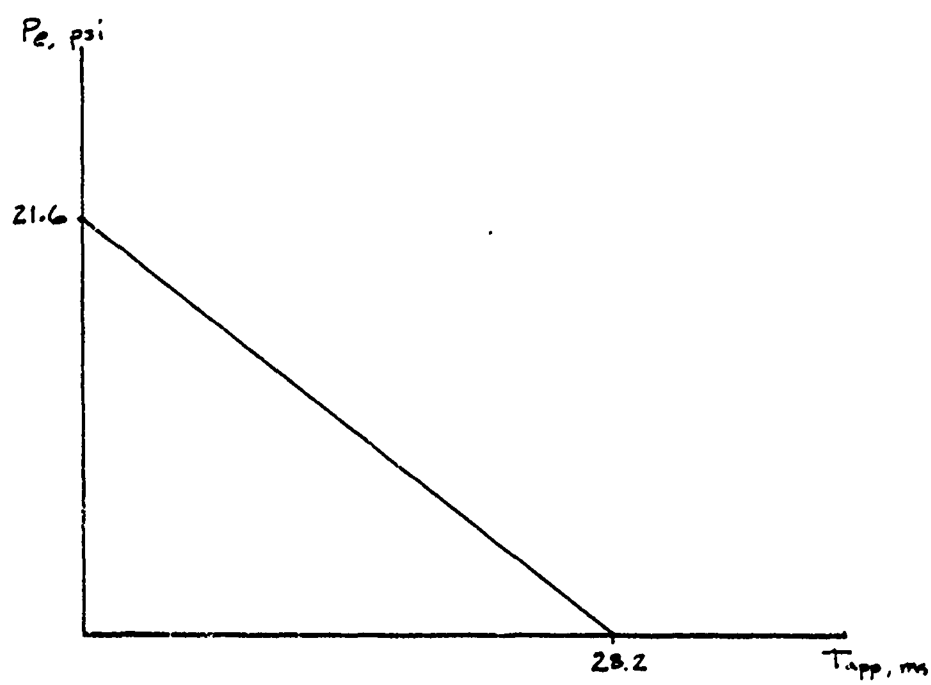


Figure 6-10. Case 1C Approximated Loading Function

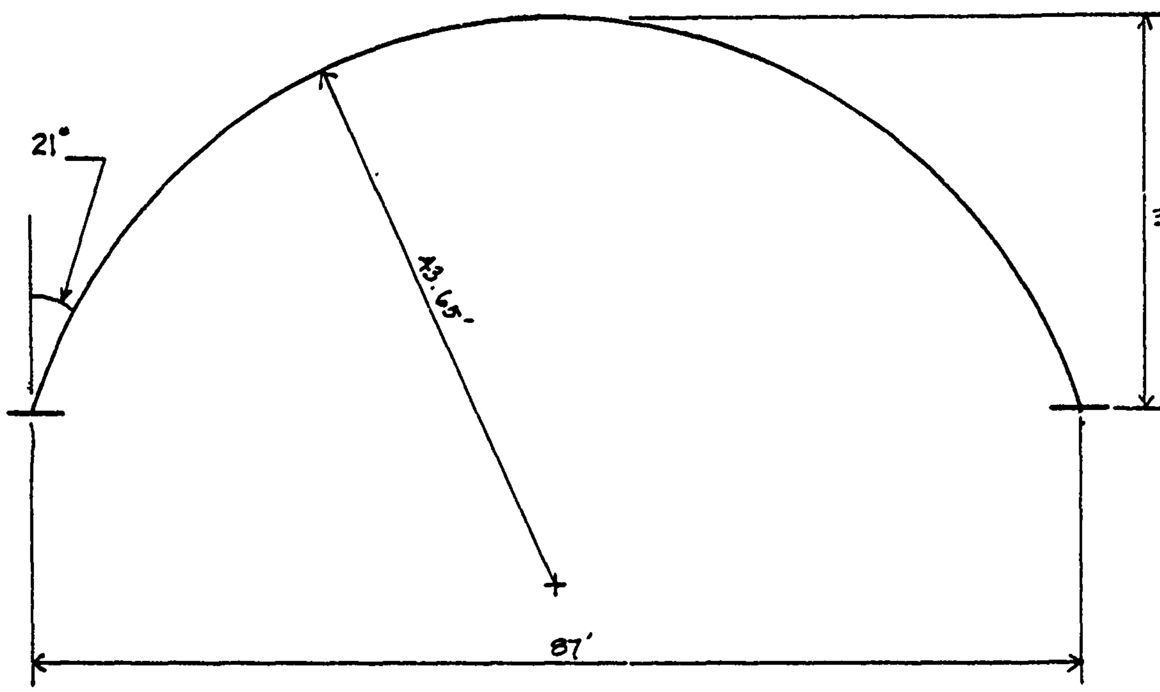


Figure 6-11. Case 1D Arch Configuration

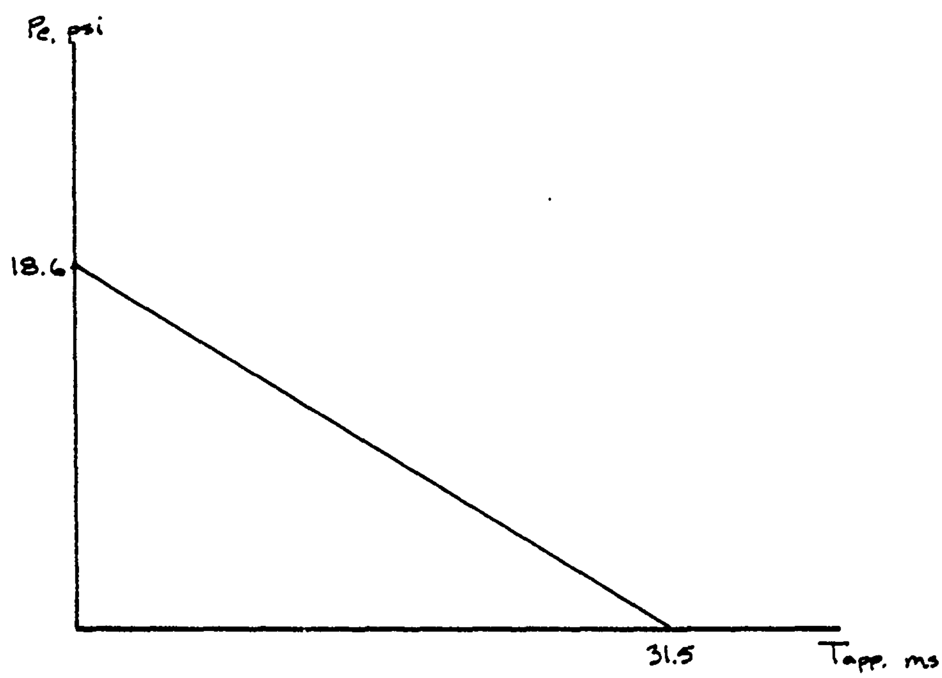


Figure 6-12. Case 1D Approximated Loading Function

6.4. Cases 2A - 2D Summarized

The final designs for the four configurations of Case 2 have been summarized and tabulated on the following page. Again, the four groups of information are as obtained from the LOTUS SE printouts in APPENDIX A. The individual configurations and load functions used are also included in this section as a reminder and a reference.

Parameter	2A	2B	2C	2D
d, in	15	14	13	12.5
h, in	17.5	16.5	15.5	15
p	0.005685	0.005876	0.006777	0.005773
p_t	0.01137	0.01175	0.01355	0.01155
p_l	0.0025	0.0025	0.0025	0.0025
$R_{mc}(P_u)$, psi	230.9	216.9	202.5	185.1
$R_c(P)$, psi	45.1	44.7	43.4	41.3
$R_{mf}(M_u)$, lbs	579.6	589.0	649.5	565.8
$R_f(M)$, lbs	312.5	340.9	364.6	375.6
P/P_u	0.1954	0.2061	0.2144	0.2231
M/M_u	0.5392	0.5788	0.5614	0.6638
M/M_u(all)	2.544	2.591	2.524	2.719
P_{cr}, psi	45.1	44.7	43.4	41.3
P/P_{cr}	1	1	1	1
V_{act}, lbs	949.1	857.0	778.6	727.2
V_{all}, lbs	10285.7	9503.0	8722.6	8333.3
VT_{all}, lbs	1897.4	1770.9	1644.4	1581.1
V_c, cy	1527.2	1337.9	1177.2	1074.9
WT_c, tons	996.5	872.9	768.1	701.4
WT_s, tons	39.9	35.6	34.8	27.6
TOTWT, tons	1036.4	908.5	802.9	729.1
$\$_c$, \$	99266	86961	76516	69872
$\$_s$, \$	47894	42677	41812	33191
TOT\$, \$	147159	129638	118328	103063

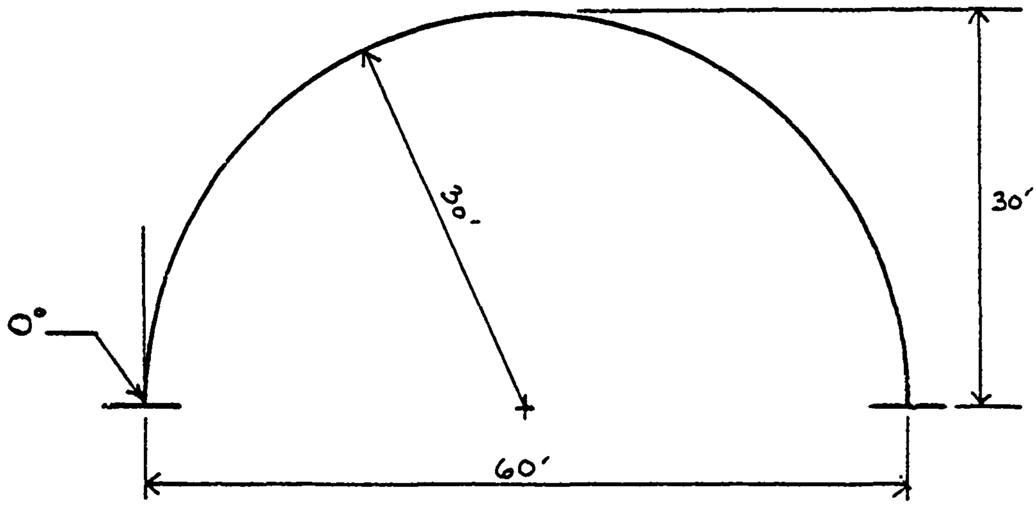


Figure 6-13. Case 2A Arch Configuration

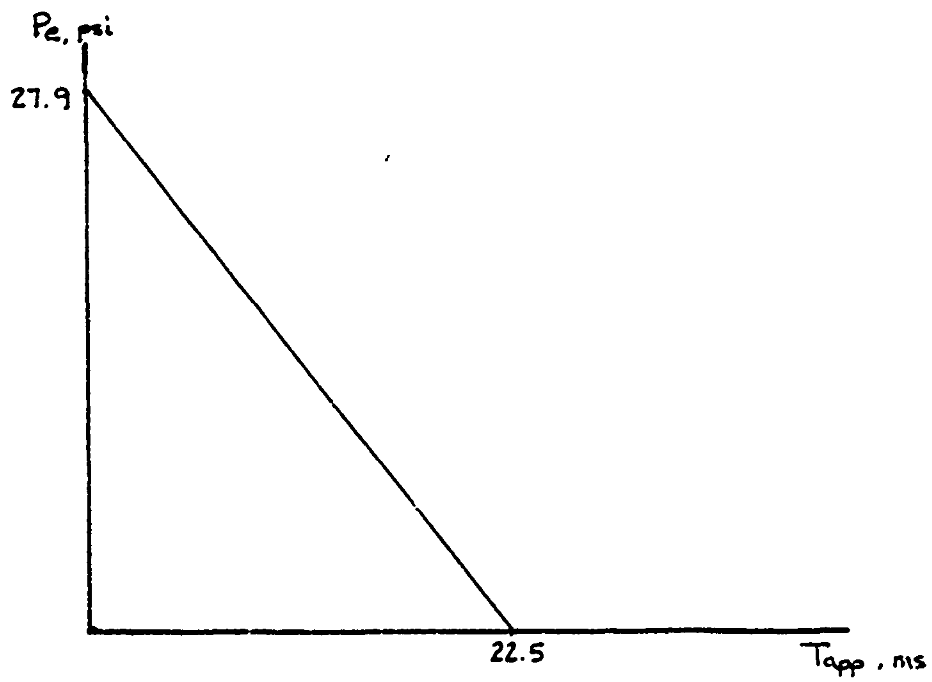


Figure 6-14. Case 2A Approximated Loading Function

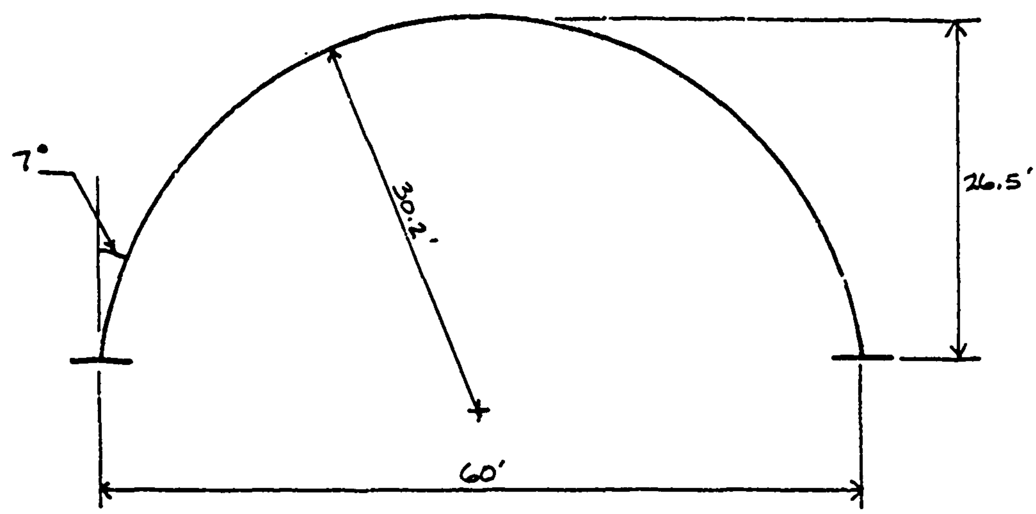


Figure 6-15. Case 2B Arch Configuration

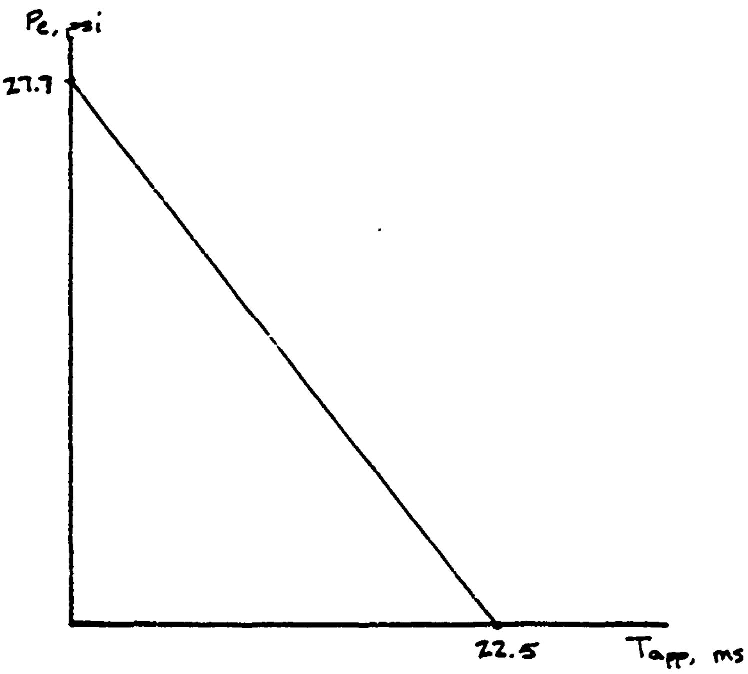


Figure 6-16. Case 2B Approximated Loading Function

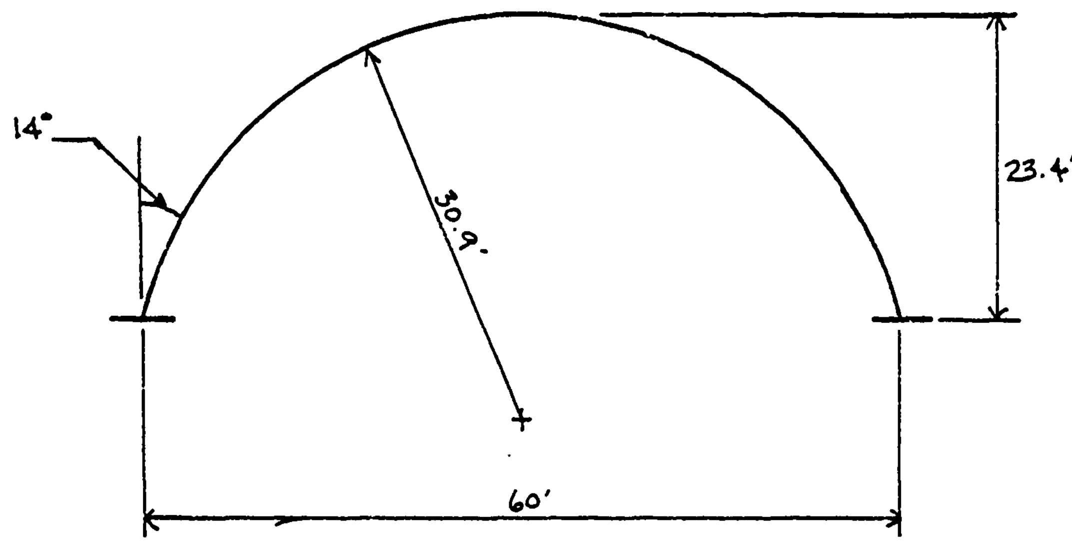


Figure 6-17. Case 2C Arch Configuration

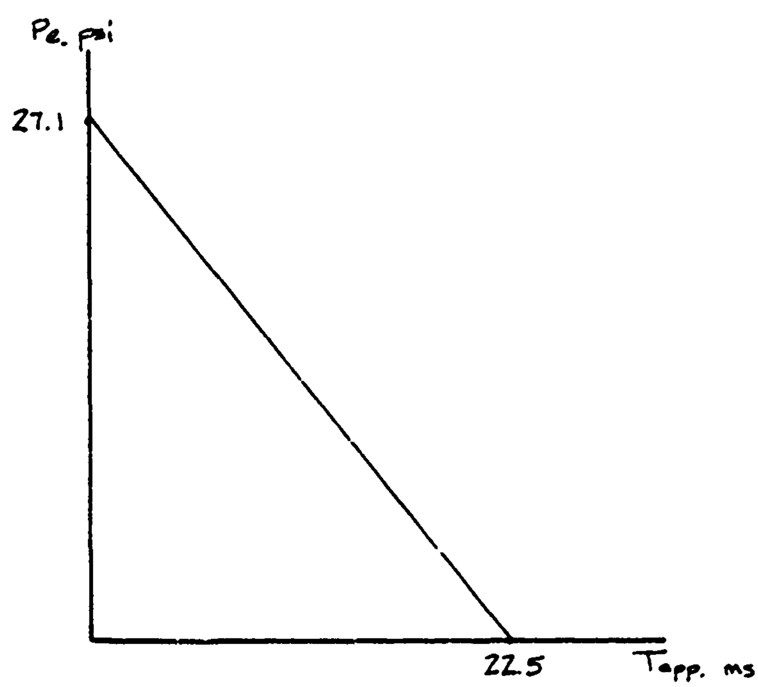


Figure 6-18. Case 2C Approximated Loading Function

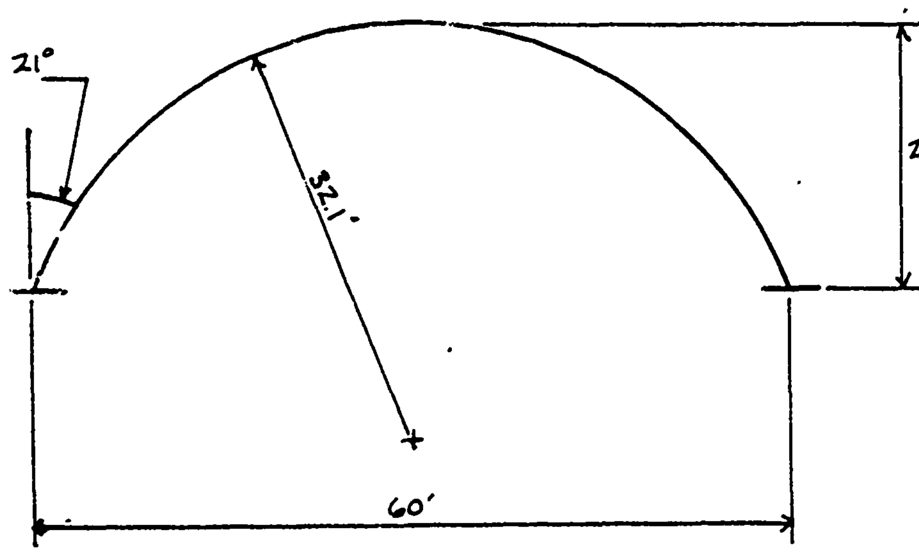


Figure 6-19. Case 2D Arch Configuration

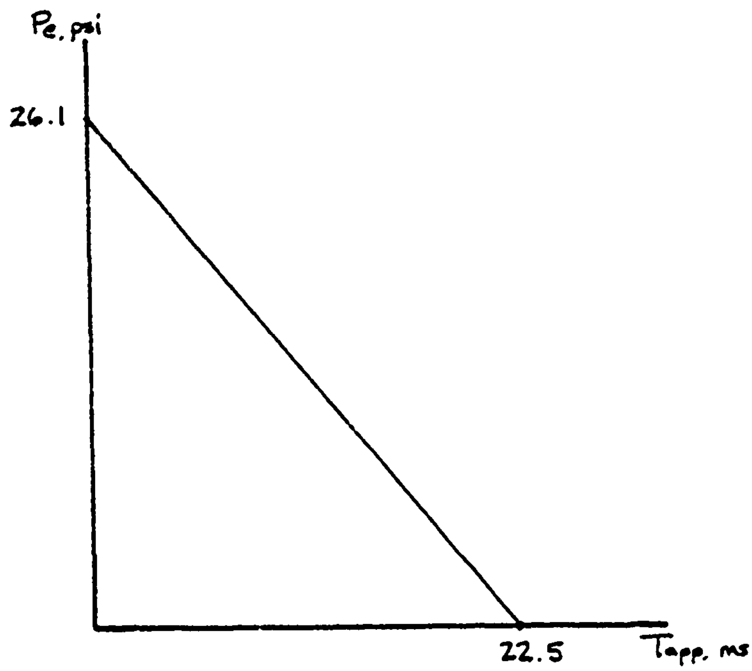


Figure 6-20. Case 2D Approximated Loading Function

CHAPTER SEVEN
FINAL DESIGN COMPARISONS

7.1 Data Analysis Setup

The weight and cost parameters introduced in Section 4.3 were deemed to be the most meaningful for the purpose of comparing these final designs. To repeat, these parameters are as follows; total volume of concrete, (V_c), total weight of concrete, (WT_c), total weight of steel, (WT_s), total structural weight, (TOTWT), estimated cost of concrete, ($\$_c$), estimated cost of steel, ($\$_s$), and total estimated structural cost, (TOT$).

The concrete weight is determined by using the total volume of concrete required. The total volume of concrete required is determined by multiplying the arc length, the concrete thickness, and the overall length of the cylindrical arch facility. Converting these to like units gives the following:

$$V_c = ((h/12)(L)(2L_a/12))/9, \text{ cy} \qquad (7.1)$$

The concrete weight is then determined simply by multiplying Equation 7.1 by the unit weight of concrete, (τ_c = 145 lb/cf). Converting to tons gives the following expression:

$$WT_c = (9V_c \times \tau_c)/2000, \text{ tons} \qquad (7.2)$$

The total weight of the steel reinforcement was calculated using the actual steel ratios as designed rather than the steel that would actually be placed in the structure, (A_s and A_l). This offers a more precise comparison since the actual steel placed for some configurations would be farther from their respective steel ratios than for other configurations.

The weight of the steel reinforcement is calculated using the same basic principles that were used for the concrete calcualations. The volume of steel is first determined and than multiplied by a unit weight of steel, (τ_s = 0.2827 lb/in^3).

The volume of steel is simply the total cross-sectional area of the steel, (A_s or A_l), multiplied by the length of the steel. The length of steel in the primary direction is the arc length, ($2L_a$, since L_a was defined as one half the arc length). The expression, A_s = pbd, would now be used to determine the total weight where the total width, (b), equals the length of the cylindrical arch, (L). Multiplying the final expression by two since the same amount of steel exists in each face and converting units gives the following:

$$WT_s(p) = 2((p(L \times 12)(d) \times 2L_a) \times \tau_s),\ \text{lbs} \quad (7.3)$$

The length of the steel in the longitudinal direction is the overall facilty length, (L). Therefore, the volume of steel in the longitudinal direction would equal, $A_l \times L$. Again, using the basic expression, $A_l = p_l bd$, where b now equals the arc length, $(2L_{m})$, gives the following:

$$WT_{w}(l) = (p_l(2L_{m})(d) \times (L \times 12)) \times \tau_{w}, \text{ lbs} \quad (7.4)$$

The total weight of steel is then found by summing Equations 7.3 and 7.4. Converting this total to tons gives the following:

$$WT_{w} = WT_{w}(p) + WT_{w}(l)/2000, \text{ tons} \quad (7.5)$$

and the total structural weight becomes:

$$TOTWT = WT_{c} + WT_{w}, \text{ tons} \quad (7.6)$$

Converting these into total estimated cost figures is then accomplished by applying an estimated unit cost. The following unit costs will be used:

Concrete: \$65/cy (in-place)

Steel: \$1200/ton (in-place)

and, therefore:

Concrete Cost: $\$_{c} = V_{c} \times \$65$

Steel Cost: $\$_{s} = WT_{s} \times \$1200$

TOTAL COST: $TOT\$ = \$_{c} + \$_{s}$

7.2 Data Analysis

7.2.1 Case 1

The data presented below is that to be compared for the final four designs of Case 1. The appropriate weight and cost data has been drawn from the summary in Section 6-3 and is shown below. Figure 7.1 shows how the total structural weight relates to the angles of incidence, (α), for the respective configurations.

Case	V	WT	WT	TOTWT	$	$	TOT$
1A	1527	110.7	39.9	150.6	99265	47893	147159
1B	1604	116.3	48.6	164.8	104239	58285	162524
1C	1750	126.8	50.2	177.0	113723	60[illegible]	173918
1D	1929	139.9	56.9	196.7	125398	68257	193655

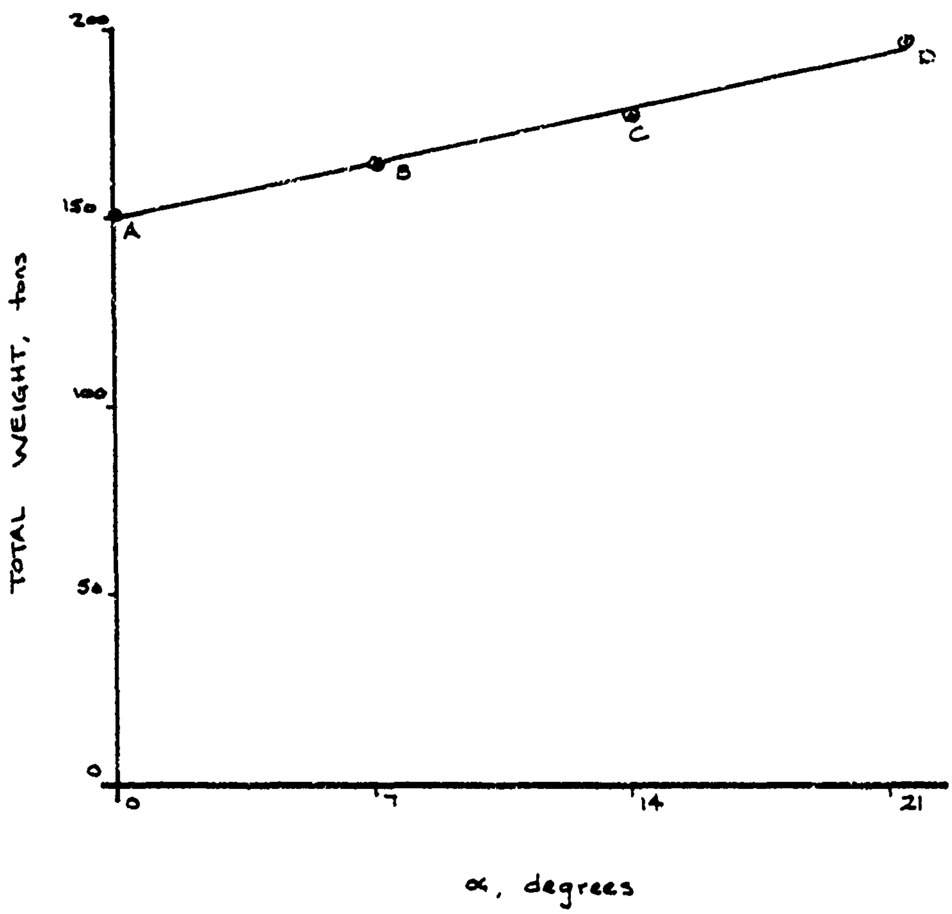


Figure 7-1. Case 1 - Total Structural Weight vs. Angle of Incidence

7.2.2 Case 2

The data presented below is that to be compared for the final four designs of Case 2. It has been compiled in the same manner as that for Case 1, (Section 7.2.1). It is also followed by a similar plot of weight vs. angle of incidence, (Figure 7.2).

Case	V	WT	WT	TOTWT	$	$	TOT$
2A	1527	110.7	39.9	150.6	99265	47893	147159
2B	1338	97.0	35.6	132.6	86961	42677	129638
2C	1177	85.3	34.8	120.2	76516	41811	118328
2D	1075	77.9	27.6	105.6	69871	33191	103063

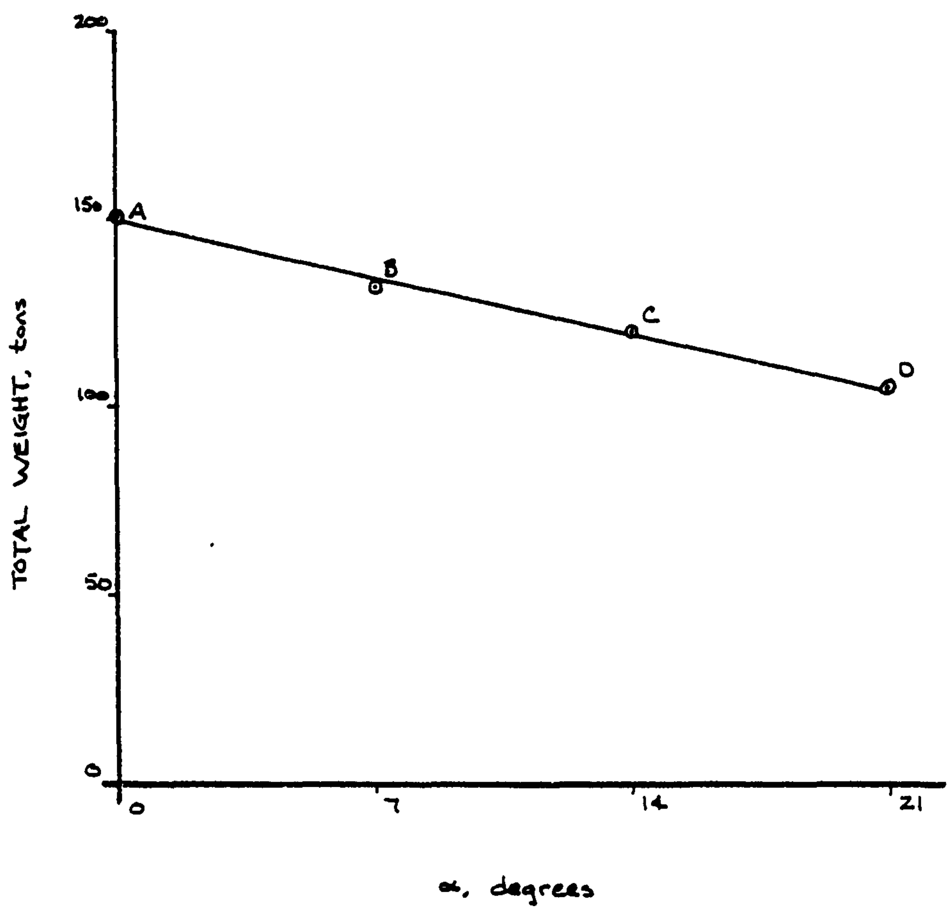


Figure 7-2. Case 2 - Total Structural Weight vs. Angle of Incidence

7.3 Interpretation

From the plots for the two case, (Figures 7-1 and 7-2), it appears that there is very nearly a linear relationship between the total structural weight, (TOTWT), and the angle of incidence, (α). These relationships would also hold for the total estimated cost, (TOT$), since these costs are determined by multiplying the weight by a constant. It is also clear that there is an inverse relationship between the two cases examined. In Case 1, the structural weight and estimated cost are increasing as the angle of incidence increases. For Case 2, the weight and cost are decreasing as the angle of incidence is increasing. The relationship between the structural weight and the angle of incidence can be approximated by the following expressions for the two cases:

CASE 1: $TOTWT = 2.150(\alpha) + 149.70$

(correlation coefficient = 0.9947)

CASE 2: $TOTWT = -2.106(\alpha) + 149.36$

(correlation coefficient = 0.9973)

From these expressions, it can be seen that the two relationships evaluated are nearly the exact inverse of one another. The slope constant for Case 1, (2.150), is nearly the opposite of the slope constant for Case 2, (-2.106). The correlation coefficients indicate that these expressions are quite reliable since they are so close to one, (1), in each case.

CHAPTER EIGHT
CONCLUSION AND RECOMMENDATIONS

8.1 General Statement

In the development of any structural system, the designer is faced with making numerous assumptions, approximations, and estimations, based on personal judgements and experience. For the method presented in this report, a critical approximation is made initially for the loading function which ultimately governs the whole design. With this in mind, the level of precision or accuracy when designing for protective structures has been estimated at about 25% at best, (Reference 1).

8.2 Conclusions

It appears that the reduction in equivalent load as a function of the reflected pressure and angle of incidence, is not enough to have a real impact on the final designs. The original thought that the section thickness could be reduced enough, because of this load reduction, to overcome the increases in arc length and facility size did not hold true. This was because other criteria, such as buckling, played a much larger role. A look at the buckling expression used in Section 6.2.1 to select a trial section, makes this much more obvious.

$$P_{cr} = 3EI/r^3$$

or:

$$I = P_{cr}r^3/3E$$

It can be seen, from these expressions, that the radius of the arch has much more impact on the moment of inertia, (I), and, hence, on the final design requirements, than does the estimated loading condition, (P_{cr}). This is why the smallest structures, (most closely fitting the minimum dimension requirements and having the smallest arc lengths), turned out to be the most economical in each case, regardless of the angle of incidence and loading conditions. It must also be pointed out again that the static critical buckling load is being used and is governing in all of the cases examined. This is probably overly conservative for an applied dynamic load, however, any refinements or reductions of the sections to account for this could be applied to all cases and so the comparisons would not change significantly from those already examined.

It was also interesting to note, (Section 5.4), that the reduction in the reflected pressure alone, due to the increase in the angle of incidence, did not produce significant reductions in the overall equivalent loads, (P_{cr}). In Case 2, where the change in the reflected pressure is the only variable, the equivalent pressure reduction was only 1.8 psi over an increase of 21° in the angle of incidence. This is approximately a 6.5% reduction. In Case 1, the 'spreading' of the arch introduces another variable into the load equation through the wave transit time, (T_{arp}), which is dependent on the arch bay width, (D). When this variable is taken in conjunction with the same reflected pressure reductions, the resulting reduction in the equivalent pressure is 9.3 psi over the same increase in angle of incidence. This is approximately a 33% reduction. Again, geometric characteristics, (arch bay width), are having much more impact on the final design than are changes in the angle of incidence. Even so, in Case 1, where the load reduction does seem significant, it was not enough to overcome the increased sizes of the arches. This is still largely due to the fact that while this increase in arch bay width results in greater load reductions, it is also creating larger arc lengths, (slenderness ratios), which brings stability and buckling back into play as the governing criteria as previously discussed.

8.3 Recommendations

Reviewing the results as tabulated in APPENDIX A, reveals that more concrete and less steel yields a more economical structural design, hence, only those designs that used the minimum allowable amounts of steel were used in the comparison studies in the prior chapters. As pointed out in Section 6.2.2, these economical designs would probably not be the ones actually chosen to construct the actual structures.

More steel would be required to insure that a 'brittle' or sudden failure does not occur at the time when the concrete, (at its' extreme compression fiber), begins to crush. One half of the maximum allowable steel ratio, as presented in References 5 and 6, is generally used as the basis to assure that a 'ductile' failure occurs at that point. The alternate design printouts in APPENDIX A, as well as the long-hand iteration of Section 6.2, show steel ratios closer to this amount. It is recommended that designs such as these be used for actual application. Again, for the comparitive nature of this report, the minimum allowable steel ratio was chosen. Similar expressions and relationships to those of Section 7.3, which were the primary purpose of this report, would most likely result for any steel ratio as long as the same basis was used for the steel ratio in all cases.

If buckling had not been critical, which could be the case with other loading conditions, (nuclear), and structural configurations, these designs would have been based on strength criteria. It was established in Section 3.4.6 that the allowable bending capacity cannot be increased for increased axial loads under these short duration loading conditions. Combine these extremely short load durations with the critical buckling nature of these structures, and the actual bending capacity comes nowhere close to that allowed on the interaction diagram, (Figure 3-9). The designer then must make a decision as to what values will be used as the allowable in his/her design. Using the interaction equation for steel in combined loading, (P/P_u + $M/M_u \leq 1$), would probably be too conservative even though all cases in this report meet that limitation, as well.

Based on the results shown in APPENDIX A, and other iterations that were performed and not included in this report, it was determined that these structures are much more sensitive to the bending or flexural response than they are to the axial or compressive mode. Variances in the steel ratios and concrete thicknesses, as well as in the structural configurations themselves, result in greater changes in bending capacities than in axial or compressive capacities. It follows then, that if these designs were based on strength and not on stability, the designer should probably use some allowable bending capacity, less than that allowed for on the interaction diagram, as the governing criteria for this design process.

Based on the simplified method used, it appears that smaller is better regardless of the angle of incidence and applied loads and that a designer should attempt to fit the dimensional requirements as closely as possible. This is not to say that more precise methods or different types of loads, (say from a nuclear explosion at a greater distance), would produce the same results and conclusions. As pointed out earlier in the report, it is recommended that some more rigorous or in-depth method, such as a non-linear analysis, (Reference 4), be used for the final designs.

It may be seen from these results that a correlation between the cost or weight of an arch structure and it's respective angle of incidence cannot be made across the board. As shown in CHAPTER SEVEN, Case 1 exhibits an increase in total weight and estimated cost as the angle of incidence increases, whereas, Case 2 exhibits the opposite behavior. It is strongly recommended that when using the method presented here, each evaluation be made individually on a case-by-case basis.

APPENDIX A
LOTUS 1-2-3 SE PRINTOUTS

Notation Used in LOTUS

The following notation is that used in the ensuing printouts that differs from that used in the body of this report. The printout notation appears first followed by its' corresponding notation or symbol used in the main body.

wc = τ_c

B = β radians

B^ = β degrees

vf = μ_f

v = μ

CASE 1A

MATERIAL PROPERTIES		DIMENSIONS		ACTUAL CAPACITY	
f'c=	4000	r=	360	Rmc(Pu)=	230.91638
f'dc=	5000	B^=	90		
wc=	145	B=	1.5707963	a=	0.8828470
Ec=	3640000	D=	60	M=	54625.234
fy=	40000	H=	30	Rmf(Mu)=	579.59174
fdy=	44000	n=	2		
		La=	565.48667	**REQUIRED CAPACITY**	
LOADING CONDITIONS		vf=	0.75		
		v=	1.8333333	Tnc=	0.0174803
Pe=	27.9	b=	1	To/Tnc=	1.2871594
Tapp=	0.0225	d=	15	Pm/Rm=	0.6182000
		h=	17.5	Rc(P)=	45.131019
STRUCT'L PROPERTIES		Ac=	17.5		
				Tnf=	0.8187249
Ia=	192.82640	p=	0.005685	Pm/Rm=	50.487333
K=	298.10000	pt=	0.01137	Rf(M)=	312.49577
m=	0.0038043	pl=	0.0025		
Mt=	2.1512822	As=	0.085275		

d	h	p	M/Mu	P/Pu	M/Mu+P/Pu
15	17.5	0.005685	0.5391653	0.1954431	0.7346084

pfdy/fdc= 0.050028

M/Mu(all)

2.5442116

SHEAR CHECK		CRITICAL BUCKLING			
Vact=	949.14332	Pcr=	45.131694	Vc=	1527.1630
				WTc=	996.47391
Vall=	10285.714	P/Pcr=	0.9999850	WTst=	39.911417
				TOTWT=	1036.3853
VTall=	1897.3665	0.1M=	1.8348099		
				$c=	99265.601
		0.05M=	2.5446091	$st=	47893.700
				TOT$=	147159.30

CASE 1A

MATERIAL PROPERTIES	
f'c=	4000
f'dc=	5000
wc=	145
Ec=	3640000
fy=	40000
fdy=	44000

LOADING CONDITIONS	
Pe=	27.9
Tapp=	0.0225

STRUCT'L PROPERTIES	
Ia=	192.82476
K=	298.09747
m=	0.0036956
Mt=	2.0898170

DIMENSIONS	
r=	360
B^=	90
B=	1.5707963
D=	60
H=	30
n=	2
La=	565.48667
vf=	0.75
v=	1.8333333
b=	1
d=	14.5
h=	17
Ac=	17
p=	0.007909
pt=	0.015818
p1=	0.0025
As=	0.1146805

ACTUAL CAPACITY	
Rmc(Pu)=	223.56073
a=	1.1872804
M=	70170.684
Rmf(Mu)=	744.53408

REQUIRED CAPACITY	
Tnc=	0.0174803
To/Tnc=	1.2871594
Pm/Rm=	0.6182000
Rc(P)=	45.131019
Tnf=	0.8069475
Pm/Rm=	49.761070
Rf(M)=	317.05664

d	h	p	M/Mu	P/Pu	M/Mu+P/Pu
14.5	17	0.007909	0.4258457	0.1932303	0.6190760

pfdy/fdc= 0.0695992

M/Mu(all)
2.2542446

SHEAR CHECK	
Vact=	927.15088
Vall=	9894.1176
VTall=	1834.1210

CRITICAL BUCKLING	
Pcr=	45.131311
P/Pcr=	0.9999935
0.1M=	1.8270452
0.05M=	2.5296573

Vc=	1483.5293
WTc=	968.00323
WTst=	50.953672
TOTWT=	1018.9569
$c=	96429.441
$st=	61144.406
TOT$=	157573.84

CASE 1B

MATERIAL PROPERTIES		DIMENSIONS		ACTUAL CAPACITY	
f'c=	4000	r=	409.92	Rmc(Pu)=	206.93545
f'dc=	5000	B^=	83		
wc=	145	B=	1.4486232	a=	1.0539811
Ec=	3640000	D=	68	M=	64830.687
fy=	40000	H=	30	Rmf(Mu)=	687.69920
fdy=	44000	n=	2.1686746		
		La=	593.81965	**REQUIRED CAPACITY**	
LOADING CONDITIONS		vf=	0.7873765		
		v=	1.6751009	Tnc=	0.0199042
Pe=	24.7	b=	1	To/Tnc=	1.2660585
Tapp=	0.0252	d=	15	Pm/Rm=	0.6216034
		h=	17.5	Rc(P)=	39.735943
STRUCT'L PROPERTIES		Ac=	17.5		
				Tnf=	0.8038574
Ia=	203.05434	p=	0.006787	Pm/Rm=	44.259392
K=	271.08855	pt=	0.013574	Rf(M)=	331.39508
m=	0.0038043	pl=	0.0025		
Mt=	2.2590694	As=	0.101805		

d	h	p	M/Mu	P/Pu	M/Mu+P/Pu
15	17.5	0.006787	0.4818895	0.1920209	0.6739105

pfdy/fdc= 0.0597256

M/Mu(all)

2.3855831

SHEAR CHECK		CRITICAL BUCKLING			
Vact=	997.93092	Pcr=	39.736319	Vc=	1603.6796
				WTc=	1046.4009
Vall=	10285.714	P/Pcr=	0.9999905	WTst=	48.570977
				TOTWT=	1094.9719
VTall=	1897.3665	0.1M=	1.8228015		
				$c=	104239.17
		0.05M=	2.5214856	$st=	58285.173
				TOT$=	162524.34

CASE 1B

MATERIAL PROPERTIES		DIMENSIONS		ACTUAL CAPACITY	
f'c=	4000	r=	409.92	Rmc(Pu)=	209.57012
f'dc=	5000	B^=	83		
wc=	145	B=	1.4486232	a=	1.370424
Ec=	3640000	D=	68	M=	80461.497
fy=	40000	H=	30	Rmf(Mu)=	853.50486
fdy=	44000	n=	2.1686746		
		La=	593.81965	**REQUIRED CAPACITY**	
LOADING CONDITIONS		vf=	0.7873765		
		v=	1.6751009	Tnc=	0.0199042
Pe=	24.7	b=	1	To/Tnc=	1.2660585
Tapp=	0.0252	d=	14.5	Pm/Rm=	0.6216034
		h=	17	Rc(P)=	39.735943
STRUCT'L PROPERTIES		Ac=	17		
				Tnf=	0.7922934
Ia=	203.05290	p=	0.009129	Pm/Rm=	43.622688
K=	271.08663	pt=	0.018258	Rf(M)=	336.23203
m=	0.0036956	pl=	0.0025		
Mt=	2.1945245	As=	0.1323705		

d	h	p	M/Mu	P/Pu	M/Mu+P/Pu
14.5	17	0.009129	0.3939427	0.1896069	0.5835496

pfdy/fdc= 0.0803352

M/Mu(all) = 2.0860365

SHEAR CHECK		CRITICAL BUCKLING			
Vact=	974.85466	Pcr=	39.736038	Vc=	1557.8602
				WTc=	1016.5037
Vall=	9894.1176	P/Pcr=	0.9999976	WTst=	60.633848
				TOTWT=	1077.1376
VTall=	1834.1210	0.1M=	1.8143306		
				$c=	101260.91
		0.05M=	2.5051738	$st=	72760.617
				TOT$=	174021.53

<u>CASE 1C</u>

MATERIAL PROPERTIES		DIMENSIONS		ACTUAL CAPACITY	
f'c=	4000	r=	474.84	Rmc(Pu)=	182.19954
f'dc=	5000	B^=	76		
wc=	145	B=	1.3264502	a=	1.0146555
Ec=	3640000	D=	77	M=	64652.690
fy=	40000	H=	30	Rmf(Mu)=	674.78687
fdy=	44000	n=	2.3684210		
		La=	629.85162	REQUIRED CAPACITY	
LOADING CONDITIONS		vf=	0.8217283		
		v=	1.5423677	Tnc=	0.0230565
Pe=	21.6	b=	1	To/Tnc=	1.2230780
Tapp=	0.0282	d=	15.5	Pm/Rm=	0.6285358
		h=	18	Rc(P)=	34.365583
STRUCT'L PROPERTIES		Ac=	18		
				Tnf=	0.8126525
Ia=	219.29248	p=	0.006323	Pm/Rm=	39.983676
K=	245.34181	pt=	0.012646	Rf(M)=	340.25873
m=	0.0039129	pl=	0.0025		
Mt=	2.4646072	As=	0.0980065		

d	h	p	M/Mu	P/Pu	M/Mu+P/Pu
15.5	18	0.006323	0.5042462	0.1886150	0.6928613

pfdy/fdc= 0.0556424

M/Mu(all)

2.4208754

SHEAR CHECK		CRITICAL BUCKLING			
Vact=	1080.6367	Pcr=	34.366029	Vc=	1749.5878
				WTc=	1141.6060
Vall=	10677.777	P/Pcr=	0.9999870	WTst=	50.162013
				TOTWT=	1191.7680
VTall=	1960.6121	0.1M=	1.8108503		
				$c=	113723.21
		0.05M=	2.4984721	$st=	60194.416
				TOT$=	173917.62

CASE 1C

MATERIAL PROPERTIES		DIMENSIONS		ACTUAL CAPACITY	
f'c=	4000	r=	474.84	Rmc(Pu)=	187.03293
f'dc=	5000	B^=	76		
wc=	145	B=	1.3264502	a=	1.6618023
Ec=	3640000	D=	77	M=	96540.197
fy=	40000	H=	30	Rmf(Mu)=	1007.6001
fdy=	44000	n=	2.3684210		
		La=	629.85162	**REQUIRED CAPACITY**	
LOADING CONDITIONS		vf=	0.8217283		
		v=	1.5423677	Tnc=	0.0230565
Pe=	21.6	b=	1	To/Tnc=	1.2230780
Tapp=	0.0282	d=	14.5	Pm/Rm=	0.6285358
		h=	17	Rc(P)=	34.365583
STRUCT'L PROPERTIES		Ac=	17		
				Tnf=	0.7896965
Ia=	219.32570	p=	0.01107	Pm/Rm=	38.854206
K=	245.37898	pt=	0.02214	Rf(M)=	350.14986
m=	0.0036956	pl=	0.0025		
Mt=	2.3276846	As=	0.160515		

d	h	p	M/Mu	P/Pu	M/Mu+P/Pu
14.5	17	0.01107	0.3475087	0.1837408	0.5312495

pfdy/fdc= 0.097416

M/Mu(all) 1.8284646

SHEAR CHECK		CRITICAL BUCKLING			
Vact=	1031.5432	Pcr=	34.371235	Vc=	1652.3885
				WTc=	1078.1835
Vall=	9894.1176	P/Pcr=	0.9998355	WTst=	76.340327
				TOTWT=	1154.5238
VTall=	1834.1210	0.1M=	1.7937465		
				$c=	107405.25
		0.05M=	2.4655366	$st=	91608.392
				TOT$=	199013.64

CASE 1D

MATERIAL PROPERTIES

f'c=	4000
f'dc=	5000
wc=	145
Ec=	3640000
fy=	40000
fdy=	44000

LOADING CONDITIONS

Pe=	18.6
Tapp=	0.0315

STRUCT'L PROPERTIES

Ia=	243.24505
K=	220.37382
m=	0.0040216
Mt=	2.7176335

DIMENSIONS

r=	561.12
B^=	69
B=	1.2042771
D=	87
H=	30
n=	2.6086956
La=	675.74401
vf=	0.8530555
v=	1.4306414
b=	1
d=	16
h=	18.5
Ac=	18.5
p=	0.006504
pt=	0.013008
pl=	0.0025
As=	0.104064

ACTUAL CAPACITY

Rmc(Pu)=	158.99185
a=	1.0773684
M=	70794.520
Rmf(Mu)=	714.96492

REQUIRED CAPACITY

Tnc=	0.0272460
To/Tnc=	1.1561312
Pm/Rm=	0.6393336
Rc(P)=	29.092789
Tnf=	0.8351705
Pm/Rm=	36.786758
Rf(M)=	341.66746

d	h	p	M/Mu	P/Pu	M/Mu+P/Pu
16	18.5	0.006504	0.4778800	0.1829828	0.6608629

pfdy/fdc= 0.0572352

M/Mu(all)

2.3635608

SHEAR CHECK

Vact=	1178.8401
Vall=	11070.270
VTall=	2023.8577

CRITICAL BUCKLING

Pcr=	29.093924
P/Pcr=	0.9999610
0.1M=	1.7910869
0.05M=	2.4604153

Vc=	1929.2074
WTc=	1258.8078
WTst=	56.880713
TOTWT=	1315.6885
$c=	125398.48
$st=	68256.856
TOT$=	193655.34

CASE 1D

MATERIAL PROPERTIES

f'c=	4000
f'dc=	5000
wc=	145
Ec=	3640000
fy=	40000
fdy=	44000

LOADING CONDITIONS

Pe=	18.6
Tapp=	0.0315

STRUCT'L PROPERTIES

Ia=	243.38697
K=	220.50239
m=	0.0036956
Mt=	2.4972848

DIMENSIONS

r=	561.12
B°=	69
B=	1.2042771
D=	87
H=	30
n=	2.6086956
La=	675.74401
vf=	0.8530555
v=	1.4306414
b=	1
d=	14.5
h=	17
Ac=	17
p=	0.01394
pt=	0.02788
pl=	0.0025
As=	0.20213

ACTUAL CAPACITY

Rmc(Pu)=	165.92571
a=	2.09264
M=	119653.26
Rmf(Mu)=	1208.3970

REQUIRED CAPACITY

Tnc=	0.0272460
Tc/Tnc=	1.1561312
Pm/Rm=	0.6393336
Rc(P)=	29.092789
Tnf=	0.8003631
Pm/Rm=	35.253599
Rf(M)=	356.52639

d	h	p	M/Mu	P/Pu	M/Mu+P/Pu
14.5	17	0.01394	0.2950407	0.1753362	0.4703770

pfdy/fdc= 0.122672

M/Mu(all)

1.4720163

SHEAR CHECK

Vact=	1099.4319
Vall=	9894.1176
VTall=	1834.1210

CRITICAL BUCKLING

Pcr=	29.110899
P/Pcr=	0.9993779
0.1M=	1.7642548
0.05M=	2.4087468

Vc=	1772.7852
WTc=	1156.7423
WTst=	100.98224
TOTWT=	1257.7246
$c=	115231.03
$st=	121178.69
TOT$=	236409.73

CASE 2A

MATERIAL PROPERTIES

f c=	4000
f'dc=	5000
wc=	145
Ec=	3640000
fy=	40000
fdy=	44000

LOADING CONDITIONS

Pe=	27.9
Tapp=	0.0225

STRUCT'L PROPERTIES

Ia=	192.82640
K=	298.10000
m=	0.0038043
Mt=	2.1512822

DIMENSIONS

r=	360
B^=	90
B=	1.5707963
D=	60
H=	30
n=	2
La=	565.48667
\f=	0.75
v=	1.8333333
b=	1
d=	15
h=	17.5
Ac=	17.5
p=	0.005685
pt=	0.01137
pl=	0.0025
As=	0.085275

ACTUAL CAPACITY

Rmc(Pu)=	230.91638
a=	0.8828470
M=	54625.234
Rmf(Mu)=	579.59174

REQUIRED CAPACITY

Tnc=	0.0174802
To/Tnc=	1.2871594
Pm/Rm=	0.6182000
Rc(P)=	45.131019
Tnf=	0.8187249
Pm/Rm=	50.487333
Rf(M)=	312.49577

d	h	p	M/Mu	P/Pu	M/Mu+P/Pu
15	17.5	0.005685	0.5391653	0.1954431	0.7346084

pfdy/fdc= 0.050028

M/Mu(all)
2.5442116

SHEAR CHECK

Vact=	949.14332
Vall=	10285.714
VTall=	1897.3665

CRITICAL BUCKLING

Pcr=	45.131694
P/Pcr=	0.9999850
0.1M=	1.8348099
0.05M=	2.5446091

Vc=	1527.1630
WTc=	996.47391
WTst=	39.911417
TOTWT=	1036.3853
$c=	99265.601
$st=	47893.700
TOT$=	147159.30

CASE 2A

MATERIAL	PROPERTIES	DIMENSIONS		ACTUAL CAPACITY	
f'c=	4000	r=	360	Rmc(Pu)=	241.41511
f'dc=	5000	B^=	90		
wc=	145	B=	1.5707963	a=	1.87704
Ec=	3640000	D=	60	M=	100208.20
fy=	40000	H=	30	Rmf(Mu)=	1063.2420
fdy=	44000	n=	2		
		La=	565.48667	REQUIRED	CAPACITY
LOADING	CONDITIONS	vf=	0.75		
		v=	1.8333333	Tnc=	0.0174803
Pe=	27.9	b=	1	To/Tnc=	1.2871594
Tapp=	0.0225	d=	13.5	Pm/Rm=	0.6182000
		h=	16	Rc(P)=	45.131019
STRUCT'L	PROPERTIES	Ac=	16		
				Tnf=	0.7825526
Ia=	192.97336	p=	0.01343	Pm/Rm=	48.256739
K=	298.32718	pt=	0.02686	Rf(M)=	326.94041
m=	0.0034782	pl=	0.0025		
Mt=	1.9668865	As=	0.181305		

d	h	p	M/Mu	P/Pu	M/Mu+P/Pu
13.5	16	0.01343	0.3074938	0.1869436	0.4944375

pfdy/fdc= 0.118184

M/Mu(all)

1.5568853

SHEAR CHECK		CRITICAL BUCKLING			
Vact=	883.45558	Pcr=	45.166090	Vc=	1396.2634
				WTc=	911.06186
Vall='	9112.5	P/Pcr=	0.9992235	WTst=	76.035998
				TOTWT=	987.09786
VTall=	1707.6299	0.1M=	1.8049852		
				$c=	90757.121
		0.05M=	2.4871781	$st=	91243.197
				TOT$=	182000.31

CASE 2B

MATERIAL PROPERTIES

f'c=	4000
f'dc=	5000
wc=	145
Ec=	3640000
fy=	40000
fdy=	44000

LOADING CONDITIONS

Pe=	27.7
Tapp=	0.0225

STRUCT'L PROPERTIES

Ia=	158.21629
K=	304.93324
m=	0.0035869
Mt=	1.8846205

DIMENSIONS

r=	362.7
B^=	83
B=	1.4486232
D=	60
H=	26.54
n=	2.1686746
La=	525.41566
vf=	0.7873765
v=	1.6751009
b=	1
d=	14
h=	16.5
Ac=	16.5
p=	0.005876
pt=	0.011752
pl=	0.0025
As=	0.082264

ACTUAL CAPACITY

Rmc(Pu)=	216.86504
a=	0.8516743
M=	49133.256
Rmf(Mu)=	589.04028

REQUIRED CAPACITY

Tnc=	0.0176114
To/Tnc=	1.2775776
Pm/Rm=	0.6197455
Rc(P)=	44.695763
Tnf=	0.6922762
Pm/Rm=	42.689773
Rf(M)=	340.92506

d	h	p	M/Mu	P/Pu	M/Mu+P/Pu
14	16.5	0.005876	0.5787805	0.2060994	0.7848799

pfdy/fdc= 0.0517088

M/Mu(all)

2.5911729

SHEAR CHECK

Vact=	857.01505
Vall=	9503.0303
VTall=	1770.8754

CRITICAL BUCKLING

Pcr=	44.697293
P/Pcr=	0.9999657
0.1M=	1.8722029
0.05M=	2.6166138

Vc=	1337.8639
WTc=	872.95623
WTst=	35.564271
TOTWT=	908.52050
$c=	86961.157
$st=	42677.126
TOT$=	129638.29

CASE 2B

MATERIAL PROPERTIES

f'c=	4000
f'dc=	5000
wc=	145
Ec=	3640000
fy=	40000
fdy=	44000

LOADING CONDITIONS

Pe=	27.7
Tapp=	0.0225

STRUCT'L PROPERTIES

Ia=	158.21625
K=	304.93317
m=	0.0034782
Mt=	1.8275108

DIMENSIONS

r=	362.7
B^=	83
B=	1.4486232
D=	60
H=	26.54
n=	2.1686746
La=	525.41566
vf=	0.7873765
v=	1.6751009
b=	1
d=	13.5
h=	16
Ac=	16
p=	0.008293
pt=	0.016586
pl=	0.0025
As=	0.1119555

ACTUAL CAPACITY

Rmc(Pu)=	219.67616
a=	1.1590687
M=	63646.756
Rmf(Mu)=	763.03721

REQUIRED CAPACITY

Tnc=	0.0176114
To/Tnc=	1.2775776
Pm/Rm=	0.6197455
Rc(P)=	44.695763
Tnf=	0.6817066
Pm/Rm=	42.037988
Rf(M)=	346.21099

d	h	p	M/Mu	P/Pu	M/Mu+P/Pu
13.5	16	0.008293	0.4537275	0.2034620	0.6571895

pfdy/fdc= 0.0729784

M/Mu(all)
2.2606224

SHEAR CHECK

Vact=	836.97935
Vall=	9112.5
VTall=	1707.6299

CRITICAL BUCKLING

Pcr=	44.697283
P/Pcr=	0.9999659
0.1M=	1.8629483
0.05M=	2.5987931

Vc=	1297.3226
WTc=	846.50301
WTst=	45.926014
TOTWT=	892.42902
$c=	84325.970
$st=	55111.217
TOT$=	139437.18

CASE 2C

MATERIAL PROPERTIES		DIMENSIONS		ACTUAL CAPACITY	
f'c=	4000	r=	371.016	Rmc(Pu)=	202.46789
f'dc=	5000	B^=	76		
wc=	145	B=	1.3264502	a=	0.9121044
Ec=	3640000	D=	60	M=	42625.911
fy=	40000	H=	23.44	Rmf(Mu)=	649.53481
fdy=	44000	n=	2.3684210		
		La=	492.13425	**REQUIRED CAPACITY**	
LOADING CONDITIONS		vf=	0.8217283		
		v=	1.5423677	Tnc=	0.0180152
Pe=	27.1	b=	1	Tn/Tnc=	1.2489418
Tapp=	0.0225	d=	13	Pm/Rm=	0.6243642
		h=	15.5	Rc(P)=	43.404153
STRUCT'L PROPERTIES		Ac=	15.5		
				Tnf=	0.5931029
Ia=	132.12043	p=	0.006777	Pm/Rm=	36.57[illegible]21
K=	309.87163	pt=	0.013554	Rf(M)=	364.63538
m=	0.0033695	p1=	0.0025		
Mt=	1.6582585	As=	0.088101		

d	h	p	M/Mu	P/Pu	M/Mu+P/Pu
13	15.5	0.006777	0.5613761	0.2143754	0.775731[illegible]

pfdy/fdc= 0.0596376

M/Mu,all [illegible]

2.5238[illegible]

SHEAR CHECK		CRITICAL BUCKLING			
Vact=	778.64850	Pcr=	43.404983	Vc=	1177.17[illegible]
				WTc=	768.1[illegible]538
Vall=	8722.5806	P/Pcr=	0.9999808	WTst=	34.843128
				TOTWT=	802.74851
VTall=	1644.3843	0.1M=	1.9012435		
				$c=	76516.24[illegible]
		0.05M=	2.6725351	$st=	41811.765
				TOT$=	118328.01

CASE 2C

MATERIAL PROPERTIES		DIMENSIONS		ACTUAL CAPACITY	
f'c=	4000	r=	371.016	Rmc(Pu)=	209.84464
f'dc=	5000	B^=	76		
wc=	145	B=	1.3264502	a=	1.5802729
Ec=	3640000	D=	60	M=	75287.237
fy=	40000	H=	23.44	Rmf(Mu)=	1005.6712
fdy=	44000	n=	2.3684210		
		La=	492.13425	**REQUIRED CAPACITY**	
LOADING CONDITIONS		vf=	0.8217283		
		v=	1.5423677	Tnc=	0.0180152
Pe=	27.1	b=	1	To/Tnc=	1.2489418
Tapp=	0.0225	d=	12	Pm/Rm=	0.5243642
		h=	14.5	Rc(P)=	43.404153
STRUCT'L PROPERTIES		Ac=	14.5		
				Tnf=	0.5736003
Ia=	132.15744	p=	0.01272	Pm/Rm=	35.371527
K=	309.95841	pt=	0.02544	Rf(M)=	377.05011
m=	0.0031521	pl=	0.0025		
Mt=	1.5512741	As=	0.15264		

d	h	p	M/Mu	P/Pu	M/Mu+P/Pu
12	14.5	0.01272	0.3749238	0.2068394	0.5817632

pfdy/fdc= 0.111936

M/Mu(all)

1.6965200

SHEAR CHECK		CRITICAL BUCKLING			
Vact=	742.07087	Pcr=	43.417139	Vc=	1101.2263
				WTc=	718.55019
Vall=	7944.8275	P/Pcr=	0.9997008	WTst=	55.97[illegible]41
				TOTWT=	774.52573
VTall=	1517.8932	0.1M=	1.8747996		
				$c=	7157[illegible].712
		0.05M=	2.6216142	$st=	67170.649
				TOT$=	138750.36

CASE 2D

MATERIAL PROPERTIES		DIMENSIONS		ACTUAL CAPACITY	
f'c=	4000	r=	385.608	Rmc(Pu)=	185.08526
f'dc=	5000	B^=	69		
wc=	145	B=	1.2042771	a=	0.7470941
Ec=	3640000	D=	60	M=	38503.307
fy=	40000	H=	20.62	Rmf(Mu)=	565.83895
fdy=	44000	n=	2.6086956		
		La=	464.37891	**REQUIRED CAPACITY**	
LOADING CONDITIONS		vf=	0.8530555		
		v=	1.4306414	Tnc=	0.0187237
Pe=	26.1	b=	1	To/Tnc=	1.2016799
Tapp=	0.0225	d=	12.5	Pm/Rm=	0.6319870
		n=	15	Rc(P)=	41.298311
STRUCT'L PROPERTIES		Ac=	15		
				Tnf=	0.5222488
Ia=	112.06201	p=	0.005773	Pm/Rm=	32.266562
K=	312.82650	pt=	0.011546	Rf(M)=	375.63002
m=	0.0032608	pl=	0.0025		
Mt=	1.5142609	As=	0.0721625		

d	h	p	M/Mu	P/Pu	M/Mu+P/Pu
12.5	15	0.005773	0.6638461	0.2231312	0.8869774

pfdy/fdc= 0.0508024

M/Mu(all)

2.7188639

SHEAR CHECK		CRITICAL BUCKLING			
Vact=	727.24412	Pcr=	41.299599	Vc=	1074.9511
				WTc=	701.40565
Vall=	8333.3333	P/Pcr=	0.9999688	WTst=	27.659366
				TOTWT=	729.06502
VTall=	1581.1388	0.1M=	1.9319676		
				$c=	69871.827
		0.05M=	2.7316980	$st=	33191.239
				TOT$=	103063.06

CASE 2D

MATERIAL PROPERTIES

f'c=	4000
f'dc=	5000
wc=	145
Ec=	3640000
fy=	40000
fdy=	44000

LOADING CONDITIONS

Ps=	26.1
Tapp=	0.0225

STRUCT'L PROPERTIES

Ia=	112.06123
K=	312.82432
m=	0.0031521
Mt=	1.4637855

DIMENSIONS

r=	385.608
B^=	69
B=	1.2042771
D=	60
H=	20.62
n=	2.6086956
La=	464.37891
vf=	0.8530555
v=	1.4306414
b=	1
d=	12
h=	14.5
Ac=	14.5
p=	0.008491
pt=	0.016982
pl=	0.0025
As=	0.101892

ACTUAL CAPACITY

Rmc(Pu)=	187.90978
a=	1.0548818
M=	51434.327
Rmf(Mu)=	755.97133

REQUIRED CAPACITY

Tnc=	0.0187237
To/Tnc=	1.2016799
Pm/Rm=	0.6319870
Rc(P)=	41.298311
Tnf=	0.5144559
Pm/Rm=	31.724339
Rf(M)=	382.05018

d	h	p	M/Mu	P/Pu	M/Mu+P/Pu
12	14.5	0.008491	0.5054434	0.2197773	0.7252207

pfdy/fdc= 0.0747208

M/Mu(all)

2.3190219

SHEAR CHECK

Vact=	710.20029
Vall=	7944.8275
VTall=	1517.8932

CRITICAL BUCKLING

Pcr=	41.299311
P/Pcr=	0.9999757
0.1M=	1.9201986
0.05M=	2.7090354

Vc=	1039.1194
WTc=	678.02546
WTst=	36.829373
TOTWT=	714.85484
$c=	67542.766
$st=	44195.248
TOT$=	111738.01

APPENDIX B
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