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NAVAL CIVIL ENGINEERING LABORATORY
Port Hueneme, California

Sponsored by
NAVAL ELECTRONIC SYSTEMS COMMAND

DESIGN OF PRESSURIZED FUEL STORAGE TANKS TO RESIST AIR OVERPRESSURES

May 1970

An Investigation Conducted by

T. Y. LIN AND ASSOCIATES
Van Nuys, California 91401

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DESIGN OF PRESSURIZED FUEL STORAGE TANKS TO RESIST AIR OVERPRESSURE

by

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for

U.S. NAVAL CIVIL ENGINEERING LABORATORY

under

CONTRACT NBy 62399-69-C-0017

with

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May 1970

FOREWORD

This report constitutes a portion of the Project Clarinet-Sanguine facilities research program which was accomplished through NCEL by contractural and in-house effort. The facilities research program was sponsored by the Earth Sciences Division of the Office of Naval Research (ONR) under their field projects program. ONR was charged by the Sanguine Division of the Naval Electronic Systems Command with the general management of the overall research program for Project Sanguine.

This facilities research program consists of work units as follows:

Facility System Study
Air Entrainment System
Water Wells
Air System Components
Fuel Storage Containers
Computer Output Display
Hardness of Buried Cable
Vertical Grounds

These work units encompass areas in which improved technology was needed or in which significant cost reduction might be achieved through improvement in extant knowledge. The research program at NCEL was initiated in early November, 1968.

General direction of the program was provided by Dr. T.P. Quinn and his staff of the Earth Sciences Division, ONR, with the counsel of Mr. W.J. Bobisch of the Naval Facilities Engineering Command. Mr. J.R. Allgood of NCEL served as project coordinator and Mr. S.K. Takahashi served as contract monitor for the work described in this report.

For T. Y. Lin and Associates the Project Supervisor was Mr. W. E. Gates who authored this report. Contributors to the report were Dr. S. Yaghamai and Mr. L. P. Prunotto.

ABSTRACT

This report presents a design procedure and cost optimization curves for cylindrical steel tanks with spherical end caps that are buried with their axis parallel to the ground surface. The fuel storage tanks are designed to resist high overpressures associated with nuclear air blast. Internal pressurization has been utilized to minimize material and fabrication costs and increase the buckling capacity of the tanks.

The design procedure is believed to be valid for overpressures ranging from 10 psi to 3000 psi. The overall reasonableness of the design procedure has been confirmed by experimental model tests on buried steel cylinders and pressurized tanks in granular materials.

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LIST OF SYMBOLS

A = arching factor

A_g = geometry factor

A_o = maximum active arching

A_s = plan area of tank

B = buoyant uplift force

C = soil cover thickness

C_s = propagation velocity of peak stress wave

C_t = compression wave velocity of the tank material

C_1 = seismic velocity of the soil layer above the water table

C_2 = seismic velocity below the water table

D = mean diameter of tank

DF = dynamic amplification factor

D_p^+ = total positive phase duration of the pressure pulse

d_o = depth of burial, from ground surface to crown

E = soil cover thickness

E_s = dynamic secant modulus of the soil in one dimensional compression

E_t = elastic modulus of the tank

e = Naperian constant

f_y = yield stress

H = depth of excavation

I = moment of inertia of the tank section

L = length of tank

L_w = depth factor

M_c = confined compression (secant) modulus of soil = $E_s/0.577$

n = experimentally determined constant

p = overpressure

p_{cr} = critical pressure

p_{DL} = dead load pressure

p_e = effective vertical interface pressure at crown of tank

p_i = internal pressure

p_o = peak surface overpressure

p_v = uniform vertical pressure in free-field at elevation of crown

p_z = vertical free-field soil pressure at a depth, z

p_{zw} = vertical soil pressure at water table

r = radius of tank cylinder

S = plan perimeter of tank

T_t = natural period of tank vibration

t = thickness of tank, time

t_d = positive time duration of equivalent triangular air blast impulse

t_i = positive time duration of triangular pulse which has the same total impulse as the actual pressure-time curve

t_{50} = positive time duration of triangular impulse which has the same time ordinate at 50 percent of peak overpressure as the actual pressure-time curve

t_∞ = positive time duration of triangular impulse which has the same slope as the initial decay portion of the pressure-time curve

W = weapon yield

z = depth below ground surface

z_w = depth of water table below ground surface

α = $\mathrm{Ln}\ \dfrac{1.5\ M_c}{E_t I/D^3}$

α_z = depth attenuation factor

γ_s = unit weight of unsaturated soil

γ_{sw} = unit weight of saturated soil

γ_t = unit weight of tank material

γ_w = unit weight of water

Δ_x = horizontal expansion of tank diameter

Δ_y = vertical expansion of tank diameter

δ = relative deflection between tank and free-field soil

μ = ductility factor

ν = Poisson's ratio

ρ_s = mass density of the soil

ρ_t = mass density of the tank

ρ_1 = unit mass of soil above the water table

ρ_2 = unit mass of saturated soil below the water table

σ_h = hoop stress

σ_m = meridional stress

σ_y = yield stress of tank material

Φ = $A_g \dfrac{M_c}{p_e}\ \delta$

ϕ = geometry - stiffness factor

ϕ_o = angle of internal friction of the soil

Section 1

INTRODUCTION

1-1 Background

The safe underground storage of large quantities of liquid fuel is a key factor in the design of certain defense facilities which must maintain full operation in the event of a nuclear attack. In high-overpressure regions of a nuclear blast, underground fuel tanks designed according to normal practice will require thick steel shells heavily reinforced with stiffeners to prevent inward buckling of the vessels. The material and fabrication costs of such structures are very high.

Preliminary studies indicated that major savings in material costs could be achieved by internally pressurizing the fuel tanks. In essence, the vessels are prestressed in tension to partially compensate for the high compressive forces induced in them under nuclear blast. Internal pressurization also stiffens the vessel, thus making it less susceptible to buckling.

A program of investigation was set up to achieve the following objectives:

1. To determine the feasibility of using pressurized flexible tanks buried near the earth's surface as an economical and blast-resistant method for storing fuel.
2. To develop design criteria for underground fuel storage facilities in the high overpressure regions of a nuclear blast.
3. To develop a finite element computer program which was applicable to the analysis of underground liquid storage containers. This program would serve as an analytical check on the design.

It was determined that verification of the above objectives could most readily be achieved through a series of static and dynamic tests on model fuel storage containers. The tests were designed to simulate the soil-structure-liquid interaction under nuclear blast loading. Results from the tests were used to:

1. Evaluate the blast-resistant properties provided by internal pressurization.

2. Establish design criteria for prototype structures based on the failure modes of the model and on the use of model similitude laws.

3. Provide experimental data which could be used to verify the analytical model and computational techniques used in the computer program.

The results of the model tests and the analytical procedure used in the computer program are presented in separate reports.[6,17]

1-2 Objective of Report

The first objective of this report is to present a procedure for the design of shallow buried, internally pressurized, fuel storage tanks constructed from steel cylinders and hemispherical end caps. The tanks are oriented horizontally and located at a depth of one diameter as shown in Figure 1-1. The environmental conditions to which the design procedure must be applicable are those associated with surface or near-surface nuclear detonations at ranges which produce overpressures as high as 3000 psi. In addition, dead weight of soil backfill and buoyant forces of high water table are included when appropriate in the environmental loading.

The soil conditions considered are typical of those known to exist at potential sites for Naval defense facilities.

The second objective is to provide cost optimization data on pressurized fuel storage tanks for various overpressures as a function of tank volume.

It was required that the design procedure and optimization data be presented in such a form that it could be immediately useful in a design office.

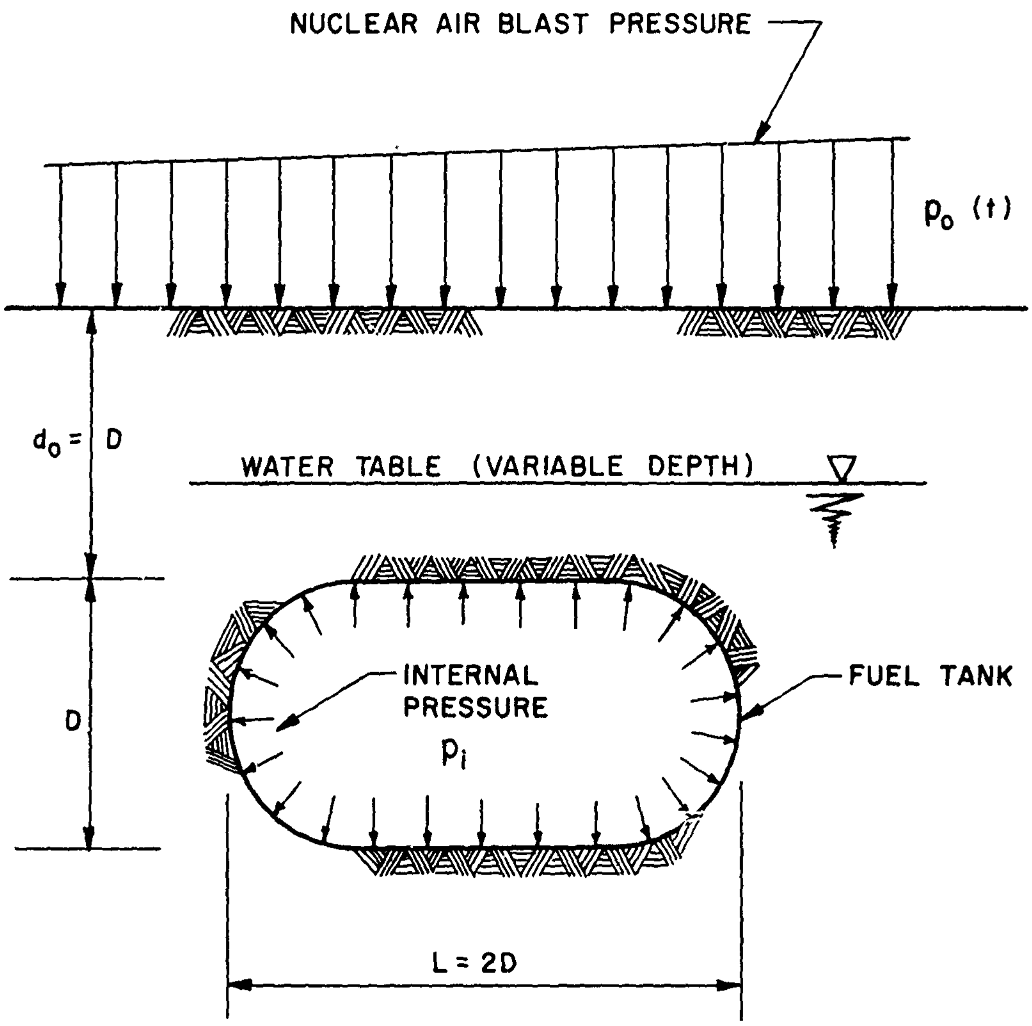


Figure 1-1 LONGITUDINAL SECTION OF BURIED FUEL TANK

1-3 Scope of Report

To pursue the first objective, that of providing a design procedure for buried fuel storage containers, the results of model tests on horizontally buried, internally pressurized steel tanks and horizontally buried steel cylinders were reviewed. Most of the available test data is for horizontal steel cylinders in sand [10] unpressurized, and without end cap constraint. To supplement these results, theoretical studies on buried fuel tanks were carried out by dynamic finite element methods on a linearly elastic tank, liquid, and soil system. The model was subjected to a static internal pressure and dynamic external pressure applied at the surface of the soil. Second order stiffness terms normally neglected in small deflection theory were included in the analysis in order to approximate the stiffening effect of internal pressurization upon the steel shell.

Nuclear blast loading used in the design of buried structures is adequately covered in numerous references on nuclear weapons effects [10,11,12,13,14]. Thus only a summary of the pertinent weapon effects is presented in this manual for the convenience of the designer. Attention is given to propagation of a surface air blast pressure pulse downward through the soil and around the tank. Both attenuation with depth and arching effects are considered in the design procedure.

Particular attention has been given to the problems associated with high water table since this is known to be a common occurrence at many of the proposed sites.

The design criteria for determining the wall thickness of the tank is based upon the requirement that the tank must be capable of surviving repeated nuclear attack without failure. This limits the maximum combined membrane stresses to the values just under the yield stress of the tank steel. Experimental data [6] have shown that there is a significant reserve capacity in the tank to resist loads well above the yield capacity, and failure in the form of leakage does not occur even after post elastic buckling. Under static loads produced by internal pressurization the tank is designed with a safety factor of 2.0.

The design procedure for determining the wall thickness of the tank is expressed as a function of the overpressure, weapon yield, dynamic amplification factor, internal pressure, soil dead loads, stress attenuation with depth, arching factor, hydrostatic loads, depths of burial, tank diameter, elastic soil properties and elastic tank properties. The procedure is presented as a detailed step-by-step method that can be used by a designer having only limited familiarity with protective construction. However, it is assumed that the designer is familiar with steel pressure vessel design.

To further simplify the designer's task a computer program was developed which will solve for the tank wall thickness. The program was used to generate a design chart giving tank thickness as a function of diameter and overpressure. (See Figure 1-2).

To achieve the second objective a series of computer solutions were carried out to design and cost evaluate horizontal steel tanks buried at one diameter in granular soils. Five water table conditions were considered:

1. No water present (i.e. water table at 100 feet below ground).
2. Water at 30 feet below ground.
3. Water at 13 feet below ground.
4. Water at 5 feet below ground.
5. Water at ground surface.

Two steel yield strengths were considered, one 90 ksi the other 36 ksi. Two cases of internal pressurization were evaluated, one with pressure the other without. In all of these design optimization studies tank volume and overpressure level were variables. The weapon yield and soil properties were held constant.

1-4 Organization of Report

This report has been organized primarily for the convenience of the designer. Hence, the body of the report contains only the design procedure, example problem, optimization curves, and design recommendations. All supplementary material and supporting data are presented in the appendices.

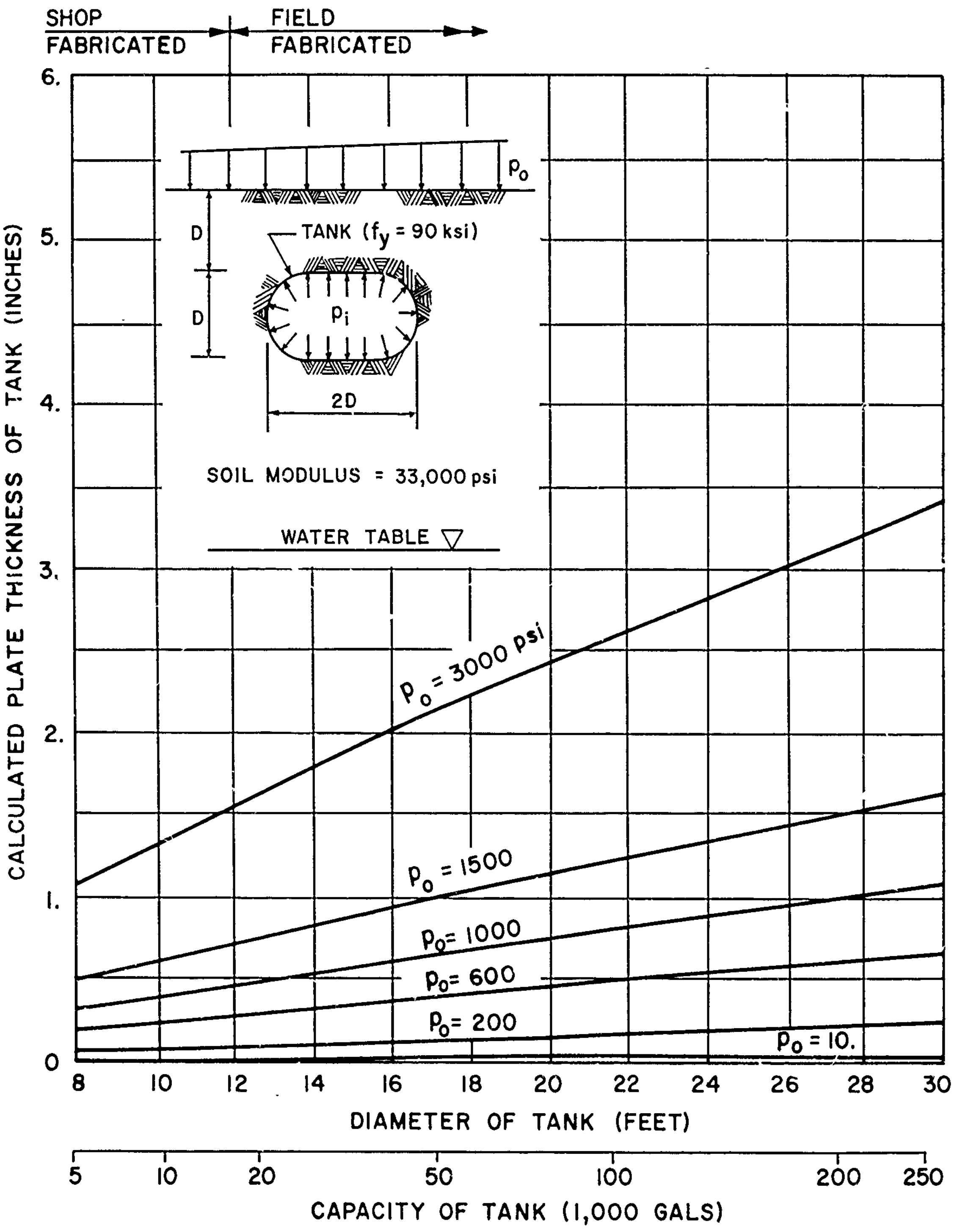


Figure 1-2 TANK THICKNESS AS A FUNCTION OF PEAK OVERPRESSURE LEVEL AND TANK DIAMETER

Section 1 contains background information, statements of objective and scope of this report, organization of material, and a summary of the major design recommendations.

Section 2 presents the basic formulation and step-by-step design procedure for calculation of tank thickness along with design examples. Attention is given to non-granular soils such as clay, silt and rock although the basic design procedure is primarily applicable to granular materials. Corrosion protection, backfilling procedures, mechanical consideration and site soil evaluation procedures are also discussed.

Section 3 presents a series of parametric studies to evaluate the minimum cost for tanks under specific design conditions. Cost optimization curves are included for various overpressure levels and tank volumes.

Appendix A contains a general flow diagram for the computer program used in tank design and cost optimization. Two example computer solutions are included.

Appendix B is devoted to the correlation of experimental data with theory.

Appendix C presents detailed recommendation on site soil investigation procedures.

1-5 Conclusions, Recommendations, and Design Criteria

The significant conclusions from study of test data, design recommendations, and associated design criteria are summarized below:

1. Internal pressurization has a distinct economical and structural benefit. It reduces the tank thickness to such an extent that material and fabrication cost savings far outweigh the added maintenance cost of internal pressurization plus the initial costs of compressor equipment and special pressure reducer valves.

2. Internal pressurization provides an elastic restoring force within the tank which prevents compressive buckling of the tank in the elastic range. Post elastic buckling

is possible, however, soil arching develops as the tank buckles, preventing the tank from rupturing.

3. Horizontally oriented tanks are superior to vertically oriented tanks in resisting nuclear blast loads. The difference in performance is attributed to the formation of a positive soil arch for horizontal tanks, while vertical tanks, behave as stiff inclusions.

4. Optimum depth of burial for tanks in dense fill is approximately one tank diameter.[10] At this depth advantage can be taken of earthcover in reducing the forces that are transmitted to the tank by the soil around it. Usually there is little advantage to be gained from a depth of burial greater than one diameter.

5. The design procedure presented here, assumes bending moments and local stress concentration to be of second order magnitude due to the relatively thin cross section required with internal pressurization. Any local yielding of the tank skin due to these second order effects is permissible under nuclear blast loading.

6. It has been specified that the tanks be designed to resist repeated nuclear air blast loading. To fulfill this criteria the principal membrane stresses are held below the yield stress of the tank material. The critical membrane stress, for which the tank is sized, is the springline thrust and axial compression.

7. A safety factor of 2.0 is applied to the design when checking for internal pressure loading alone. Thus for optimum design the effective static external pressure on the crown of the tank is three times the internal pressure.

8. For surface overpressure levels less than 200 psi it is probable that the final design of the cylinder will be controlled by non-blast-load conditions such as soil dead load forces, imposed by construction equipment and surface vehicular loads.

9. In areas where there is a potential for high water table, precautions should be taken to prevent buoyant uplift. This may be accomplished by providing a dewatering system, tie downs, or cut and cover techniques which place the tank well above the critical water level.

10. The design procedure presented in this report is directly applicable to granular soils in a dry or saturated state. However, for tanks located just above the water table, a more detailed analysis than outlined here may be required in order to determine the possible amplification of blast induced pressures caused by upward reflection at the water table. Ideally, the tank should be checked by dynamic finite element procedures similar to those used in predicting and evaluating the model tank tests reported in reference 6. The analytical model would be composed of three basic elements: soil, liquid, and tank steel. A schematic drawing of the finite element grid is shown in Figure 1-3.

11. There are no available test data for pressurized tanks buried in clay. In fact, there are insufficient data available for horizontal cylinders buried in clay to formulate a specific design procedure.[10] What data are available indicate that moments and thrusts induced in the cylinders buried in clay are considerably higher than those which would be produced under the same circumstances in a cylinder buried in sand. Thus, it is recommended that tanks in clay or other non-granular soils be backfilled with densely compacted granular material. The tanks can then be treated as though they were buried in sand, resulting in a more economical design.

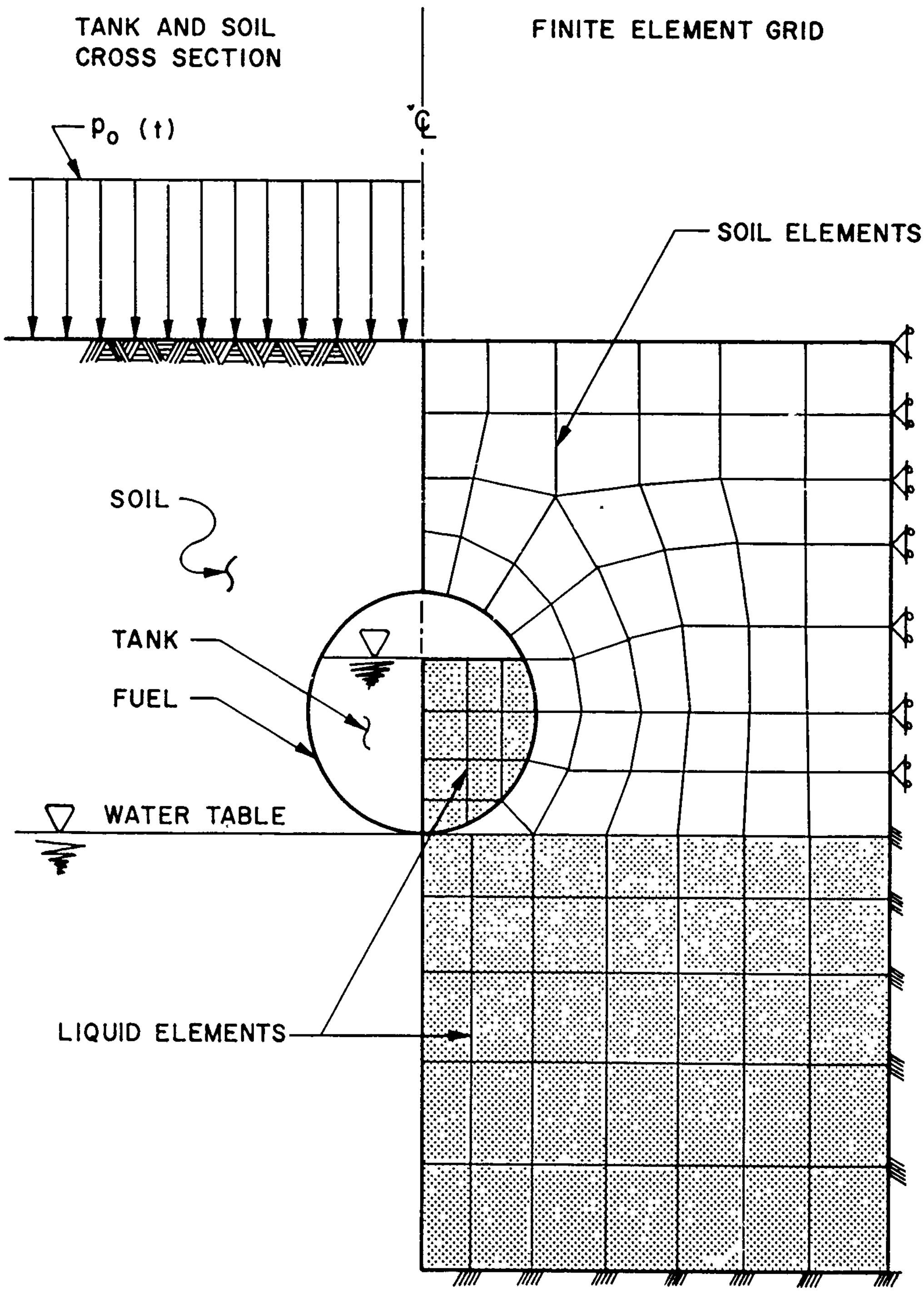


Figure 1-3 FINITE ELEMENT MODEL FOR HIGH WATER TABLE

12. For rock sites it will be necessary to overcut the excavation and backfill with dense granular material. A concrete cap should be provided which keys into the rock to prevent the tank from rebounding out of the ground. Because of possible amplification of blast induced pressures at the rock interface, the design procedure developed in this report is not fully applicable. An ideal analysis under these circumstances would be by dynamic finite element [6] utilizing a model similar to the one shown schematically in Figure 1-4.

13. Based on economic comparisons, high strength steels with yield stresses in the 90,000 psi range are more economical for pressurized tanks at high overpressure levels than normal structural steel. Material and fabrication costs double in going from 36,000 psi to 90,000 psi steel, while the yield strength increases by two and one-half times. Thus, it is possible to get more strength for less money.

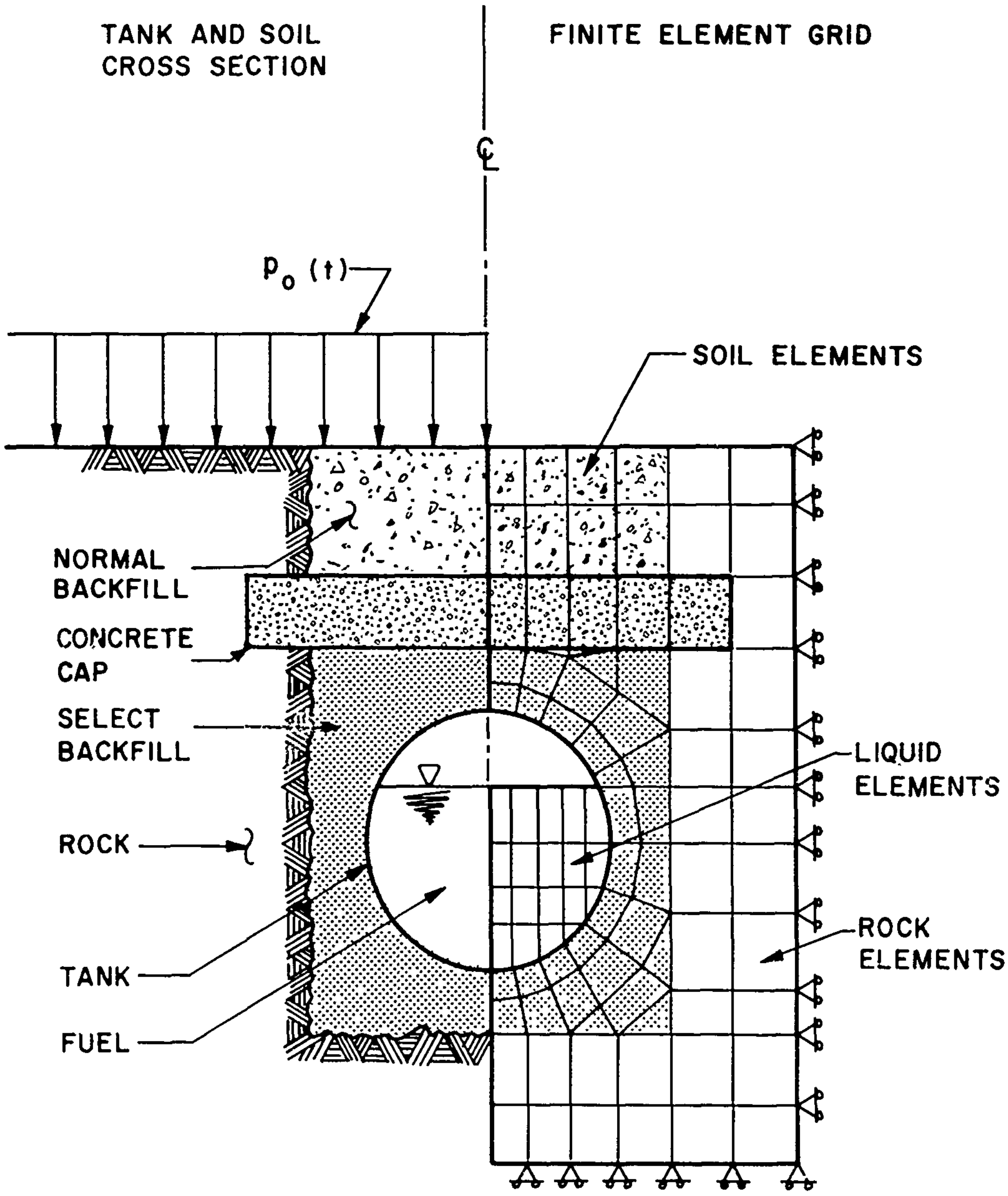


Figure 1-4 FINITE ELEMENT MODEL FOR ROCK SITE

Section 2

RECOMMENDED DESIGN PROCEDURE

2-1 Introduction

In this section recommendations are given for the design of buried fuel storage tanks to survive the environmental loadings associated with nuclear blast induced surface overpressures ranging up to 3000 psi. The design procedure which is developed and demonstrated by example is directly applicable to internally pressurized steel tanks, horizontally buried in dry granular soil. The method has been extended to saturated granular soils. However, no substantiating experimental data exists for the saturated soil case.

Figure 2-1 depicts the tank under nuclear blast loading. The ground surface pressures, p_o, generated by an air blast are normally defined as a function of weapon yield and distance from detonation. The effective soil pressure at the crown of the tank, p_e, is simply the ground surface pressure modified by the depth of burial and soil conditions. Due to the dynamic nature of the air blast loading, interaction between the buried tank and the surrounding soil must also be considered in arriving at the effective design loads.

2-2 Nuclear Weapons Effects

The nuclear weapon effects which are of major concern in the design of horizontally buried steel tanks are: air blast pressures, air blast induced ground shock, and directly-induced ground shock.

2-2.1 Air Blast Pressures

The general shape for the surface, air blast induced, overpressure-time function is shown in Figure 2-2 as a family of normalized curves for various overpressure levels. For design purposes a triangular pressure-time function, shown in Figure 2-3, may be used in place of the exact load function. To evaluate the normalized overpressure-time functions, the following data must be known:

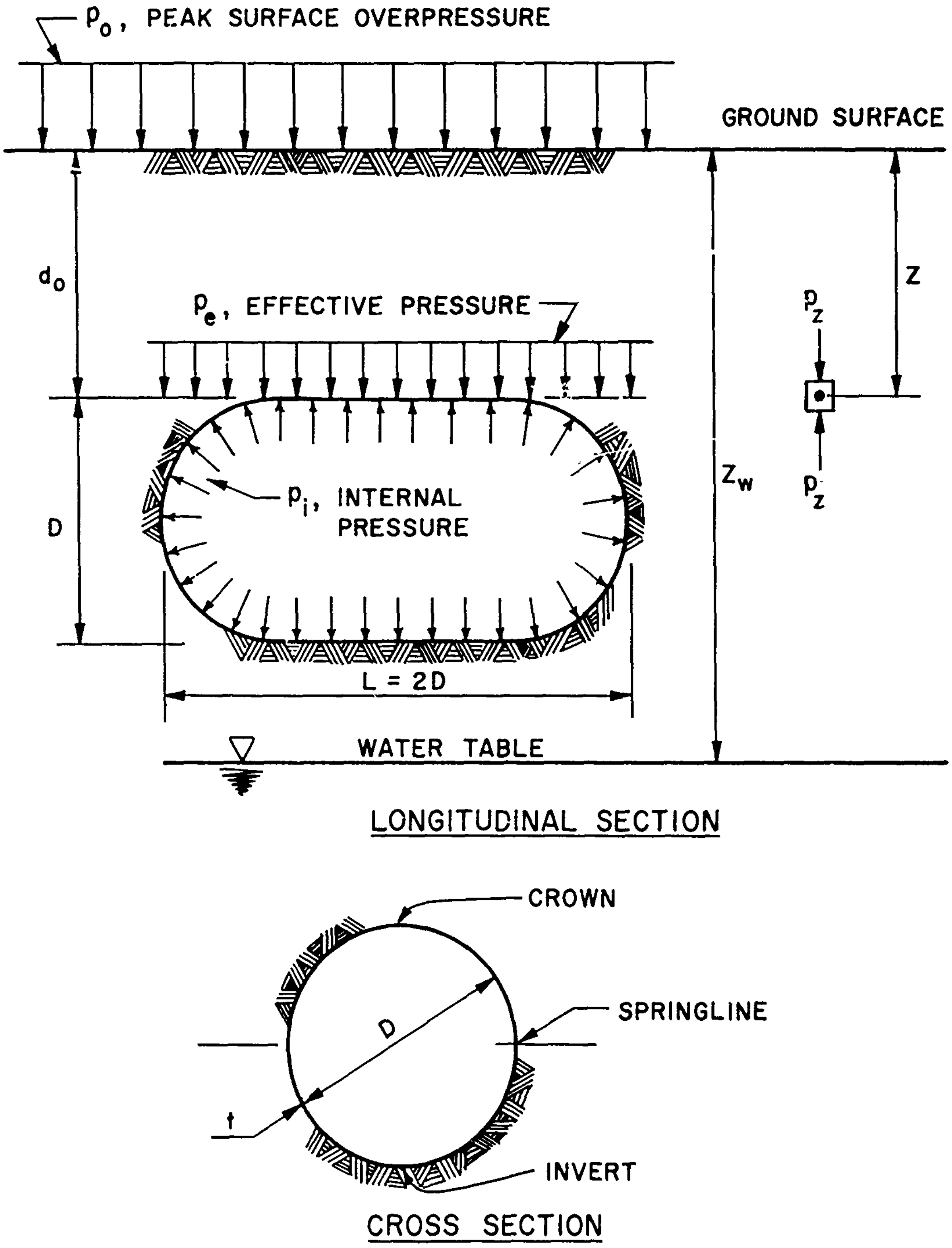


Figure 2-1 BURIED TANK NOTATION

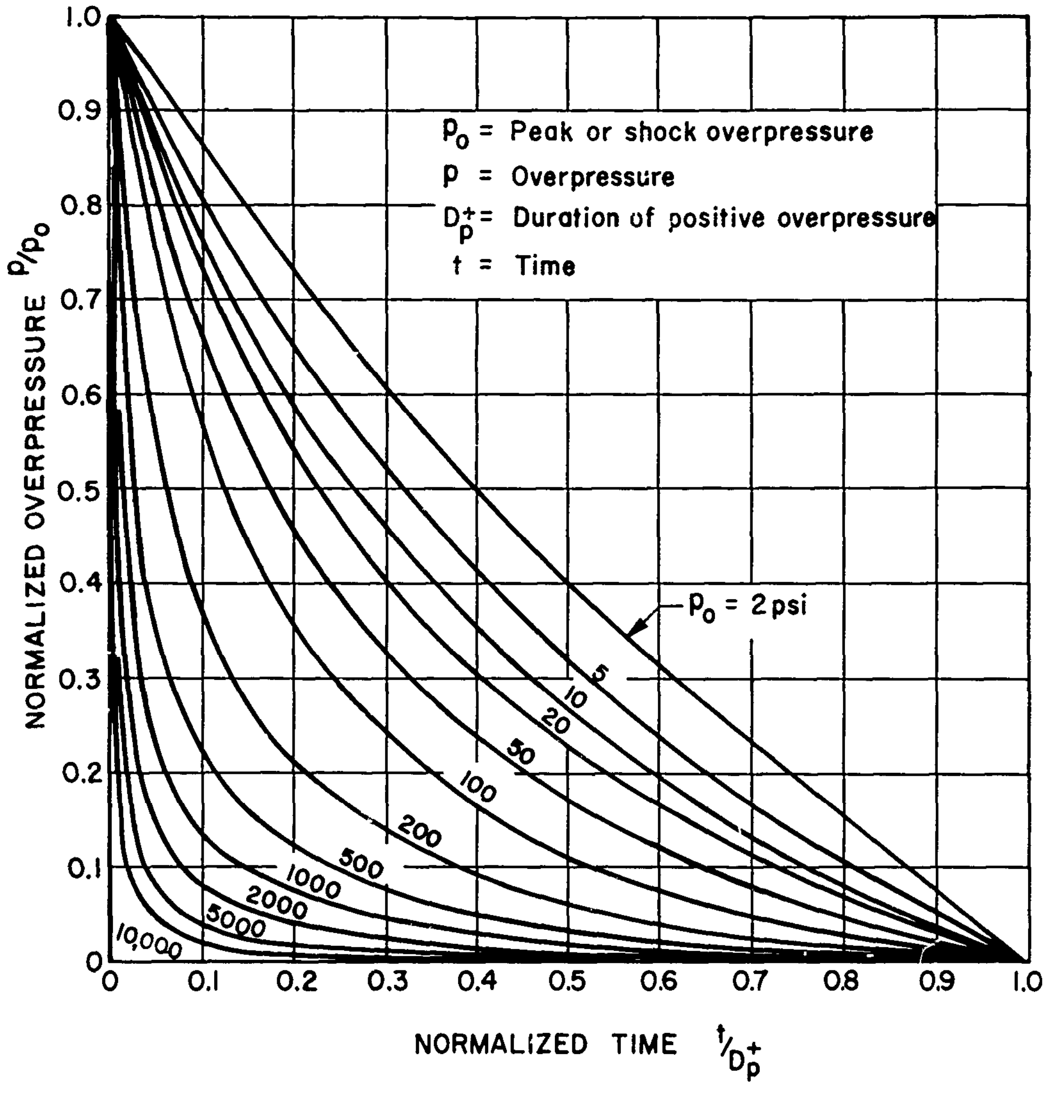


Figure 2-2 NORMALIZED OVERPRESSURE WAVE FORMS - I MT SURFACE BURST

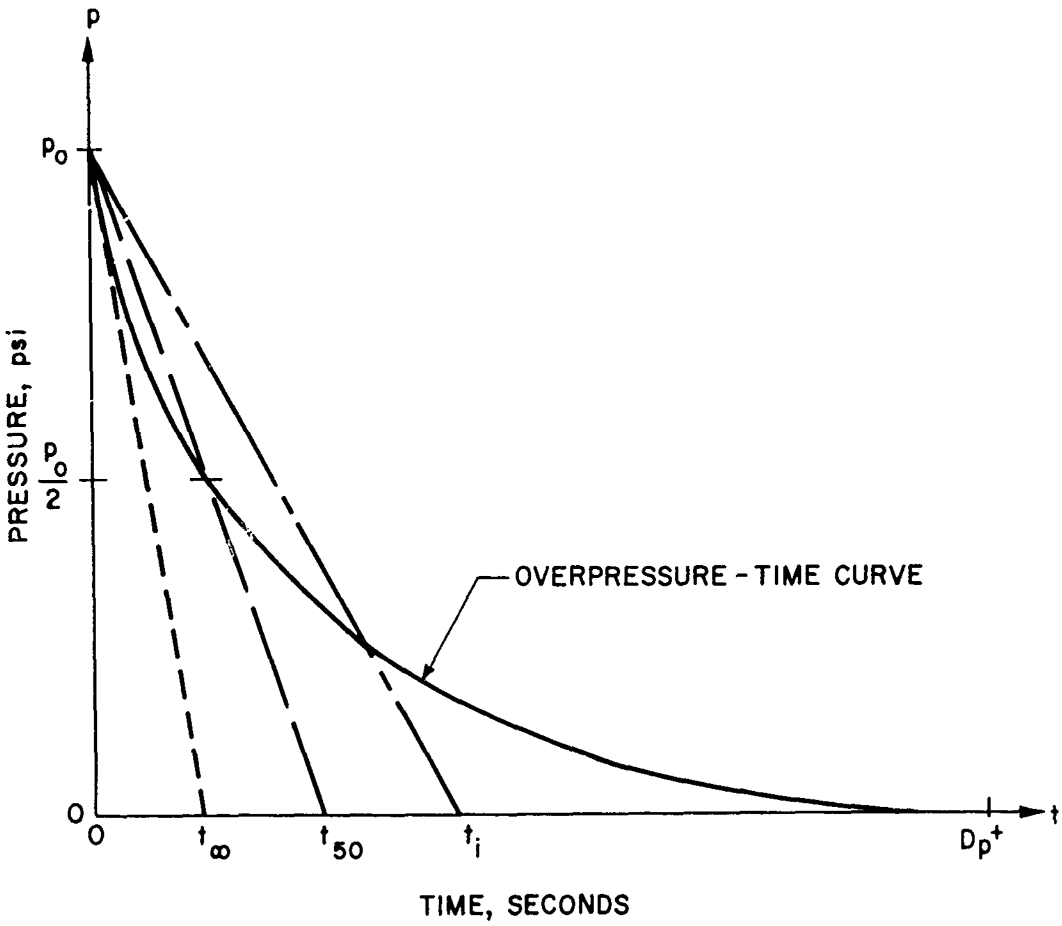


Figure 2-3 TRIANGULAR REPRESENTATIONS OF OVERPRESSURE-TIME CURVES

W, weapon yield

p_o, peak surface overpressure

D_p^+, total positive phase duration of the pressure pulse

t_∞, the positive time duration of a triangular impulse which has the same slope as the initial decay portion of the pressure-time curve

t_{50}, the positive time duration of a triangular impulse which has the same time ordinate at 50 percent of peak overpressure as the actual pressure-time curve

t_i, the positive time duration of a triangular pulse which has the same total impulse as the actual pressure-time curve

Values of D_p^+, t_∞, t_{50}, and t_i, are given in Figure 2-4 for a wide range of peak overpressure levels. The durations shown are for a 1 MT surface burst, as indicated on the figure. They may be scaled for other weapon yields by multiplying by

$$\left(\frac{W}{1MT}\right)^{1/3}$$

The triangular pulse of t_i duration is normally used when maximum response of the structure occurs after the pressure pulse has passed. A small triangular impulse of duration t_∞ produces critical loads on very stiff structures which reach maximum response early in the pressure history. Triangular loads of duration t_{50} are applicable if the response of the structure is intermediate to those mentioned above.

If greater accuracy in analysis is desired than that possible with the single triangle, a multiple-triangle representation of the pressure-time curve may be used.(12)

2-2.2 <u>Crater Induced Ground Shock</u>

The cratering action of a surface burst will produce motion of significant magnitude in buried structures located near the perimeter of the crater. These motions

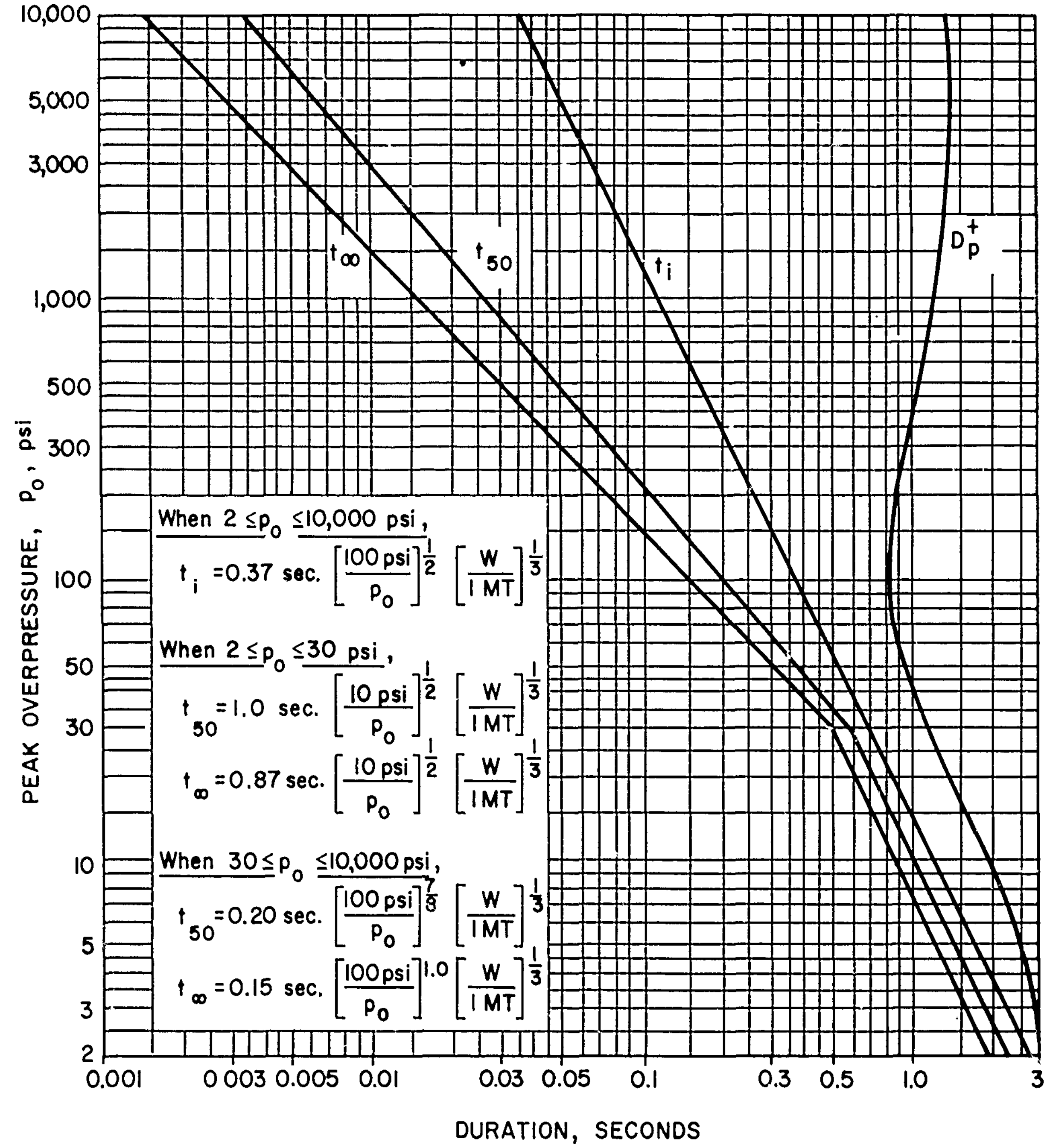


Figure 2-4 DURATION OF DATA FOR REPRESENTATION OF OVERPRESSURE - TIME CURVES - 1 MT SURFACE BURST

will have direct bearing on the design of shock isolation equipment for personnel and equipment. However, for the overpressures considered in this report (3000 psi), the structural resistance of pressurized steel tanks will be more than adequate if they are designed to resist the air blast induced soil pressures developed by the surface burst.

2-2.3 Air Blast Induced Ground Motion

The air blast induced soil stresses can be grouped into two categories depending on the relative time of arrival between ground shock and air blast. Super seismic conditions occur when the air blast velocity exceeds the dilatational wave velocity of the soil. Under these conditions the air blast arrives before the blast induced ground motion. The outrunning case occurs when the air blast velocity is less than the dilatational wave velocity of the soil.

The characteristics of the ground motion produced under super seismic conditions differs considerably from those produced under outrunning conditions. However, the detailed difference is only significant to shock isolation design. As far as the buried tank design is concerned, the vertical blast pressure induced soil stresses, p_e, at the crown of the tank are of primary importance, and these stresses are identical under both of the ground motion regimes.

2-2.4 Attenuation of Dynamic Stress with Depth

The pressure to be used in the design is the vertical free field stress, p_z, at a depth corresponding to the crown of the tank (see Figure 2-1). The magnitude of p_z is a function of the air blast characteristics at the ground surface above the point of interest and of the stress-strain properties of the soil. At the ground surface the vertical soil stress will have the same magnitude and time variation as the air blast overpressure. As the pressure wave propagates down through the soil, attenuation of peak pressure takes place along with a lengthening of the rise time and positive phase duration. These effects, which are shown schematically in Figure 2-5, are caused by the non-linear stress-strain behavior of the soil under initial loading and by the energy absorbed in permanent strain deformation.

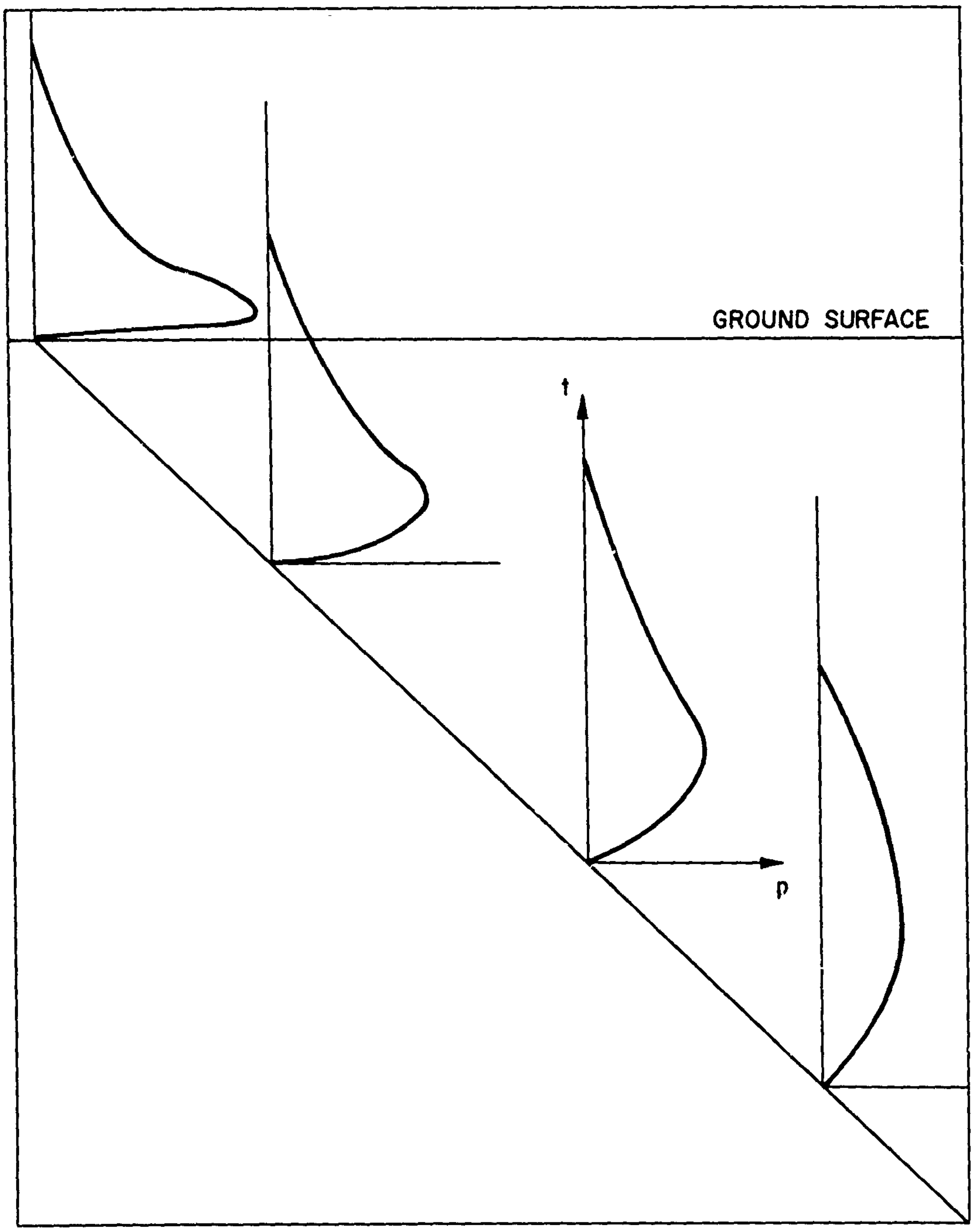


Figure 2-5 CHANGE IN PRESSURE - TIME CURVES WITH DEPTH

The vertical free-field soil stress at depth, z, can be expressed as a function of the peak surface overpressure and a depth attenuation factor, α_z, in the following manner [3,9] :

$$p_z = \alpha_z \, p_o \tag{2-1}$$

in which

$$\alpha_z = \frac{1}{1 + \frac{z}{L_w}} \qquad \text{for } p_o < 1000 \text{ psi} \tag{2-2a}$$

or

$$\alpha_z = \left(\frac{1}{1 + \frac{z}{L_w}}\right)\left(\frac{27.5}{p_o^{\,0.48}}\right) \qquad \text{for } p_o > 1000 \text{ psi} \tag{2-2b}$$

and

$$L_w = 230 \text{ ft.} \left(\frac{100 \text{ psi}}{p_o}\right)^{1/2} \left(\frac{W}{1\text{MT}}\right)^{1/3} \left(\frac{C_s}{622 \text{ fps}}\right) \tag{2-3}$$

where:

C_s = propagation velocity of the peak stress wave

L_w = depth factor

W = weapon yield

z = depth below ground surface

The effective propagation velocity of the peak soil stress during the initial loading cycle is given by:

$$C_s = \sqrt{\frac{E_s}{\rho_s}} \tag{2-4}$$

where:

ρ_s = mass density of the soil

E_s = dynamic secant modulus of the soil in one dimensional compression

The secant modulus is determined at stress levels commensurate with the dynamic overpressure expected. If the dynamic secant modulus is unknown, C_s may be taken as one-half the seismic velocity for surface soils above the water table.[10]

2-2.5 Pressure Amplification in Saturated Soils

The presence of the ground water table in the vicinity of the buried tank will alter the attenuation of blast pressure. If the water table is at ground surface, no attenuation of the air blast pressure should be considered (i.e., $\alpha_z = 1.0$).[10] If the water table lies between the ground surface and the structure as shown in Figure 2-6, the attenuation factor, α_z, for the dry soil, from ground surface down to the water table at depth, z_w, should be computed by Equation 2-1. At the water table, the vertical pressure transmitted through the soil-water interface will be:

$$p_{zw} = 2\left[\left(\frac{\rho_2 C_2}{\rho_1 C_1 + \rho_2 C_2}\right)\right] \alpha_z\, p_o \tag{2-5}$$

where:

C_1 = seismic velocity of the soil layer above the water table

C_2 = seismic velocity below the water table, (not less than 5100 ft/sec.)

ρ_1 = unit mass of soil above the water table

ρ_2 = unit mass of saturated soil below the water table

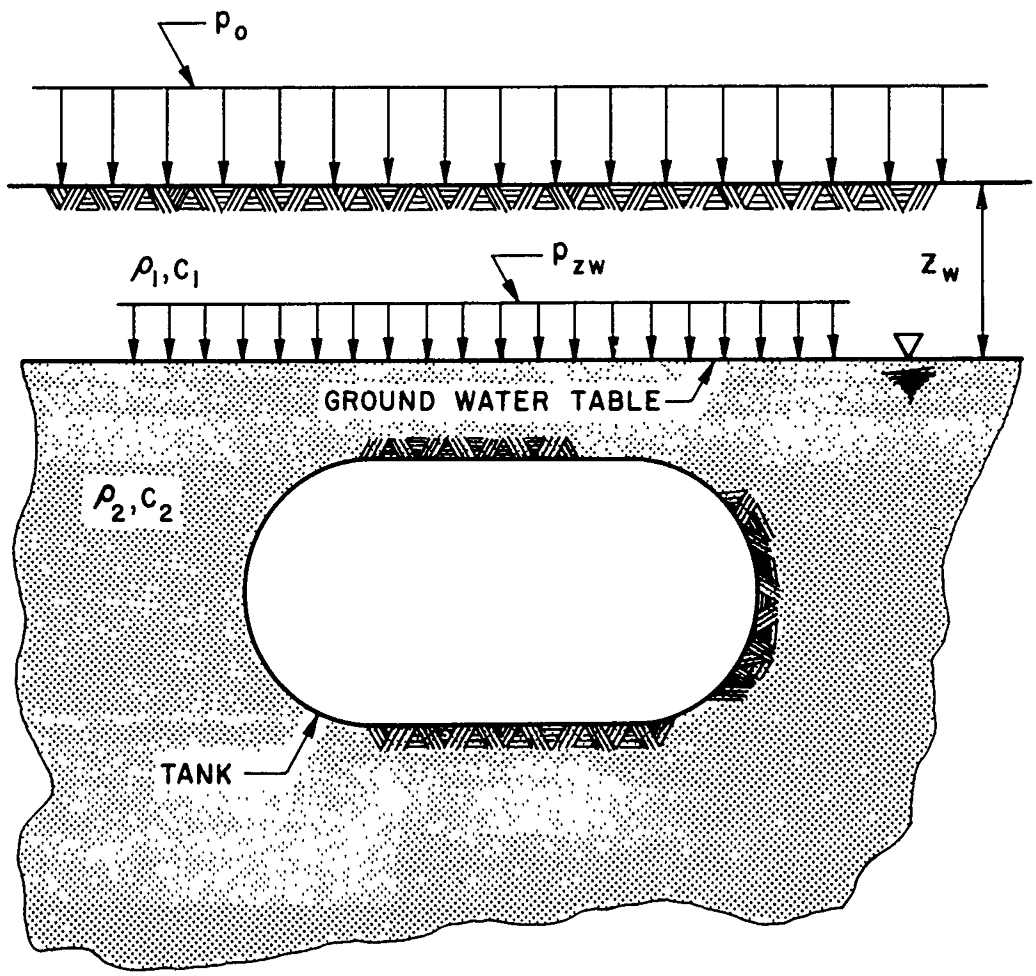


Figure 2-6 TANK BELOW GROUND WATER TABLE

Below the water table the transmitted pressure should not be attenuated with depth.

Due to possible pressure reflection at the water table interface it would be conservative to consider the soil pressure immediately above the water table to be equal to the pressure below the water table.

The presence of a high water table presents several design problems in addition to the pressure amplification. Excavation and construction will require expensive dewatering equipment. When the tank is finally placed and backfilled, provisions must be made to keep it in the ground under buoyant uplift forces. There are three alternatives:

1. Provide permanent dewatering.

2. Provide massive hold down anchors.

3. Use a shallow excavation and mound the soil as shown schematically in Figure 2-7.

From a practical design standpoint, every effort should be made to keep the tank above the water table.

2-2.6 Soil Arching

A steel tank buried in a homogeneous soil behaves as an inclusion, producing a discontinuity in the soil field. If the soil is stiffer than the tank, under vertical loads the soil will transfer the load around the tank by arching action as shown in Figure 2-8. If on the other hand, the tank is stiffer than the surrounding soil, vertical loads in the soil will be transferred to the tank, thus increasing the effective pressure acting on the crown of the tank above the normal free-field pressure.

By definition, arching is that fraction of the free-field soil pressure at the crown elevation of the tank which is transferred to or away from the tank by shear in the soil. Or in equation form:

$$A = \left(1 - \frac{p_e}{p_v}\right) \qquad (2\text{-}6)$$

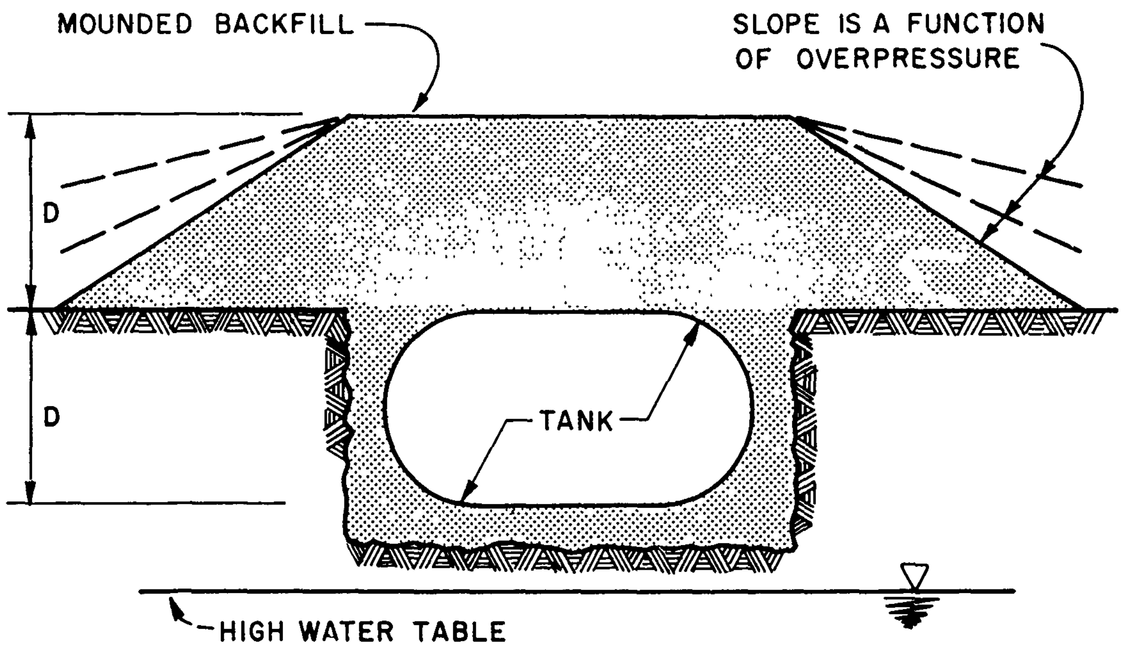


Figure 2-7 SHALLOW EXCAVATION WITH MOUNDED BACKFILL

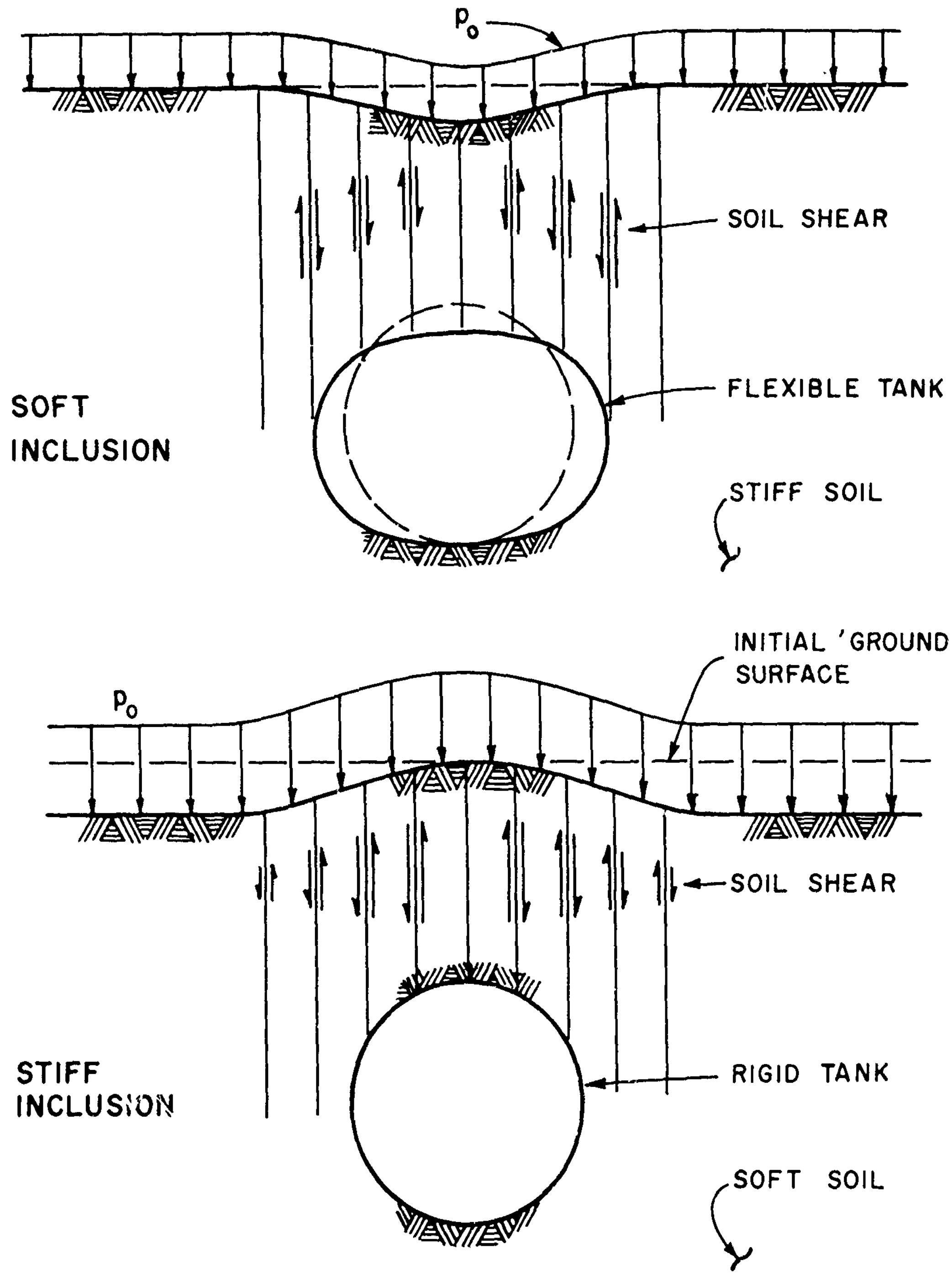


Figure 2-8 SOIL ARCHING

where:

p_e = the vertical soil pressure at interface of tank crown

p_v = uniform pressure in free-field at elevation of crown

The effective pressure at the crown of the tank due to soil arching is:

$$p_e = (1 - A)\, p_v \tag{2-7}$$

For tanks that are stiffer than the surrounding soil the arching factor, A, will be negative, thus producing higher design forces than if the tank were at the ground surface.

The following empirical arching equation has been established by Gill and True [7] for buried cylinders in dry dense granular soils:

$$\left(1 - \frac{A}{A_o}\right) = e^{-n\Phi} \tag{2-8}$$

where:

$$\Phi = A_g \frac{M_c}{p_e} \delta \tag{2-9}$$

$$A_g = \frac{Sd_o}{A_s D} \tag{2-10}$$

and

A = arching factor

A_o, n = experimentally determined constants ($A_o = 0.87$ and $n = 0.135$ for sharp grained sandblaster's sand)

A_g = geometry factor

e = Naperian constant

M_c = confined compression (secant) modulus of soil = $E_s/0.577$

p_e = effective interface pressure at crown of tank = $(1-A)\ p_v$

δ = relative deflection between soil and tank

S = plan perimeter of tank

d_o = depth of cover over crown

A_s = plan area of tank

D = mean diameter of tank

For horizontally buried tanks of length L, the geometry factor is:

$$A_g = \frac{4d_o}{D}\left(\frac{\pi D - 2D + 2L}{\pi D^2 + 4DL - 4D^2}\right) \qquad (2\text{-}11)$$

The relative deflection between tank and surrounding soil is:

$$\delta = \left(\frac{\Delta_y}{D} - \frac{p_v}{M_c}\right) D \approx \left(\frac{\Delta_x}{D} - \frac{p_v}{M_c}\right) D \qquad (2\text{-}12)$$

where:

Δ_x = horizontal expansion of tank diameter

Δ_y = vertical expansion of tank diameter

$\frac{\Delta_x}{D}$ ≈ average vertical strain in the tank

$\frac{p_v}{M_c}$ = average vertical strain in the soil

Substituting Equation 2-7, 2-11 and 2-12 into Equation 2-9 yields the following expression for Φ:

$$\Phi = \frac{4d_o(\pi D - 2D + 2L)}{(\pi D^2 + 4DL - 4D^2)} \left(\frac{\Delta_x}{D} \cdot \frac{M_c}{p_v} - 1 \right) \frac{1}{(1-A)} \qquad (2\text{-}13)$$

For convenience, Equation 2-13 may be expressed as:

$$\phi = \frac{\Phi}{1-A}$$

where:

$$\phi = \frac{4d_o(\pi D - 2D + 2L)}{(\pi D^2 + 4DL - 4D^2)} \left(\frac{\Delta_x}{D} \cdot \frac{M_c}{p_v} - 1 \right) \qquad (2\text{-}14)$$

Then Equation 2-8 may be rewritten as:

$$\left(1 - \frac{A}{A_o}\right)^{(1-A)} = e^{-n\phi} \qquad (2\text{-}15)$$

Experimental evidence indicates that for dry granular soils the maximum value of active arching is:

$$A_o \simeq \mathrm{Tan}\ \phi_o \qquad (2\text{-}16)$$

where, ϕ_o is the angle of internal friction of the soil.

From elastic theory and tests data on buried cylinders (2) the deflection ratio in Equation 2-14 is:

$$\frac{\Delta_x}{D} \cdot \frac{M_c}{p_v} = 2\ (1 - e^{-0.024\alpha^2}) \qquad (2\text{-}17)$$

where:

$$\alpha = \mathrm{Ln}\left(\frac{1.5\ M_c}{E_t I/D^3}\right)$$

and

E_t = elastic modulus of the tank

I = Moment of inertia of the tank section = $t^3/12$

t = thickness of tank

thus:

$$\phi = \frac{4d_o(\pi D - 2D + 2L)}{(\pi D^2 + 4DL - 4D^2)}\ (1 - e^{-0.024\alpha^2}) \quad (2\text{-}18)$$

To assist the designer in solving the arching Equation 2-15 for the arching factor, A, a graphical plot of the function has been prepared in Figure 2-9.

2-2.7 Dynamic Amplification

Nuclear air blast is a short duration high intensity form of loading which excites dynamic inertia forces in a soil-tank system. Dynamic amplification of vertical soil pressure at the crown of a tank can be approximated by the following relationship [13]:

$$DF = \frac{1}{\frac{T_t}{\pi t_d}\sqrt{2\mu - 1} + \frac{1 - \frac{1}{2\mu}}{1 + .7\frac{T_t}{t_d}}} \quad (2\text{-}19)$$

in which the tank-soil system is idealized as a single-degree-of-freedom oscillator, having an elasto-plastic restoring force, and excited dynamically by a triangular impulse of instantaneous rise time.

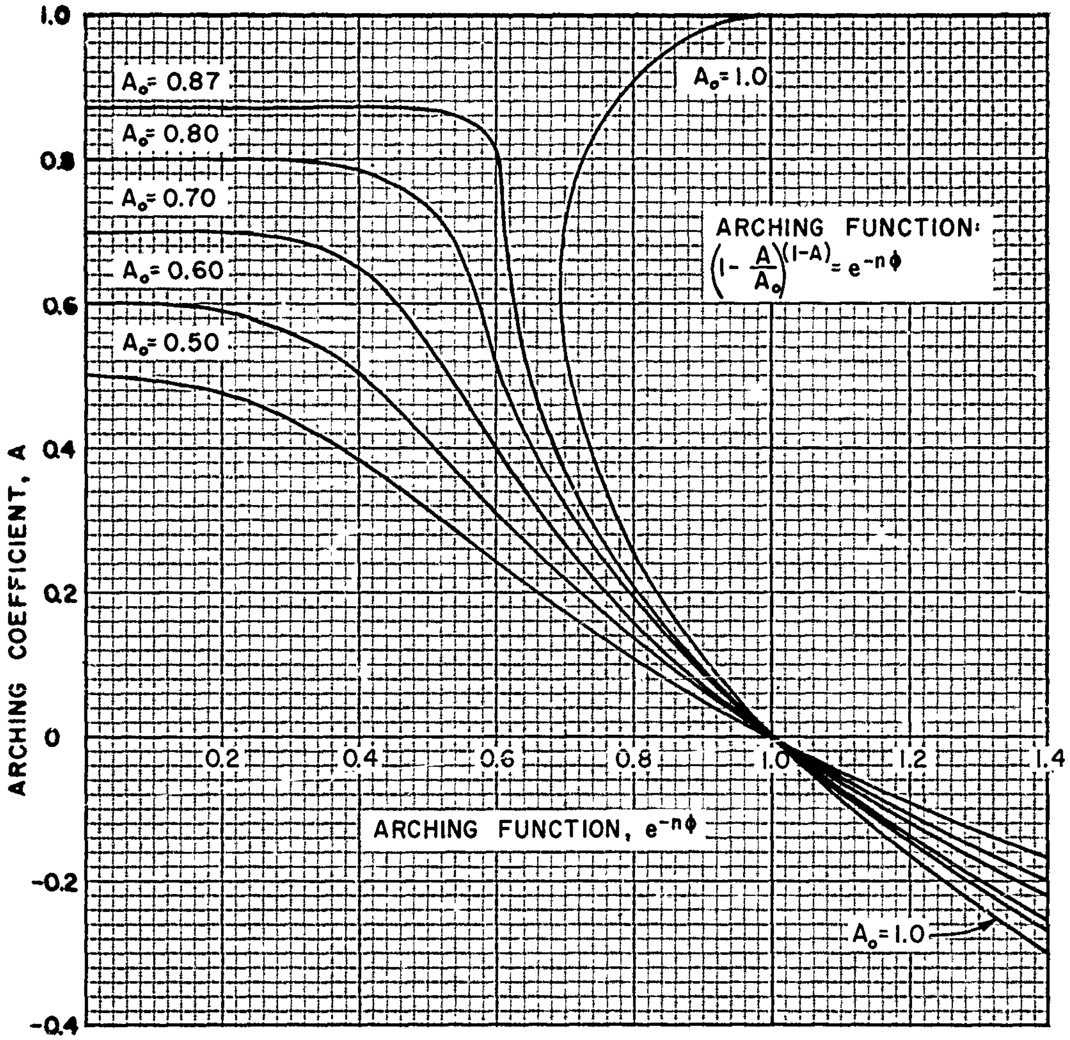


Figure 2-9 GRAPH OF ARCHING FUNCTION

The term, t_d, is the positive time duration of the air blast impulse which is defined as an equivalent triangular impulse of durations t_i, t_{50}, and t_∞ in Section 2-2. For design purposes select the triangular pulse which produces the largest dynamic amplification.

The ductility factor, μ, is a ratio of the total plastic strain in the tank under peak loading to the elastic strain at yield. Since the tanks must be designed to survive repeated blast loading, strains in the tank cannot exceed the yield limit. Thus, if μ is set equal to one, the response of the tank to dynamic loading will be elastic.

The term, T_t, in Equation 2-19 is the natural period of vibration for the tank in its fundamental compressional mode. This is the primary mode of oscillation in which the tank will respond to dynamic vertical soil pressures. There are other modes which will be excited as well, such as the flexural modes shown in Figure 2-10. These modes are generally not significant for tanks with length to diameter ratios of two.

The tank's natural period in the compressional mode is:

$$T_t = \frac{\pi D}{C_t} \qquad (2\text{-}20)$$

where, C_t is the compressional wave velocity in the tank material which may be computed by the relationship:

$$C_t = \sqrt{\frac{(1 - \nu)\, E_t}{(1 + \nu)\,(1 - 2\nu)\, \rho_t}} \qquad (2\text{-}21)$$

where:

ν = Poisson's ratio

E_t = elastic modulus of the tank

ρ_t = mass density of the tank

For steel, C_t = 18,470 ft/sec.

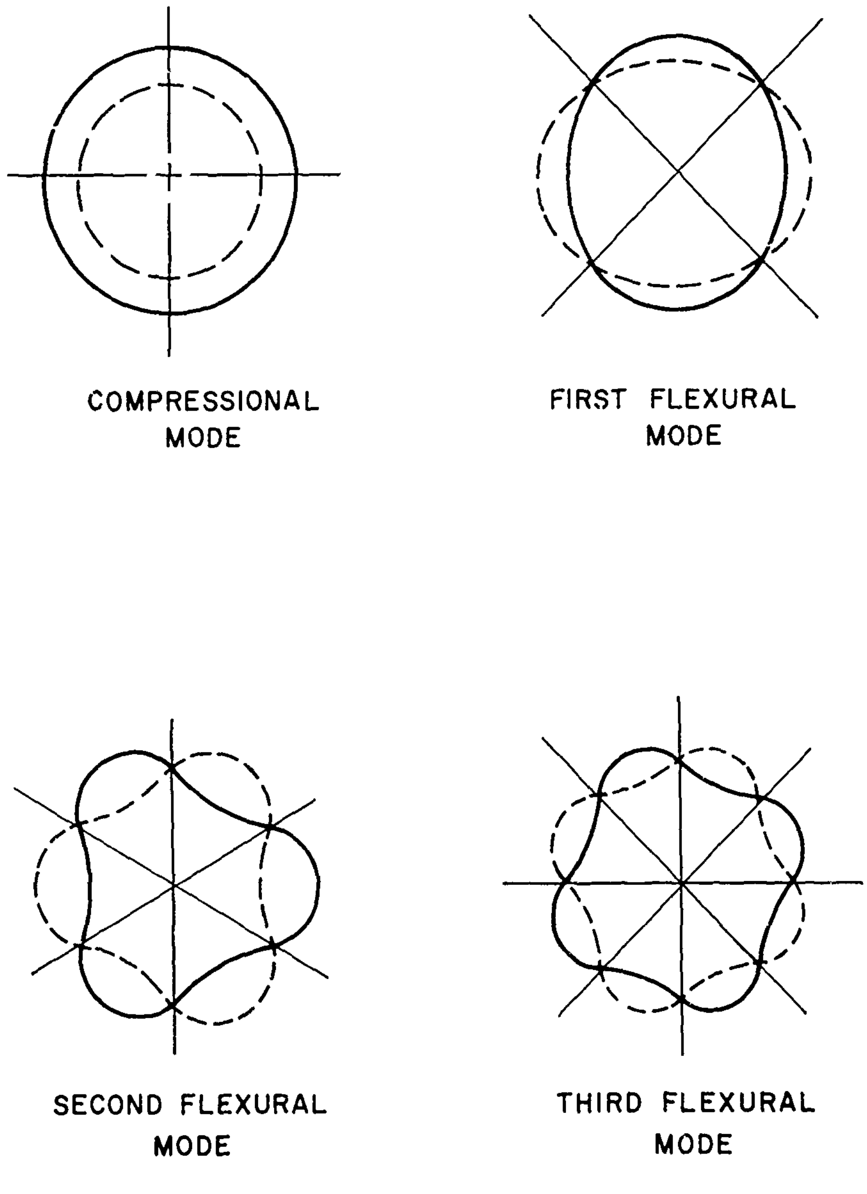


Figure 2-10 NATURAL MODES OF VIBRATION FOR TANK CROSS SECTION

2-3 Yield Load Capacity of Tank

The critical pressure distribution on the tank is produced by uniform pressure as shown in Figure 2-11. This pressure distribution is a very close approximation for tanks buried below the water table. For tanks buried in dry granular soils, the lateral pressures will normally be smaller than the vertical pressures. Under this imbalance of loading a certain amount of lateral expansion will take place until the passive soil pressure against the sides of the tank equalizes the vertical pressure. At this point the pressure on the tank may be approximated by a uniform distribution.

Secondary bending stresses will be produced in the cylinder as it ovalates. The magnitude of these stresses will not be significant in the design, provided the tank is buried at least one-half diameter and preferably one diameter below the surface.

The membrane stresses in the cylinder portion of the tank under uniform pressure are:

$$\sigma_h = \frac{p_{cr} D}{2t} \tag{2-22}$$

and

$$\sigma_m = \frac{p_{cr} D}{4t} \tag{2-23}$$

for the hoop and meridional directions respectively.

Assuming failure criterion to be the yielding of the tank and assuming the stresses in Equations 2-22 and 2-23 to be the principal stresses, then by von Mises yield criterion:

$$\sqrt{\sigma_h^2 - \sigma_h \sigma_m + \sigma_m^2} = \sigma_y \tag{2-24}$$

where σ_y is the yield stress in compression.

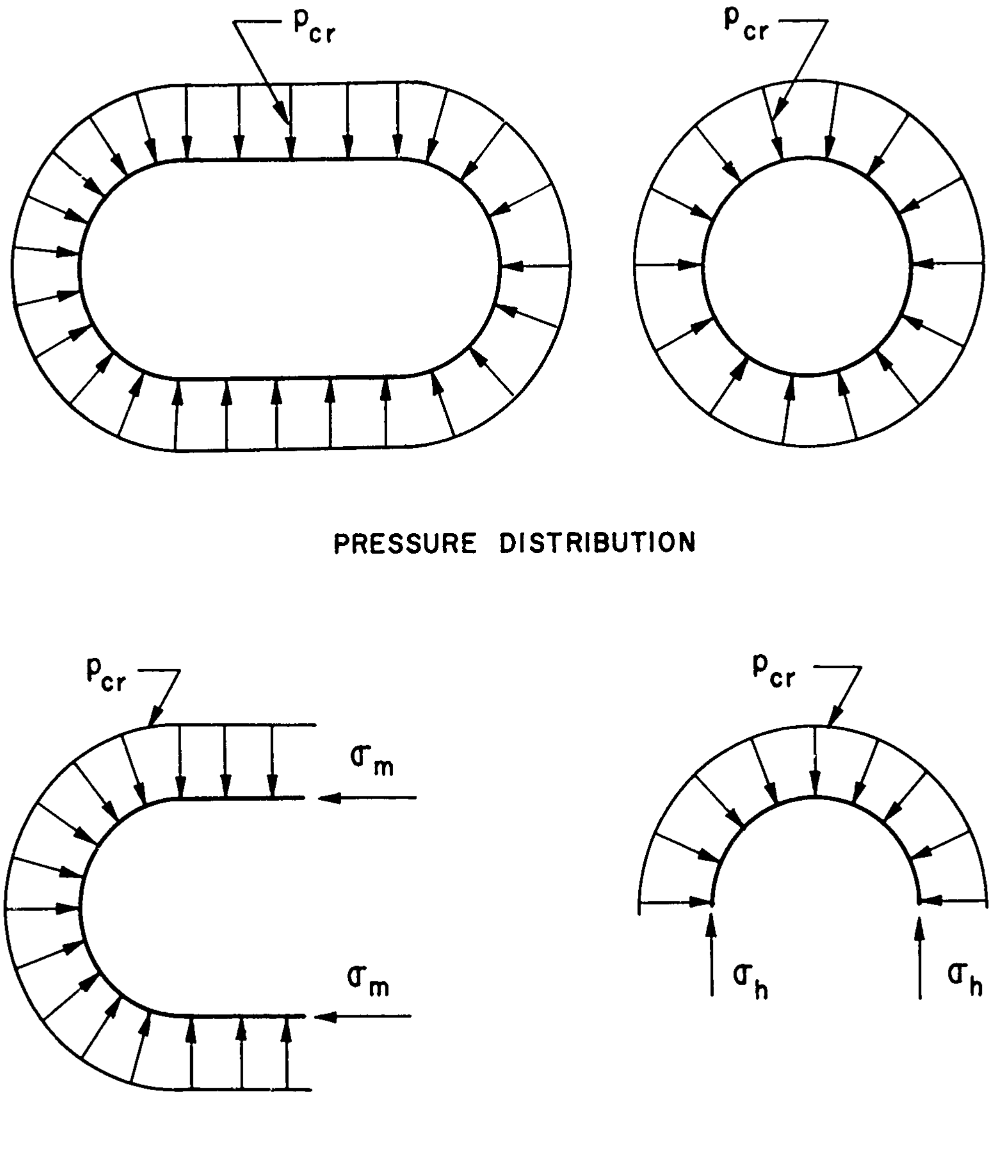


Figure 2-11 PRESSURE DISTRIBUTION FOR DESIGN CAPACITY OF TANK

By substituting Equations 2-22 and 2-23 into Equation 2-24, the yield criteria reduces to a simple relationship for the critical design pressure as a function of yield stress, tank thickness, and diameter:

$$p_{cr} = \frac{2.31t}{D} \sigma_y \qquad (2\text{-}25)$$

2-4 Buckling

There are no analytical relationships for the buckling capacity of horizontally buried cylindrical tanks with spherical end caps. However, experimental results from static tests [6] on internally pressurized tanks indicate that buckling will not occur as long as the principal membrane stresses remain below the yield limit of the tank material.

Chelapati [4] has derived an expression for the critical buckling pressure of buried cylinder which has been shown to agree with experimental results [1]. For cylinders of large length-to-diameter ratios, large diameter-to-wall-thickness ratios, and large values of foundation coefficient - conditions existing in the tests [1] - the critical buckling pressure can be expressed as:

$$p_{cr} = 0.612\sqrt{\frac{E_s E_t}{(r/t)^3}} = 4.5\sqrt{M_c \frac{E_t I}{D^3}} \qquad (2\text{-}26)$$

where:

p_{cr} = critical buckling pressure

t = cylinder thickness

r = cylinder radius

E_t = elastic modulus of the cylinder material

E_s = elastic modulus of the soil = $0.577\ M_c$

M_c = one-dimensional compression secant modulus at the surface pressures of interest

I = $t^3/12$ = moment of inertia of the tank section

which applies to the case with all-around soil support. As a conservative check on the buckling capacity of the tank, Equation 2-26 may be used.

2-5 Internal Pressurization

The primary benefit achieved through internal pressurization is a thinner tank thickness with resulting savings in material, fabrication, and construction costs. Internal pressurization pre-tensions the tank shell to partially offset the compressive stresses produced in the tank under nuclear air blast.

Since the tank will be internally pressurized almost continuously throughout its life, except for times when it is inspected and cleaned, it is suggested that a safety factor of 2.0 be applied to this design loading.

2-6 Dead Load of Soil

In the high overpressure range from 200 psi to 3000 psi, the influence of soil dead load on the tank design is negligible. However, in the overpressure range under 200 psi soil dead load should be considered.

2-7 Design Procedure

2-7.1 Proportioning of Tank Thickness

In this section, a design procedure for selection of the tank wall thickness is outlined in a step-by-step manner. In section 2-8, two design examples are presented to illustrate this procedure. One example deals with the design of a tank in dry granular soils, the other covers the design case in saturated granular soils where the water table is above the tank.

The design procedure presented here is limited to horizontally oriented cylindrical tanks with spherical end caps.

The following parameters must be known or assumed in order to complete the design:

D = diameter of the tank

L = length of tank

d_o = depth of burial, from ground surface to crown

z_w = depth of water table below ground surface

σ_y = yield stress of tank material

E_t = elastic modulus of the tank

ν = Poisson's ratio for the tank

γ_t = unit weight of the tank material

E_s = elastic secant modulus of the soil

γ_s = unit weight of unsaturated soil

γ_{sw} = saturated unit weight of the soil

ϕ_o = angle of internal friction of the soil

p_o = peak overpressure level

W = weapon yield

A = arching factor (assumed)

Steps in the design procedure are as follows:

1. Determine the positive phase duration for the equivalent triangular air blast impulse from Figure 2-4, as suggested in Section 2-2.1.

2. Determine the tank's natural period of vibration in the compressional mode by Equation 2-20:

$$T_t = \frac{\pi D}{C_t}$$

3. Calculate the dynamic amplification factor using Equation 2-19:

$$DF = \frac{1}{\dfrac{T_t}{\pi t_d}\sqrt{2\mu - 1} + \dfrac{1 - \dfrac{1}{2\mu}}{1 + .7\,\dfrac{T_t}{t_d}}}$$

Substitute t_i, t_{50} and t_∞ into the equation for t_d and select the largest dynamic amplification factor. For an elastic system, $\mu = 1$, and the maximum dynamic amplification value which could possibly be obtained is two.

4. Calculate the pressure attenuation factor, α_z, by Equation 2-2a or 2-2b:

$$\alpha_z = \frac{1}{1 + \dfrac{z}{L_w}} \qquad \text{for } p_o < 1000 \text{ psi}$$

$$\alpha_z = \left(\frac{1}{1 + \dfrac{z}{L_w}}\right)\left(\frac{27.5}{p_o^{0.48}}\right) \qquad \text{for } p_o > 1000 \text{ psi}$$

If the crown level is above the water table calculate α_z at the crown level of the tank. If the crown level is below the water table, calculate α_z at the water table.

5. If more than 3/4 of the tank diameter is below the water table, calculate the pressure amplification factor by Equation 2-5:

$$p_{zw} = \left[2\left(\frac{\rho_2 C_2}{\rho_1 C_1 + \rho_2 C_2}\right)\right] \alpha_z \, p_o$$

If less than 3/4 of the tank diameter is below the water table no pressure amplification factor needs to be considered.

6. Compute the arching factor, A, from Equation 2-7.

a. Guess at a tank thickness or use Figure 1-2 tc get an approximate value.

b. Compute the expotential term in Equation 2-17:

$$\alpha = \text{Ln} \frac{1.5\, M_c}{E_t I/D^3}$$

c. Compute the arching exponent in Equation 2-18:

$$\phi = \frac{4d_o(\pi D - 2D + 2L)}{(\pi D^2 + 4DL - 4D^2)} \cdot (1 - e^{-0.024\alpha^2})$$

d. Compute the right hand side of the arching function in Equation 2-15:

$$e^{-n\phi} = e^{-0.135\phi}$$

e. Compute the active arching factor, A_o, from Equation 2-16:

$$A_o = \text{Tan}\ \phi_o$$

f. Enter Figure 2-9 with the value for $e^{-n\phi}$ and A_o and read out, A.

7. Compute the dead load pressure acting on the crown of the tank, including the arching effects -

With no water table present:

$$p_{DL} = (1 - A)\gamma_s d_o$$

For the case with water table:

$$p_{DL} = (1 - A)\gamma_s z_w + (1 - A)(\gamma_{sw} - \gamma_w)(d_o - z_w) + \gamma_w(d_o - z_w)$$

where, γ_w is the unit weight of water.

8. Calculate the total effective pressure acting at the crown of the tank due to all external loads. Neglect internal pressure.

For the case without water table above the tank:

$$p_e = DF\,(1-A)\alpha_z p_o + p_{DL}$$

For the case with water table above the tank:

$$p_e = DF\left[2\left(\frac{\rho_2 C_2}{\rho_1 C_1 + \rho_2 C_2}\right)\right](1-A)\alpha_z p_o + p_{DL}$$

9. Calculate the internal pressure, using a safety factor of 2.0:

$$p_i = \frac{1}{3} p_e$$

10. Determine the critical yield pressure on the tank:

$$p_{cr} = p_e - p_i$$

11. Determine the required tank thickness to resist, p_{cr}, from Equation 2-25:

$$t = \frac{p_{cr} D}{2.54\sigma_y} = \frac{(p_e - p_i) D}{2.54\sigma_y}$$

12. If the computed tank thickness is the same as the assumed value used in calculating the arching factor, A, no further work is required. If not, use computed tank thickness and repeat steps 6,7,8,9,10 and 11 as many times as required to make the solution converge.

13. Check the critical buckling pressure of the tank using Equation 2-26:

$$p_{cr} = 0.621 \sqrt{\frac{E_s E_t}{(r/t)^3}} = 4.5 \sqrt{M_c \frac{E_t I}{D^3}}$$

14. If the tank has been designed for an overpressure level under 200 psi, the final selection of thickness may be controlled by a combination of non-blast induced loads. Check the combined influence of dead load, construction equipment loads, normal vehicular loads, and any other surface loading which might be present.

To further simplify the design process, a computer program has been written which carries out each of the steps, 1 through 12. It also checks the buoyant uplift force produced on the empty tank by high water table and gives the required hold-down forces. A general flow diagram of the program along with example solutions are presented in Appendix A. This program is available through the U.S. Naval Civil Engineering Laboratory, Port Hueneme, California.

2-7.2 Discussion of Design Procedure

The design procedure, as presented, is based on several conservative assumptions. First, it is assumed that the maximum resistance of the tank section is controlled by the yield stress of the material. In actual fact the tank can resist external loads which produce plastic yielding and even buckling of the cylinder section, without rupturing. [6] Yield stress was selected as the design limit in order to insure no loss of function under repeated nuclear air blast.

Secondly, dynamic increase in the strength characteristic of the tank was neglected. For mild grade steels the increase in yield strength can range from 20% to 40%. For high strength steels, which are strain hardened, the increase generally is negligible.

Thirdly, only partial advantage has been made of internal pressurization. The safety factor of 2.0, limits the internal pressure to 1/3 the total effective external pressure acting on the tank. By reducing the safety factor to 1.5, internal pressurization could be increased, and the required tank thickness would be reduced by about 20%.

Dynamic forces produced by the sloshing action of liquid fuel stored in the tank have not been included in the design procedure. Experimental evidence and analytical verification indicate that the natural period for liquid sloshing is very long in comparison with the fundamental period of the tank in its compressional mode. Thus, peak response of the tank to blast pressure will occur long before internal loads due to liquid sloshing are developed. In addition, the inertia loads generated by the sloshing liquid are small in magnitude compared to the externally applied soil pressures.

The design procedure outlined in this section is based on two loading conditions, the dynamic external blast pressure and the static internal air pressure. For the dynamic loading no safety factor is considered other than the reserve capacity of the tank section above yield stress. For internal pressure, a safety factor of 2.0 has been suggested. This margin of safety is on the low side when compared to the 2.5 which is normally specified between allowable design stresses and yield stress in the ASME Boiler and Pressure Vessel Code - Section VIII - Division 1. The fact that the tanks are buried tends to minimize the hazards resulting from accidental rupture under static internal pressure. In addition, the primary purpose for the installation will be that of fulfilling a military function. These two factors tend to justify the lower safety factor suggested. However, the designer should exercise his own judgment in establishing a safety factor which he feels is consistent with the unique features and conditions at the site under design.

The design and fabrication of the vessel should follow the ASME Code recommendations as closely as possible. Particular attention should be given to the design of pipe connections and manholes to the tank, so that proper pressure vessel details are used in the nozzle geometry and weld procedures.

2-7.3 Backfill Requirements

The design procedure which has been presented is valid only if granular material is used as backfill around the tank. It is recommended that construction specifications require backfill with material placed at a dry density corresponding to 95% of the standard Proctor maximum dry density.

The specified thickness of dense granular backfill should depend on the characteristics of the adjacent soil field and should be specified by a competent soils engineer. It is recommended that a minimum thickness equal to 20% of the tank diameter be used all around the tank.

2-7.4 Design for Special Soil Conditions

Because of the erratic nature of measured forces in tanks and cylinders buried in clayey or silty soil, it is recommended that any tank installation in such material be backfilled with dry dense granular soil. The backfill must extend below the bottom of the tank to be effective.

For rock sites, over cut the excavation in length, width, and depth to allow for a generous layer of dense granular backfill, which should be placed around the entire tank. Provide a concrete cap slab as shown in Figure 1-4 to prevent the tank from rebounding out of the ground during nuclear blast loading.

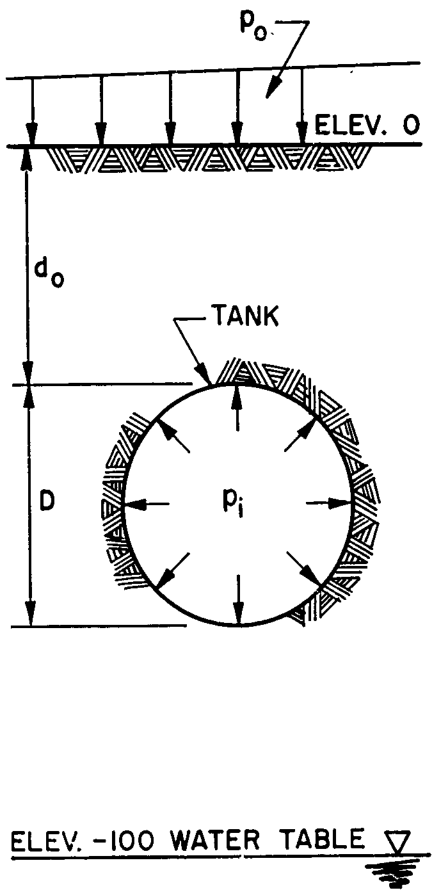
High water table is another site condition which will require special design considerations, as noted in Section 2-2.5. Dewatering for excavation will be a difficult, if not impossible task in coarse grained materials because of the large potential flows. In fine silty sands dewatering might be economically feasible with the use of well points. As noted earlier, every effort should be taken to place the buried tank above the water table. Under high water table conditions, the use of several small diameter tanks may be more economical than one or two large diameter tanks if the smaller tanks can be placed above the water table.

For sites which combine high water table conditions with loosely compacted granular soils, the potential of soil liquefaction due to ground motion generated by nuclear blast or earthquake should be carefully evaluated. If the tanks were to be placed below the water table under these site conditions, positive measures should be taken to preconsolidate the soils through the use of vibro-flotation techniques. It is recommended that a positive hold-down force be included in the design of the tank. This could be provided by concrete encasement of the tank or by rock anchor techniques.

2-8 Design Examples

Two design examples are presented in this section to demonstrate the procedure outlined in Section 2-7. Both examples deal with a 16 foot diameter tank buried one diameter in granular soil. The weapon yield is 5 M.T. and the overpressure level is 1500 psi. The distinguishing difference between the two cases is the location of the water table. In Example No. 1 it is 100 below the ground surface, whereas in Example No. 2 it is 5 feet below the surface.

Design Example No. 1



D = 16 ft. I.D.

L = 2D

d_o = D = depth of burial

z_w = 100 ft.

σ_y = 90,000 psi, static

E_t = 30×10^6 psi

ν = 0.25

γ_t = 490 lbs/cu.-ft.

E_s = 33,000 psi

γ_s = 110 lbs/cu.-ft.

γ_{sw} = 130 lbs/cu.-ft.

ϕ_o = 37°

p_o = 1500 psi

W = 5 M.T.

1. Determine t_i, t_{50}, and t_∞ from Figure 2-4 -

$$t_i = 0.37 \text{ sec. } \left[\frac{100 \text{ psi}}{1500 \text{ psi}}\right]^{1/2} \left[\frac{5 \text{ MT}}{1 \text{ MT}}\right]^{1/3} = 0.164 \text{ sec.}$$

$$t_{50} = 0.0195 \text{ sec. } \left[\frac{5 \text{ MT}}{1 \text{ MT}}\right]^{1/3} = 0.033 \text{ sec.}$$

$$t_\infty = 0.01 \text{ sec.} \left[\frac{5 \text{ MT}}{1 \text{ MT}}\right]^{1/3} = 0.017 \text{ sec.}$$

2. Determine period of tank vibration in compressional mode -

$$C_t = 18{,}470 \text{ fps}$$

$$T_t = \frac{\pi\ (16 \text{ ft.})}{18{,}470 \text{ fps}} = 0.0027 \text{ sec.}$$

3. Calculate dynamic amplification factor -

Let $\mu = 1$, linear-elastic system

For $\dfrac{T_t}{t_d} = \dfrac{T_t}{t_i} = \dfrac{0.0027}{0.164}$

$$DF = \frac{1}{\dfrac{.0027}{\pi\ (.167)} + \dfrac{1/2}{1 + .7\left(\dfrac{.0027}{.167}\right)}} = 2.0$$

This is the maximum value which DF can have in an elastic system so there is no need to consider t_{50} or t_∞.

4. Calculate the pressure attenuation factor, α_z, at the crown -

$$C_s = \sqrt{\frac{E_s}{\rho_s}} = \sqrt{\frac{g\,E_s}{\gamma_s}}$$

$$C_s = \sqrt{\frac{(32.2 \text{ ft/sec}^2)(33{,}000 \text{ psi})(144 \text{ in}^2/\text{ft}^2)}{(110 \text{ pcf})}}$$

$$C_s = 1180 \text{ fps}$$

$$L_w = 230 \text{ ft.} \left(\frac{100 \text{ psi}}{1500 \text{ psi}}\right)^{1/2} \left(\frac{5 \text{ MT}}{1 \text{ MT}}\right)^{1/3} \left(\frac{1180 \text{ fps}}{622 \text{ fps}}\right)$$

$$L_w = 193 \text{ ft.}$$

For p_o greater than 1000 psi use Equation 2-2b

$$\alpha_z = \frac{1}{1 + \frac{z}{L_w}} \cdot \frac{27.5}{p_o^{0.48}} = \frac{1}{\left(1 + \frac{16 \text{ ft}}{193 \text{ ft}}\right)} \cdot \frac{27.5}{(1500)^{0.48}}$$

$$\alpha_z = 0.759$$

5. Calculate pressure amplification factor due to water table. In this case the water table is 100 feet below ground surface. Therefore no stress amplification will occur.

6. Compute arching factor, A -

From Figure 1-2 select a trial tank thickness of 1.00 inch.

$$\alpha = \text{Ln}\left(\frac{1.5 M_c}{E_t I/D^3}\right)$$

$$\alpha = \text{Ln} \frac{1.5(33{,}000 \text{ psi})(16 \text{ ft x } 12\text{in/ft})^3}{(30 \text{ x } 10^6 \text{ psi})(1 \text{ in}^3/12)(0.577)} = 12.40$$

$$\phi = \frac{4d_o(\pi D - 2D + 2L)}{(\pi D^2 + 4DL - 4D^2)} (1 - e^{-0.024\alpha^2})$$

for $L = 2D$

$$\phi = \frac{4d_o(\pi + 2)}{D(\pi + 4)} (1 - e^{-0.024\alpha^2})$$

$$\phi = \frac{4(16\ ft)(\pi + 2)}{(16\ ft)(\pi + 4)} \left(1 - e^{-0.024(12.40)^2}\right)$$

$$\phi = 2.81$$

$$e^{-n\phi} = e^{-0.135(2.81)} = 0.684$$

$$A_o = \text{Tan } 37° = 0.7535$$

Enter Figure 2-9 with these values for $e^{-n\phi}$ and A_o and read, A = 0.315.

Thus, (1-A) = 0.685

7. Calculate dead load of soil -

$$p_{DL} = (1 - A)\ \gamma_s d_o$$

$$= \frac{0.685\ (110\ pcf)(16\ ft)}{144\ in^2/ft^2}$$

$$p_{DL} = 8.37\ psi$$

8. Calculate the total effective external pressure -

$$p_e = DF\ (1-A)\alpha_z p_o + p_{DL}$$

$$= 2.0(0.685)(0.759)(1500\ psi) + 8.37\ psi$$

$$p_e = 1562\ psi + 8.37\ psi = 1570\ psi$$

9. Determine internal pressure -

$$p_i = \frac{1570\ psi}{3} = 523.5\ psi$$

10. Calculate critical yield pressure -

$$p_{cr} = 1570 \text{ psi} - 523.5 \text{ psi} = 1056 \text{ psi}$$

11. Determine tank thickness -

$$t = \frac{p_{cr}D}{2.31\ \sigma_y} = \frac{(1056 \text{ psi})(16\text{ft} \times 12\text{in/ft})}{2.31\ (90{,}000 \text{ psi})} = .967 \text{ in.}$$

12. The closest rolled plate thickness is 1 inch, so use 1 inch plate. There is no need to go through another cycle of iteration since the arching factor is based on an assumed tank thickness of 1 inch.

13. Check for elastic buckling -

$$p_{cr} = 0.621 \sqrt{\frac{E_s E_t}{(r/t)^3}}$$

$$= 0.621 \sqrt{\frac{(33{,}000\text{psi})(30\text{x}10^6\text{psi})(1 \text{ in})^3}{(8 \text{ ft} \times 12\text{in/ft})^3}} = 657 \text{ psi}$$

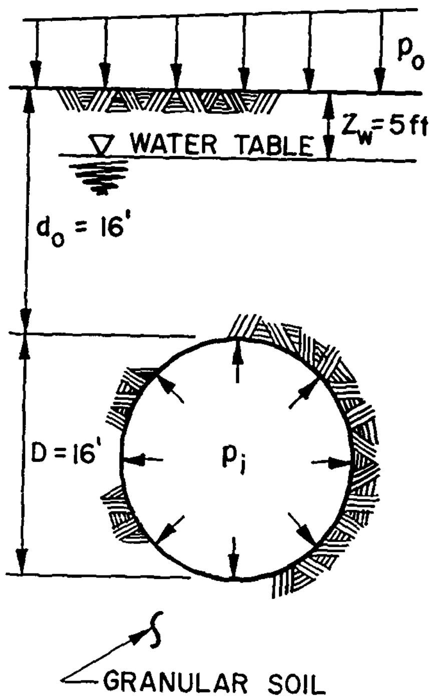
Thus, the critical pressure for elastic buckling is less than the net external pressure of 1056 psi calculated in Step #10. Based on these results, the tank should buckle elastically. However, the buckling equation is very conservative in this case. It was derived for long thin-walled cylinders without stiffening end caps. The actual tank under analysis has a short cylindrical section and is stiffened by the two spherical end caps. Static tests on buried model tanks, internally pressurized [6] indicate the buckling resistance of the tank to be three times as large as the value given by Equation 2-26.

Using these test results as a guide, the critical buckling pressure would be predicted as -

$$p_{cr} = 3\ (657 \text{ psi}) = 1970 \text{ psi}$$

which is well above the design pressure of 1056 psi.

Design Example No. 2



$D = 16$ ft.

$L = 2D$

$d_o = D$

$z_w = 5$ ft.

$\sigma_y = 90{,}000$ psi

$E_t = 30 \times 10^6$ psi

$\nu = 0.25$

$\gamma_t = 490$ lbs/cu.-ft.

$E_s = 95{,}000$ psi

$\gamma_s = 110$ lbs/cu.-ft.

$\gamma_{sw} = 130$ lbs/cu.-ft.

$\phi_o = 41°$

$p_o = 1500$ psi

$W = 5$ M.T.

1. Determine t_i, t_{50} and t_∞ from Figure 2-4

$t_i = 0.164$ sec.

$t_{50} = 0.033$ sec.

$t_\infty = 0.017$ sec.

2. Determine period of tank vibration in compressional mode -

$$C_t = 18{,}470 \text{ fps}$$

$$T_t = \frac{\pi(16 \text{ ft})}{18{,}470 \text{ fps}} = 0.0027 \text{ sec.}$$

3. Calculate dynamic amplification factor - this will be the same as in Example No. 1

$$DF = 2.0$$

4. Calculate the pressure attenuation factor, α_z, at the water table -

$$C_s = \sqrt{\frac{(32.2 \text{ ft/sec}^2)(95{,}000 \text{ psi})(144 \text{ in}^2/\text{ft}^2)}{(110 \text{ pcf})}}$$

$$C_s = 2000 \text{ fps} \quad \text{(for soil above the water table)}$$

$$L_w = 230 \text{ ft.} \left(\frac{100 \text{ psi}}{1500 \text{ psi}}\right)^{1/2} \left(\frac{5 \text{ MT}}{1 \text{ MT}}\right)^{1/3} \left(\frac{2000 \text{ fps}}{622 \text{ fps}}\right)$$

$$L_w = 327 \text{ ft.}$$

$$\alpha_z = \frac{1}{\left(1 + \frac{16 \text{ ft}}{327 \text{ ft}}\right)} \cdot \frac{27.5}{(1500)^{0.48}} = 0.810$$

This is the pressure attenuation down to the water table.

5. Determine the pressure amplification factor at the water table -

$$2\left(\frac{\rho_2 C_2}{\rho_1 C_1 + \rho_2 C_2}\right) = 2\left(\frac{\gamma_2 C_2}{\gamma_1 C_1 + \gamma_2 C_2}\right)$$

$$= 2\left[\frac{(130\text{ pcf})(5100\text{ fps})}{(110\text{ pcf})(2000\text{ fps}) + (130\text{ pcf})\ (5100\text{ fps})}\right]$$

$$= 1.5$$

6. Compute arching factor, A -

Assume a plate thickness of 1.5 inches

$$\alpha = \text{Ln}\left[\frac{1.5\ M_c}{E_t I/D^3}\right]$$

$$= \text{Ln}\left[\frac{1.5(95{,}000\text{ psi})(16\text{ ft x }12\text{in/ft})^3(12)}{(30 \times 10^6\text{ psi})(.282\text{ in})^3(0.577)}\right]$$

$$\alpha = \text{Ln}\ 2.48 \times 10^5 = 12.42$$

$$\phi = \frac{4d_o(\pi + 2)}{D(\pi + 4)}\ (1 - e^{-0.024\alpha^2})$$

$$= \frac{4(16\text{ ft})(\pi + 2)}{(16\text{ ft})(\pi + 4)}\ (1 - e^{-0.024(12.42)^2})$$

$$\phi = 2.81$$

$$e^{-n\phi} = e^{-0.135(2.81)} = 0.685$$

$$A_o = \text{Tan } 41° = 0.87$$

Enter Figure 2-9 with this value and read, A = .401

Thus, (1-A) = 0.599

7. Calculate dead load of soil -

$$p_{DL} = (1 - A)\gamma_s z_w + (1 - A)(\gamma_{sw} - \gamma_w)(d_o - z_w) + \gamma_w(d_o - z_w)$$

$$= \frac{(0.599)(110\ \text{pcf})(5\ \text{ft})}{(144\ \text{in}^2/\text{ft}^2)} + \frac{(0.599)(130\ \text{pcf} - 62.4\ \text{pcf})(16\text{ft} - 5\text{ft})}{(144\ \text{in}^2/\text{ft}^2)} + \frac{(62.4\ \text{pcf})(16\ \text{ft} - 5\ \text{ft})}{(144\ \text{in}^2/\text{ft}^2)}$$

$$p_{DL} = 2.30\ \text{psi} + 3.12\ \text{psi} + 4.74\ \text{psi} = 10.16\ \text{psi}$$

8. Calculate the total effective external pressure -

$$p_e = DF \quad 2\left[\left(\frac{\rho_2 C_2}{\rho_1 C_1 + \rho_2 C_2}\right)\right] (1 - A)\alpha_z p_o + p_{DL}$$

$$= (2.0)(1.5)(0.599)(0.810)(1500\ \text{psi}) + 10.16\ \text{psi}$$

$$p_e = 2192 + 10.16 = 2202\ \text{psi}$$

9. Determine internal pressure -

$$p_i = \frac{2202}{3} = 734\ \text{psi}$$

10. Calculate critical yield pressure -

$$p_{cr} = 2202 \text{ psi} - 734 \text{ psi} = 1468 \text{ psi}$$

11. Determine tank thickness -

$$t = \frac{p_{cr} D}{2.31\ \sigma_y} = \frac{(1468 \text{ psi})(16 \text{ ft.} \times 12 \text{ in.})}{2.31\ (90{,}000 \text{ psi})}$$

$$t = 1.36 \text{ inches}$$

12. The closest rolled plate thickness is 1.375 inches, so use this size. The assumed plate thickness of 1.5 inches used in computing the arching coefficient is not sufficiently different from the final calculated thickness to warrant a further round of analysis.

13. Check for elastic buckling -

$$p_{cr} = 0.621 \sqrt{\frac{E_s E_t}{(r/t)^3}}$$

$$= 0.621 \sqrt{\frac{(95{,}000 \text{ psi})(30 \times 10^6 \text{ psi})(1.375 \text{ in})^3}{(8 \text{ ft} \times 12 \text{ in/ft})^3}}$$

$$p_{cr} = 1{,}797 \text{ psi}$$

The critical pressure for elastic buckling is greater than the net external pressure of 1468 psi, calculated in Step #10. Thus, buckling will not be critical in this case.

14. Check buoyant uplift force -

Buoyant uplift on empty tank,

$$B = \frac{10\pi D^3 \gamma_w}{24}$$

$$= \frac{10\pi \, (16 \text{ ft})^3 (62.4 \text{ lbs/cu ft})}{24}$$

$$B = 334{,}567 \text{ lbs.}$$

Dead weight of tank	=	114,940 lbs.
Dead weight of soil	=	1,370,000 lbs.
Total dead load	=	1,484,940 lbs.

Safety factor against buoyant uplift,

$$S.F. = \frac{1{,}484{,}940}{334{,}567} = 4.45$$

Thus, buoyancy will not be a problem once the tank is covered with 16 feet of backfill. If soil instability due to the high water table is a potential problem, positive measures should be taken to tie the tank down. Otherwise it will float to the surface as soil liquefaction occurs due to dynamic loading from nuclear air blast or ground shock.

In comparing the design thickness for the tank which was calculated in Example 1 with that from Example 2, the primary cause for increased thickness was due to the presence of high water table. The resulting pressure amplification of 1.5 forced the design thickness to increase 50% over the size in Example 1. The stiffer soil condition in Example 2 produced a slightly larger arching factor, A, which reduced the effective pressure on the tank. This reduction in design pressure partially offset the influence of high water table and resulted in a net increase in tank thickness of 37.5%.

2-9 Corrosion Protection

For internally pressurized steel tanks, corrosion protection is a very important design consideration. Failure of an internally pressurized tank due to corrosion represents a greater hazard than the corrosion of unpressurized fuel tanks. The potential of stress corrosion is also enchanced by internal pressurization which places the tank in a state of moderately high working stress throughout its life.

There are many factors which affect the corrosive nature of the soil in which the tanks are buried. An understanding of some of the more important factors will help the designer select the proper backfill material. The following are the primary factors affecting soil corrosivity [5] :

1. Soil Texture - Fine, even-textured soils are less corrosive than coarse soils. Soils of uniform composition, such as sand or the sandy or silty loams, are less corrosive than a mixture of soils, such as sand and clay.

2. Soil Acidity - Highly acid soils are corrosive, while mildly acid soils are not necessarily corrosive.

3. Organic Matter - Soils low in organic matter, such as clean sands, are usually less corrosive than soils high in organic matter.

4. Bacterial Content - Soils with anaerobic bacteria (usually heavy, water-logged soils, such as clay) are very corrosive.

5. Electrical Resistivity - Low resistivity soils (0 to 3000 Ohm-cm) are corrosive because their high conductivity allows an easy path for the flow of current from anodic areas to cathodic areas on the structure.

6. Moisture Content - Wet soils are usually more corrosive than dry soils.

Dry, clean sand without soluble salt content is one of the best non-corrosive soil conditions and forms an excellent backfill material. It is recommended that a positive form of corrosion protection be provided in addition to a non-corrosive soil. The most economical method is the use of a good quality commerical coating on the outside of the tank to reduce the exposed metal area to a minimum and thus isolate the metal from the potentially corrosive environment of the soil. In conjunction with the coating, cathodic protection should be provided to prevent corrosion at discontinuities or faults in the coating material.

Every effort should be made to avoid dissimilar metal couples, such as a brass or copper pipe coupling in combination with steel. All piping should be coated and wrapped.

If the corrosive environment is removed from the exterior surface of the tank by providing a coating barrier and cathodic protection, stress corrosion will have effectively been prevented. There are additional steps which may be taken to insure against stress corrosion. A full anneal and slow cool of the fabricated tank is a dependable means of preventing stress-corrosion cracking. For high strength, cold worked steels, a full anneal is not always permissible because it lowers the strength properties of the metal.

2-10 Mechanical Consideration for Internal Pressurization

The use of internal pressurization in a buried fuel tank presents mechanical design situations which would not normally be considered in the case of unpressurized tanks. Allowance must be made for depressurization of the tank at times of refueling, cleaning and inspection. During operation the high internal pressure will force fuel out of the tank down the fuel supply line to the engines. Thus, a pumping system will not be required. However, in its place a compressor must be substituted which will provide the internal pressure. Pressure reduction valves must be installed on the fuel supply lines to drop the fuel pressure to atmospheric level before it is supplied to the engine.

These and other mechanical considerations are presented in this section of the report.

2-10.1 Compressor

To provide internal pressure to the fuel tanks at pressures up to 1000 psi, an Ingersoll-Rand compressor (Model 15T2X) or equivalent will serve the purpose economically. Because of possible down time due to repairs and maintenance, two of these compressors should be provided at each site. The cost of a single compressor with high operating reliability which would insure continuous internal pressurization is greater than the cost of two compressors of the type recommended.

In order to insure resistance of the fuel storage tanks to repeated nuclear air blast, the compressor system must survive nuclear attack in an operable manner. Thus, the compressors should be placed in the hardened shelter which the fuel tanks supply. Each compressor occupies a space 58 inches in length, 28 inches in width, and 33 inches in depth.

2-10.2 Piping

Nuclear blast pressures will cause large relative displacements in the soil as the pressure front moves over the soil. Prediction equations are available [8] for calculating the probable maximum relative displacement which would be expected between adjacent tanks for any given soil condition, overpressure level, and weapon yield. With this information the designer can provide flexibility in the pipe system to meet the relative displacement criteria.

Particular attention must be given to the design of pipe attachments at the pressurized tank. Due to relative motion of the soil adjacent to the tank under arching action, large shear forces are developed at the tank-to-soil interface along the side walls of the vessel. Any pipe connection made in this zone will experience these high shearing forces and should be isolated from the soil through use of a crushable wrapping or isolation sleeve. [15,16]

If flexible piping is used between tanks, the stiffening influence of internal pressure should be accounted for in the design of the flex-hose.

2-10.3 Valves

Three separate valves must be provided in the pressurized tank system.

In the fuel supply line to the engines a pressure reduction valve will be required to reduce the high internal pressure in the tank system to atmospheric level or slightly higher depending on the head losses in the line.

Because of potential rupture of a single line or premature failure of a tank due to undetected weaknesses in material a system of pressure closure valves should be provided at each tank-to-pipe connection. These could be quick closing solenoid valves, activated by a pressure switch in the pipeline which respond to a sudden pressure drop. Thus, failure at one point in the tank system would not weaken the entire system to subsequent nuclear attack.

A system of pressure relief valves will be required in order to depressurize a portion of the tank system for refueling, cleaning, and inspection.

2-10.4 Access

A special manhole will be required in the top of each tank with access passages to the surface. The manhole must be pressure tight and be easily opened from outside the tank.

2-11 Site Investigation

Throughout this report every effort has been made to emphasize the important part which the soil medium surrounding the tank plays in its design. The tank derives much of its strength and protection from a favorable soil environment. Thus, it is only appropriate that attention be given to the particular soil properties needed by the designer to properly evaluate the site-soil conditions and the improvements which may be required to upgrade the soil immediately surrounding the tank.

General information on the site-soil conditions such as thickness of overburden, uniformity of soil layers, depth to bedrock and water table is basic to the design. For construction purposes information about soil stability during excavation, the availability of select backfill, and the anticipated settlement is required. For sites with high water table, data on the permeability and draw down characteristics must be established in order to design dewatering procedures.

Specific material properties should be determined for each soil type encountered at the site or used as backfill. The following list contains the properties most useful to the designer:

1. Elastic secant modulus, E_s, (static and dynamic)
2. Shear modulus (dynamic)
3. Confined and triaxial stress-strain relationships, (static and dynamic)
4. Poisson's Ration, ν
5. Propagational or compressional wave velocity, C_s
6. In situ unit weight
7. Saturated unit weight
8. Void ratio
9. Relative density
10. Bearing capacity
11. Shear strength
12. Angle of internal friction, ϕ_o
13. Electric resistivity
14. Corrosion potential

Items 1, 4, 5, 6, 7, and 12 are directly applicable to the design procedure outlined in Section 2-7. If a more refined analysis should be required using dynamic finite element techniques, items 2 and 3 may also be required.

The bearing capacity and shear strength of the soil is of direct use in designing for construction loads. This information is also applicable to the foundation design for support facilities which may be located at the site.

The void ratio and relative density are field measurable properties used to control the placing of backfill around the tanks.

Appendix C has been prepared as a guide in specifying and reviewing soil investigation procedures. The appendix deals with siting studies, geological exploration, sub-surface exploratory drilling and sampling, geophysical surveys, laboratory testing, and final report recommendations.

Section 3

OPTIMIZATION STUDY

3-1 Introduction

An economic study was made to determine the optimum tank volume at various overpressure levels. Parametric variables in the study included the depth to water table, internal pressure, and yield strength of the tank steel. In order to optimize the tank size on an economical basis, realistic cost figures were obtained from tank fabricators and major construction contractors who were located near the area of proposed construction.

A computer program was written to carry out a series of analyses for each parametric case. Results are presented in graphical form for direct interpretation and application by the designer.

3-2 Cost Factors

The major costs of a buried fuel storage tank are in the tank material, excavation and backfill. Each of these costs is directly dependent on the tank diameter or volume as will be shown. Additional costs result from internal pressurization, piping and maintenance. These are minor in nature and are directly dependent on the total storage volume at the site rather than the volume of a given tank.

3-2.1 Tank Costs

Tank costs may be broken down into the following basic items:

1. Material
2. Fabrication
3. Transportation
4. Placing
5. Corrosion Protection

All of these costs except transportation are directly dependent on the volume of the tank. Transportation costs are primarily dependent on the route of travel not on the size of the tank. Thus for cost comparison purposes, the transportation costs from fabricator to job site, have been excluded from the total tank costs.

There is a limitation on the maximum size of shop fabricated tanks which can be hauled by truck or train. By truck the maximum diameter is 10 feet and by rail it is 12 feet. Tanks of greater diameter must be transported in segments and field fabricated at the site.

To establish realistic values for the total cost of a fabricated tank, a survey was conducted of the major fabricators. Each was presented with information regarding the range of possible tank sizes, the material type (high strength steel with a yield stress of 90 to 100 ksi) and fabrication criteria (ASME Pressure Vessel Code). Both field and shop fabrication were considered. Results from the survey were plotted and two cost equations were established (See Figure 3-1). The equations were deliberately fitted to the high side of the data in order to be conservative.

As can be seen from the results, the minimum cost occurs at plate thickness of 1½ inches. Plates of this thickness are ideal for bending, require the minimum amount of jigging and are readily welded. For thinner or thicker plates the fabrication costs increase due to the above factors. The cost differential between shop and field fabrication is roughly 25 cents per pound.

3-2.2 Excavation Costs

Excavation costs can range widely due to the variation in anticipated soil conditions which range from hardrock to nearly impregnable swamps. In order to establish an estimate of the probable cost for excavation and shoring, three soil conditions were considered, rock, sand, and clay. The cost of dewatering was not included in the excavation costs.

For rock the most economical excavation is a vertical pit. Estimated excavation costs range from $15 per cubic yard to $45 per cubic yard. In sand and clay soils vertical excavation using driven piles and sheathing to hold back the soil appears to be the most economical method. The cost of excavation and shoring will vary with depth of total excavation as shown by the cost equaof Figure 3-2. Costs increase at a rate of 15 cents per

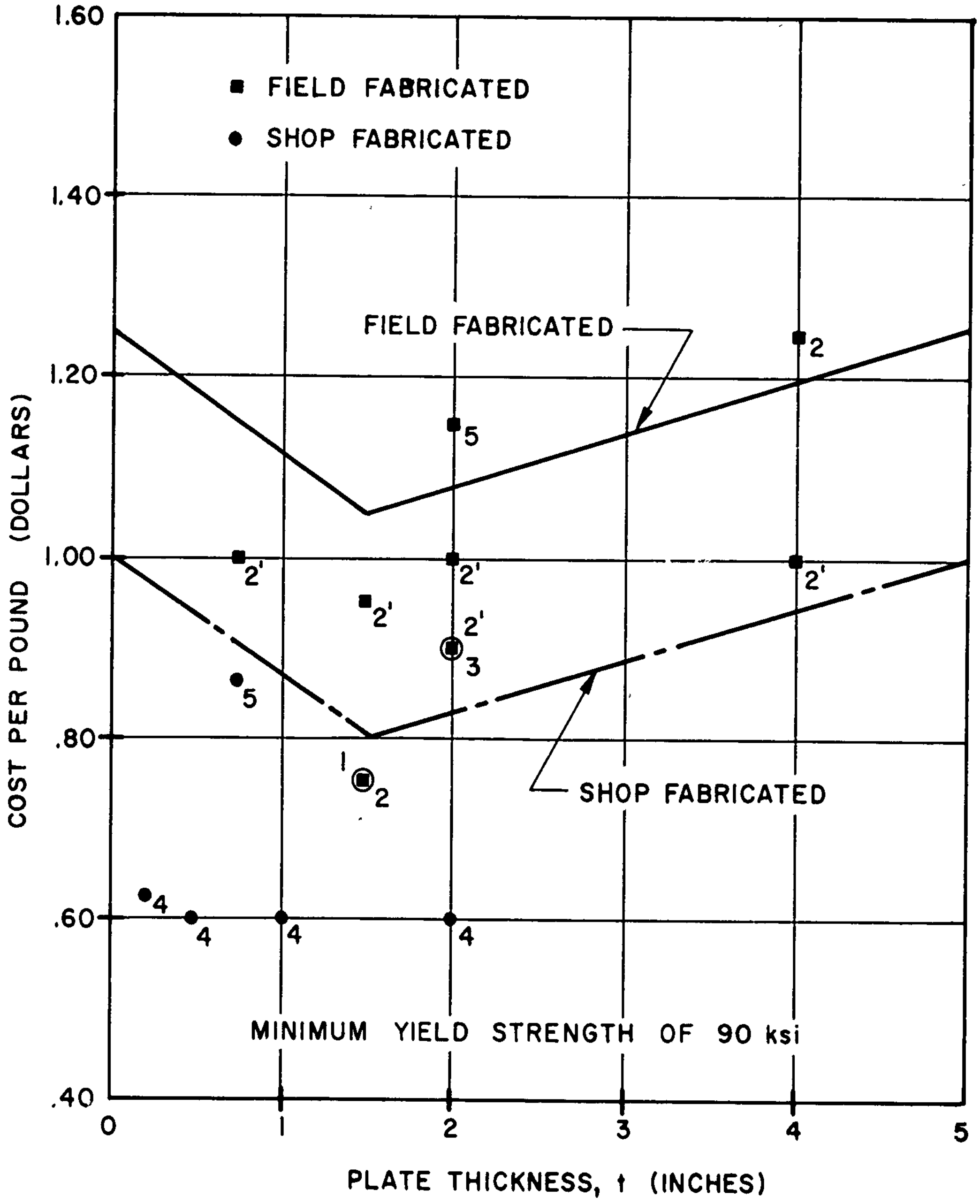


Figure 3-1 TOTAL COST PER POUND OF STEEL PLATE-INCLUDING MATERIAL, FABRICATION, PLACING, AND CORROSION PROTECTION

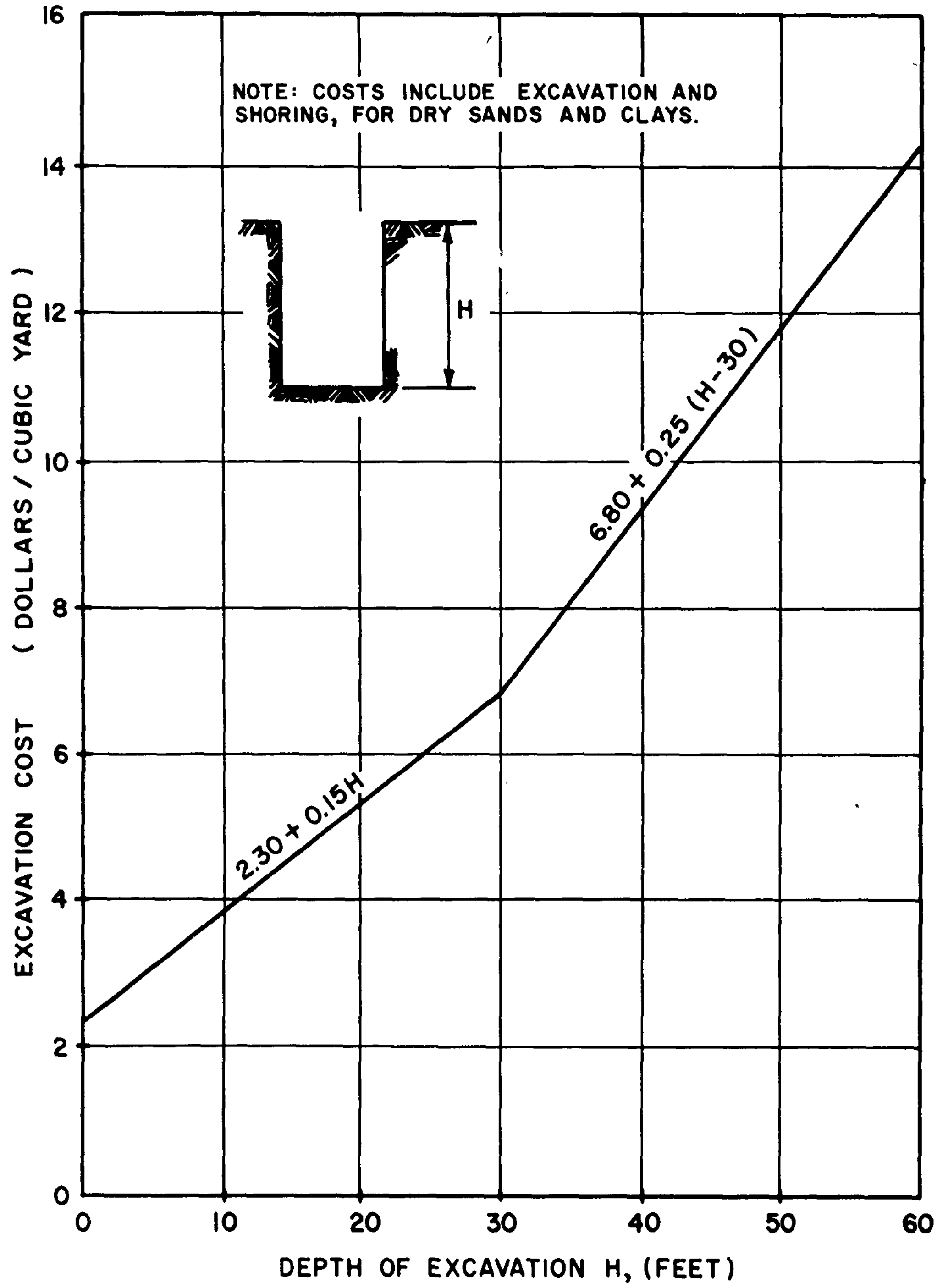


Figure 3-2 EXCAVATION COSTS VS. DEPTH OF TOTAL EXCAVATION

foot for excavations up to 30 feet in depth. Below this depth, costs increase at a rate of 25 cents per foot.

At sites with high water table, dewatering will be necessary in order to excavate. The cost is directly dependent on the depth of water table, depth of excavation, and soil permeability. For gravelly soils the permeability is so high that dewatering may not be economically feasible; in clayey soils dewatering will normally be a simple process. Because of the potentially wide range in dewatering costs from site to site, these costs have been omitted from the optimization study.

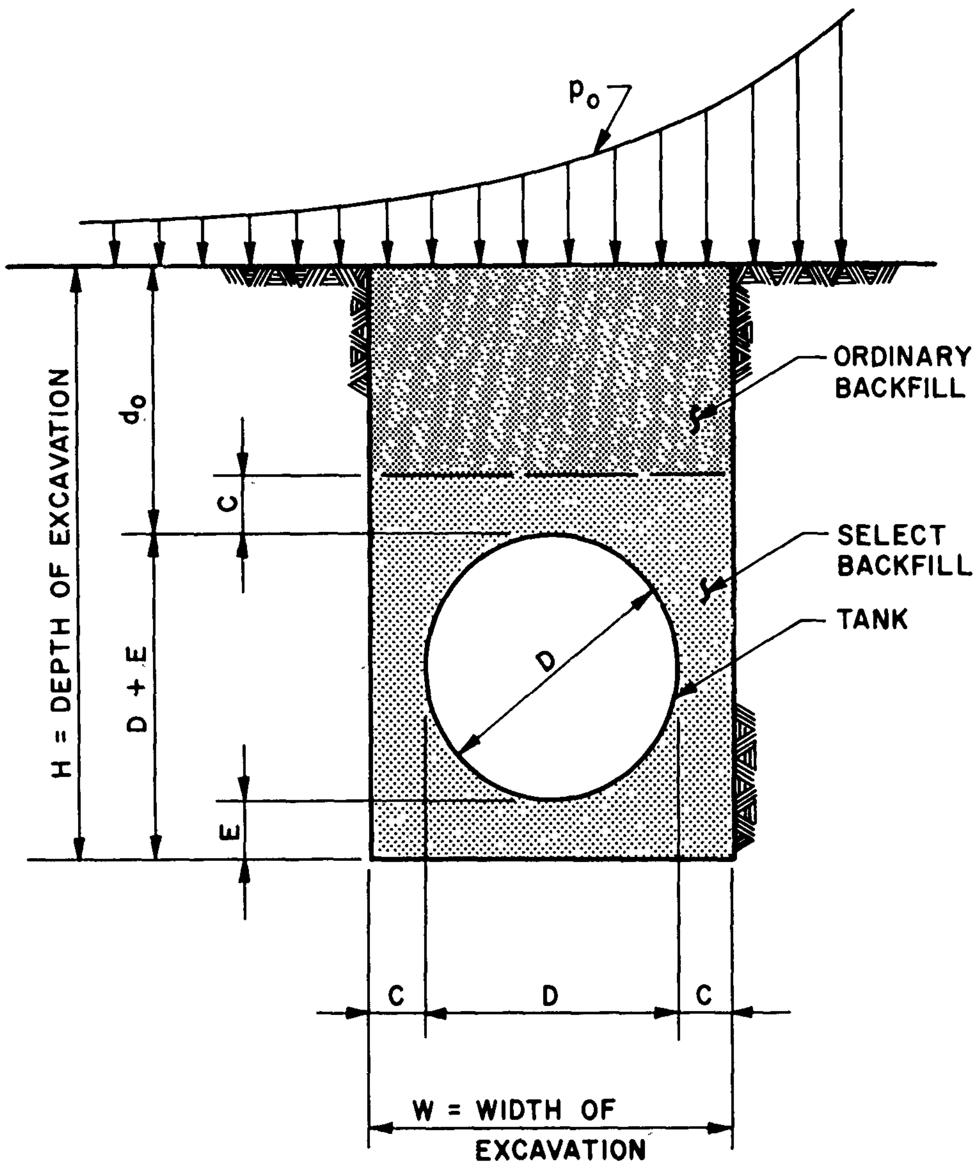
3-2.3 Backfill Costs

Select backfill material will normally be placed around the entire tank (See Figure 3-3). Washed or dredged sand is ideal for this purpose because of its low corrosion potential and uniform material properties when properly compacted. Washed sand costs $4.50 per cubic yard, dredged sand is less. Both would probably have to be hauled to the site at a rate of $1.30 per cubic yard per hour of turn around time. Hand or machine compaction will range in price from $1.50 per cubic yard to $3.00 per cubic yard. For cost optimization purposes the select backfill in place was estimated at $7.00 per cubic yard.

The remaining portion of the excavation above the tank and select backfill (See Figure 3-3) is filled in with ordinary backfill obtained at the site. This material may be a coarse sand with some rock fragments. It is placed under careful compaction controls similar to the select backfill, however, its cost is minimal. Ordinary backfill normally costs $1.50 per cubic yard in place. For the optimization study it was assumed to cost $3.00 per cubic yard due to the strict placing requirements.

3-2.4 Pressurization Costs

A 15 horsepower compressor, which is capable of pressurization to 1000 psig, will range in cost from $3,000 to $5,000. In Section 2-10.1 it was recommended that two compressors be purchased in order to have one as a backup. Thus, the total compressor cost could be as high as $10,000. This is negligible in comparison to the cost of the tanks alone.



NOTE: FOR OPTIMIZATION STUDY,
$C = E = 0.2D \geq 3.0$ feet

Figure 3-3 EXCAVATION AND BACKFILL

Operation and maintenance of the compressor should not exceed $15 per month. This cost is directly proportional to the total storage volume at the site, not the individual tank volume. Thus, for economic comparison purposes between tanks, all compressor costs have been excluded from the study.

Miscellaneous costs of valves, piping, etc. have also been neglected in the economic comparison.

3-3 Parameters Considered in Study

The following parameters represent the more significant variables which could be considered in an optimization study:

1. Costs
   a. Tank
   b. Excavation
   c. Backfill
2. Tank Geometry and Properties
   a. Shape
   b. Diameter to Length Ratio
   c. Orientation in the Ground
   d. Yield Strength of the Material
3. Site Factors
   a. Depth of Burial
   b. Depth to Water Table
   c. In situ Soil Properties
   d. Backfill Properties
4. Nuclear Weapon Characteristics
   a. Weapon Yield
   b. Overpressure Level

5. Internal Pressurization

It is evident from the list of possible parameters that an exhaustive study of each one would involve a major effort. For the purposes of this report certain key parameters were selected for study. The others were held constant or allowed to vary in a prescribed manner.

The variable parameters are:

1. Tank volume or diameter which varies in 2 foot increments from 8 feet to 30 feet.

2. Overpressure level - 3000, 1500, 1000, 600, 200, and 10 psi.

3. Depth of water table - 0, 5, 13, 30, and 100 feet.

4. Internal pressurization - with pressure and without pressure.

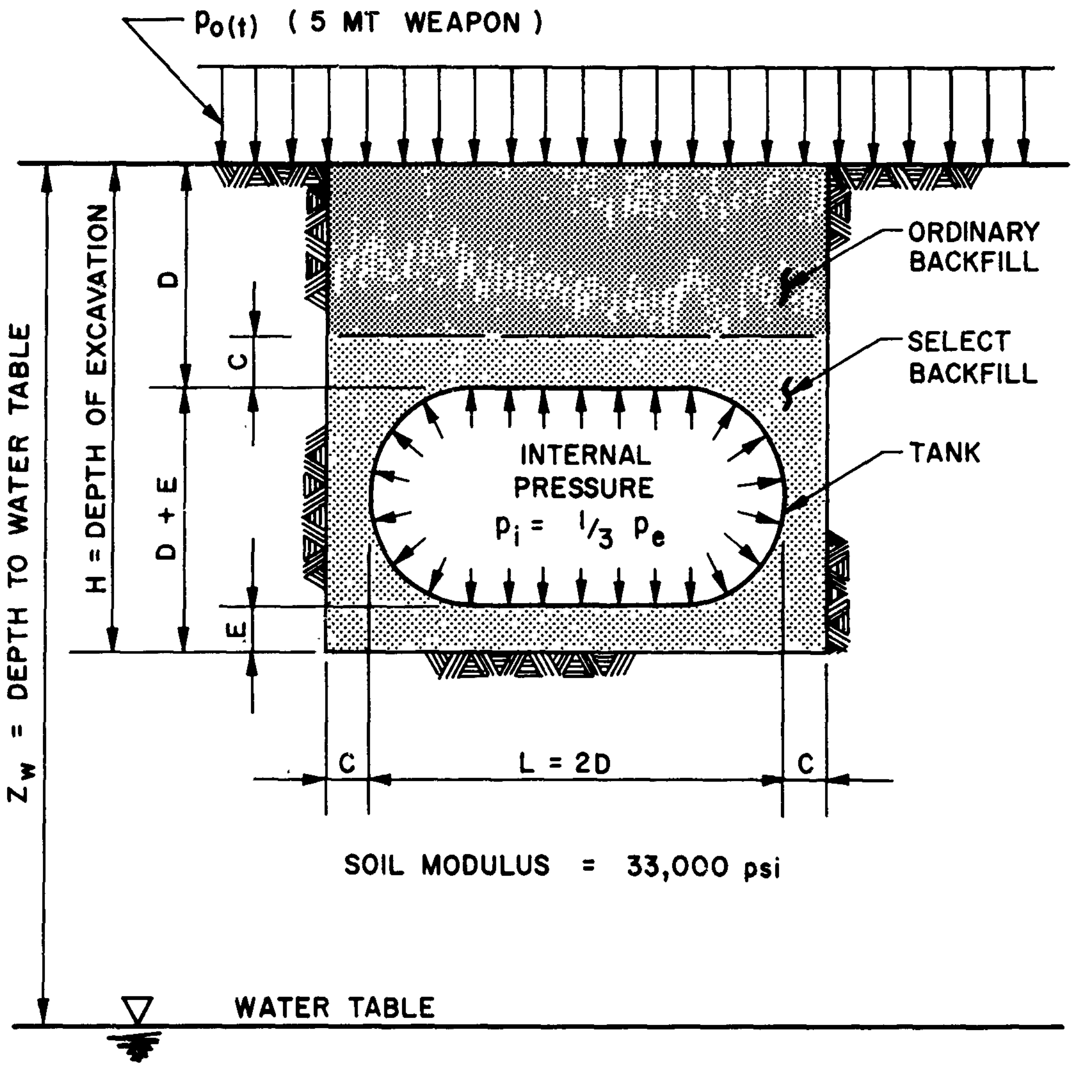
5. Yield strength of tank steel - 90 ksi versus 36 ksi yield stresses.

6. Excavation costs for rock site versus sand or clay site.

The tank diameter and overpressure level were variables in each of the four cases studied. The remaining parameters were varied individually within each case.

Tank costs are based on shop and field fabrication curves of Figure 3-1. For tanks 12 feet in diameter or under, shop fabrication is used, while tanks over 12 feet are field fabricated. The cost curves of Figure 3-1 are based on high yield strength steel such as T-1 or equivalent with a minimum yield strength of 90 ksi. For mild steel, such as A-36 with a yield strength of 36 ksi, the cost equations of Figure 3-1 are reduced by 50% to reflect the lower material and fabrication costs.

Excavation costs in sand and clay are based on the cost function of Figure 3-2, while excavation in rock is $28 per cubic yard and is independent of depth. The cross-sectional area of excavation is shown in Figures 3-3 and 3-4 along with the specified areas of select and



C = E = 0.2D ≤ 3 FEET

VARIABLE PARAMETERS:

TANK DIAMETER,	D = 8 FEET TO 30 FEET
OVERPRESSURE,	p_o = 3000, 1500, 1000, 600, 200 AND 100 psi
DEPTH TO WATER TABLE,	Z_w = 0, 5, 13, 30 AND 100 FEET
INTERNAL PRESSURE,	p_i = 0 AND $p_e/3$
YIELD STRESS OF TANK,	f_y = 90 ksi AND 36 ksi

Figure 3-4 PARAMETERS OF TANK OPTIMIZATION STUDY

ordinary backfill. A clearance between excavation and tank along its sides, bottom and ends is 20% of the tank diameter with a minimum of 3 feet. This minimum is based on working space requirements.

Cost for select backfill is $7.00 per cubic yard and for ordinary backfill $3.00 per cubic yard.

The tank diameter to length ratio is held constant with the length equal to two diameters as shown in Figure 3-4. It is oriented horizontally in the ground. High strength steel with a 90 ksi yield is considered in all but one of the cases studied. For this case A-36 steel with a yield stress of 36 ksi was evaluated.

The tank is buried at a depth of one diameter below ground surface in sandy soil. The elastic secant modulus for the soil was selected at 33,000 psi, which was found to be a good average for sandy soils.

A nuclear weapon yield of 5 MT was selected for all cases studied.

Internal pressure within the tank is calculated in each analysis to equal one third the effective external pressure acting on the tank.

Figure 3-4 summarizes the key parameters, both variable and non-variable, used to analytically model the buried fuel tank in the following optimization studies.

3-4 Results from Case Studies

3-4.1 Case 1 - Optimum Costs as a Function of Overpressure and Tank Volume

For this case the water table is located 100 feet below ground surface, giving a solution for dry soil conditions. Both overpressure and tank diameter are variables in the analyses.

The plot of plate thickness in Figure 1-2 was developed from this case study. It reflects the theoretical value for plate thickness. Due to fabrication consideration the minimum plate thickness for shop fabricated tanks is ¼ inch and for field fabrication it is ½ inch. In addition, available plate sizes come in incremental thicknesses which become coarser as the total plate thickness increases. These practical limitations have been included in the economic evaluation of the tanks and have resulted in certain discontinuites in the theoretically smooth optimization curves, as will be seen in many of the following figures.

Total tank costs for a single tank including materials, fabrication, installation and corrosion protection, excluding excavation and backfill, are given in Figure 3-5. There is a distinct discontinuity in costs between shop fabricated and field fabricated tanks due to the material and fabrication cost differentials. It should be noted that for low overpressures ranging from 200 psi on down to 10 psi, minimum plate thickness controls the design. In fact even at 600 psi, the design of field fabricated tanks up to 20 feet in diameter is controlled by the minimum workable plate thickness of ½ inch.

Total cost for a single tank including tank cost, excavation, and backfill is given in Figure 3-6. In the high overpressure range excavation and backfill account for roughly 12% of the total cost. While in the low overpressure range this figure increases to 50% of the total cost for large diameter tanks. Note also that for any given tank diameter the maximum range in total tank cost with overpressure can be as much as 500%.

The tank size which yields the minimum cost for enclosing a given volume is normally considered the optimum design. In this study the total tank cost to enclose the volume equivalent to that contained in a 30 foot diameter tank was computed for each of the tank sizes considered. The tank costs were then normalized by dividing each tank cost by the minimum tank cost, for any given overpressure level. The resulting ratio of total tank cost to minimum tank cost was plotted as a function of tank diameter and a smooth curve was passed through the values to produce the resulting optimization curve shown in Figure 3-7.

From this figure it is obvious that the 10 to 12 foot diameter shop fabricated tank is the most economical solution. Field fabrication of the tank will cost 30 to 50% more depending on the tank volume and overpressure level. For field fabrication in the high overpressure range of 3000 psi there appears to be an optimum tank size between 18 and 20 feet in diameter. At 200 psi overpressure, minimum plate thickness for field fabrication produces a sharp discontinuity at the 12 foot diameter tank size.

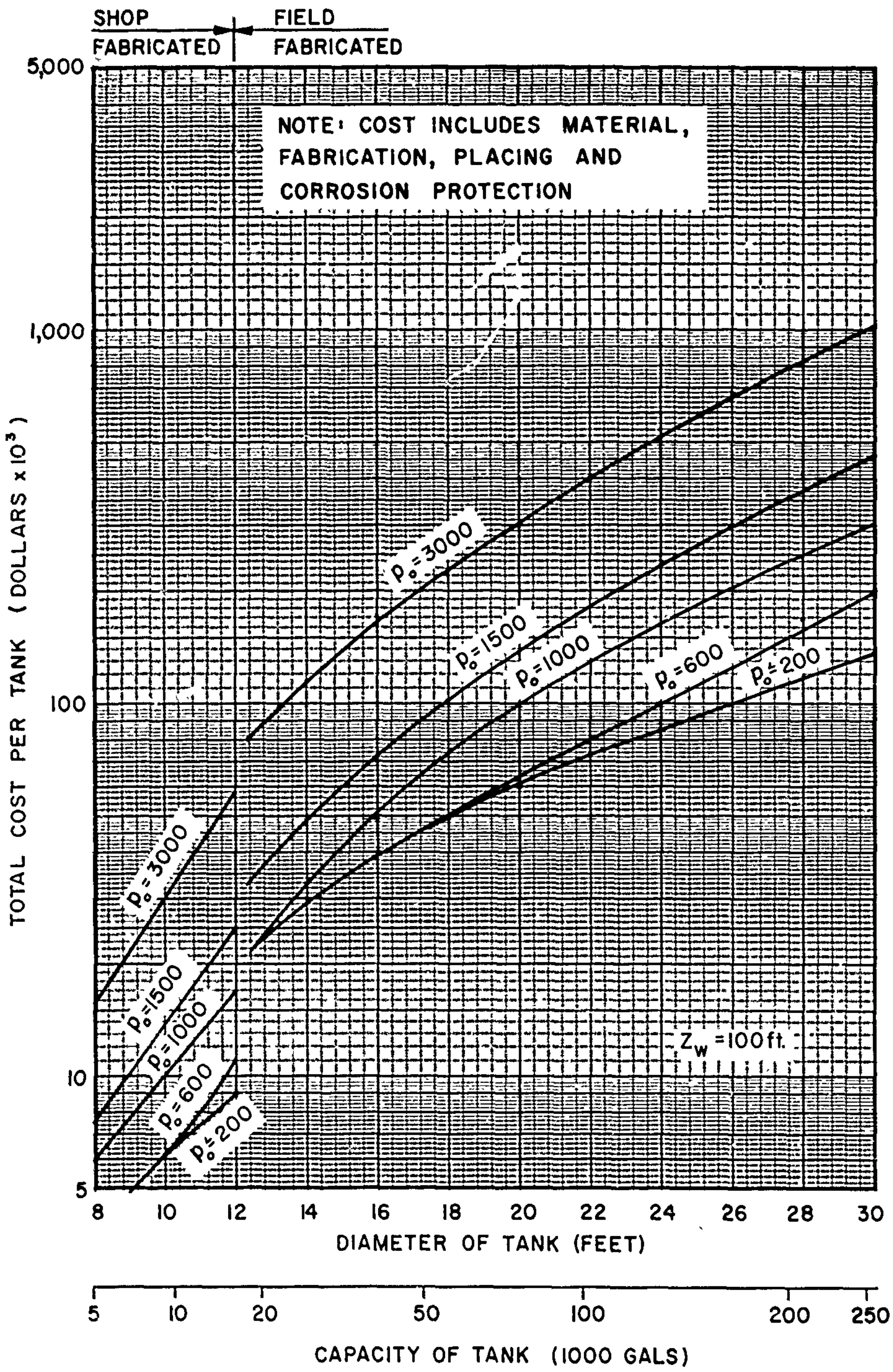


Figure 3-5 TOTAL TANK COST AS A FUNCTION OF PEAK OVERPRESSURE LEVEL AND TANK DIAMETER - CASE I

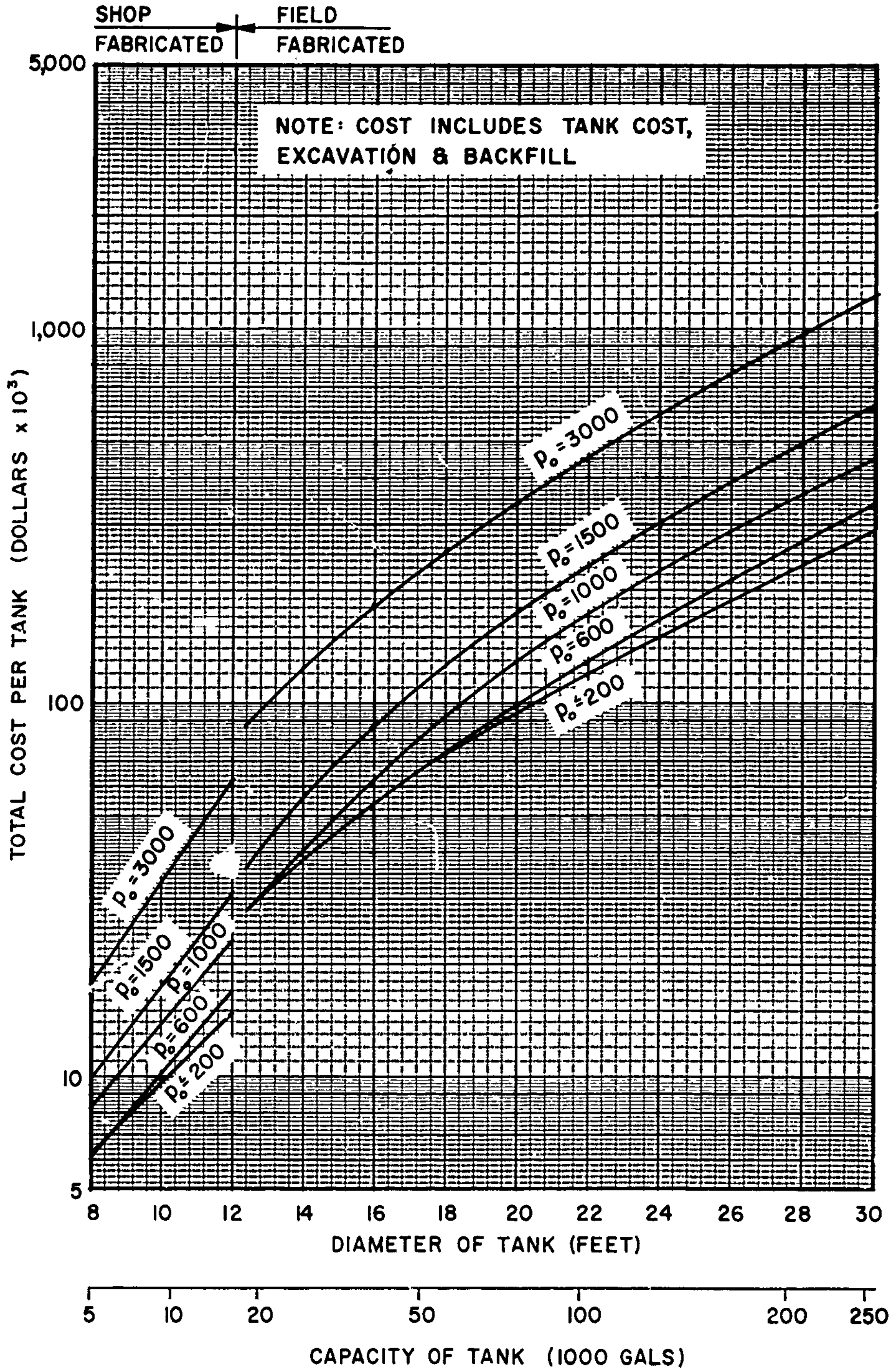


Figure 3-6 TOTAL COST PER TANK INSTALLED AS A FUNCTION OF PEAK OVERPRESSURE LEVEL AND TANK DIAMETER - CASE I

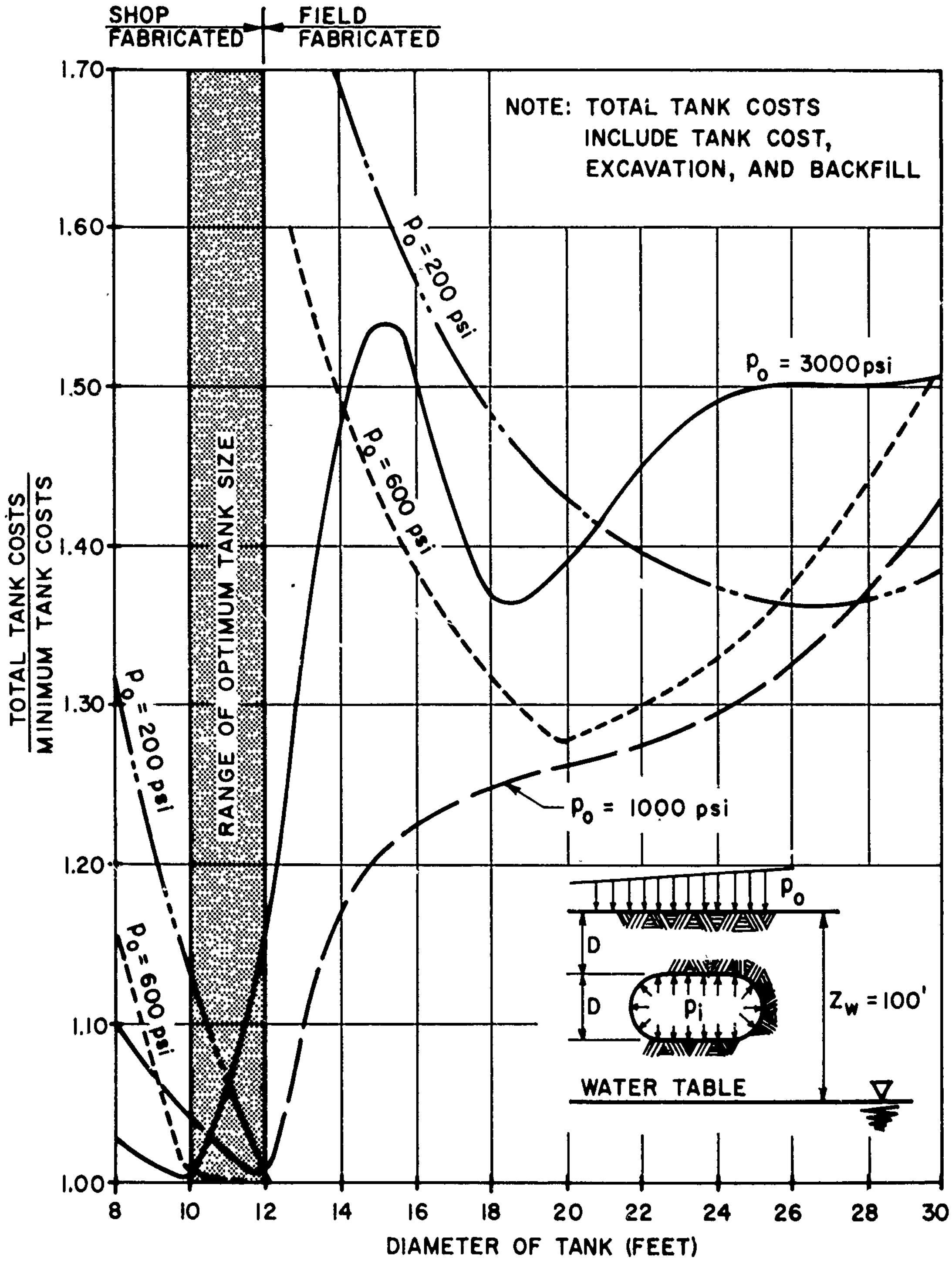


Figure 3-7 NORMALIZED TOTAL TANK COSTS AS A FUNCTION OF PEAK OVERPRESSURE LEVEL AND TANK DIAMETER - CASE I

This same plate thickness limitation forces the optimum field fabricated tank size up into the large diameter range between 26 and 28 feet. At 600 psi overpressure the limitation on plate thickness does not influence the results over as wide a range of tank sizes. However, it does alter the pattern of the optimization curve in the 12 to 20 foot diameter range.

3-4.2 Case 2 - Influence of Water Table Depth on Costs

For this case study the depth of water table was varied, starting at ground surface with $z_w = 0$ and moving on down to 5, 13, 30 and 100 foot depths. Overpressure and tank diameter were varied as well.

Figure 3-8 shows the influence of water table on the required design thickness of the tank. Since 6 inch plate is the maximum size available in high strength steel, large diameter tanks will not be feasible under high water table conditions and high overpressure levels. At 1500 psi overpressure level, the required plate thickness for the large diameter tanks is reduced to 3½ inches which is a more workable size, see Figure 3-9.

The influence of water table on total tank cost is graphically presented in Figure 3-10 and Figure 3-11 for overpressure levels of 3000 psi and 1000 psi respectively. The range in tank cost can be as much as 300% between a site with no water table and one with water table at the surface. These costs do not include dewatering or tank hold down costs. By comparing Figure 3-10 and Figure 3-11 it is evident that the cost differential due to water table decreases with reduced overpressure level. At 200 psi overpressure there is no further cost differential. Thus the presence of water table at this low overpressure does not appear to influence the tank design to a point that it is economically significant.

The variation of total tank cost as a function of overpressure, holding the water table constant, is shown in Figure 3-12. By comparing the results at 600 psi overpressure with those of Figure 3-5, the case without water table, it is evident that the minimum plate thickness for field fabricated tanks is no longer the controlling factor in the tank design. Instead, the design is dictated by the blast pressures which are amplified as they pass through the saturated soil.

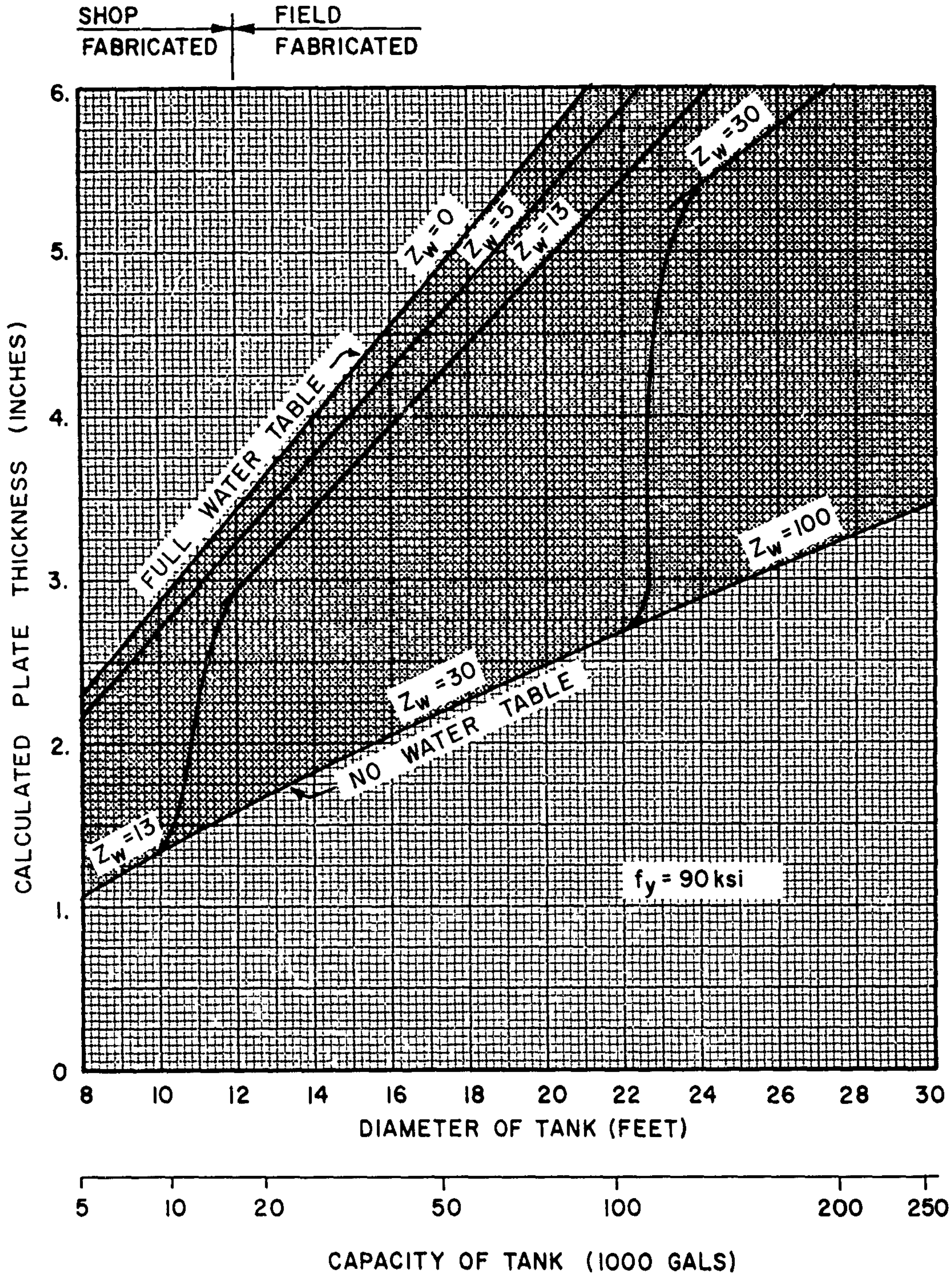


Figure 3-8 PLATE THICKNESS AS A FUNCTION OF WATER TABLE DEPTH AND TANK DIAMETER - CASE 2 ($P_0 = 3000$ psi)

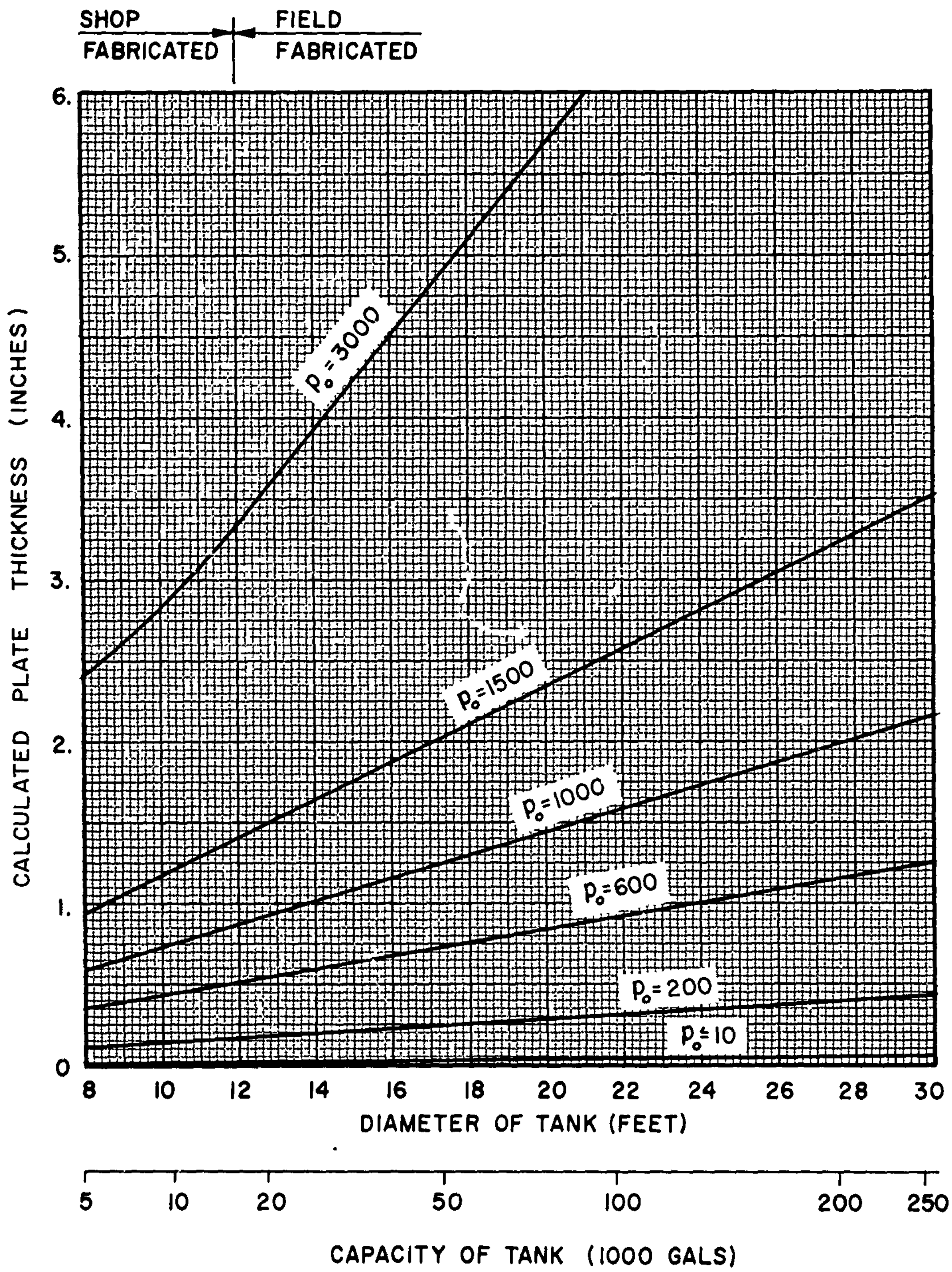


Figure 3-9 PLATE THICKNESS AS A FUNCTION OF OVERPRESSURE LEVEL AND TANK DIAMETER - CASE 2 (FULL WATER TABLE)

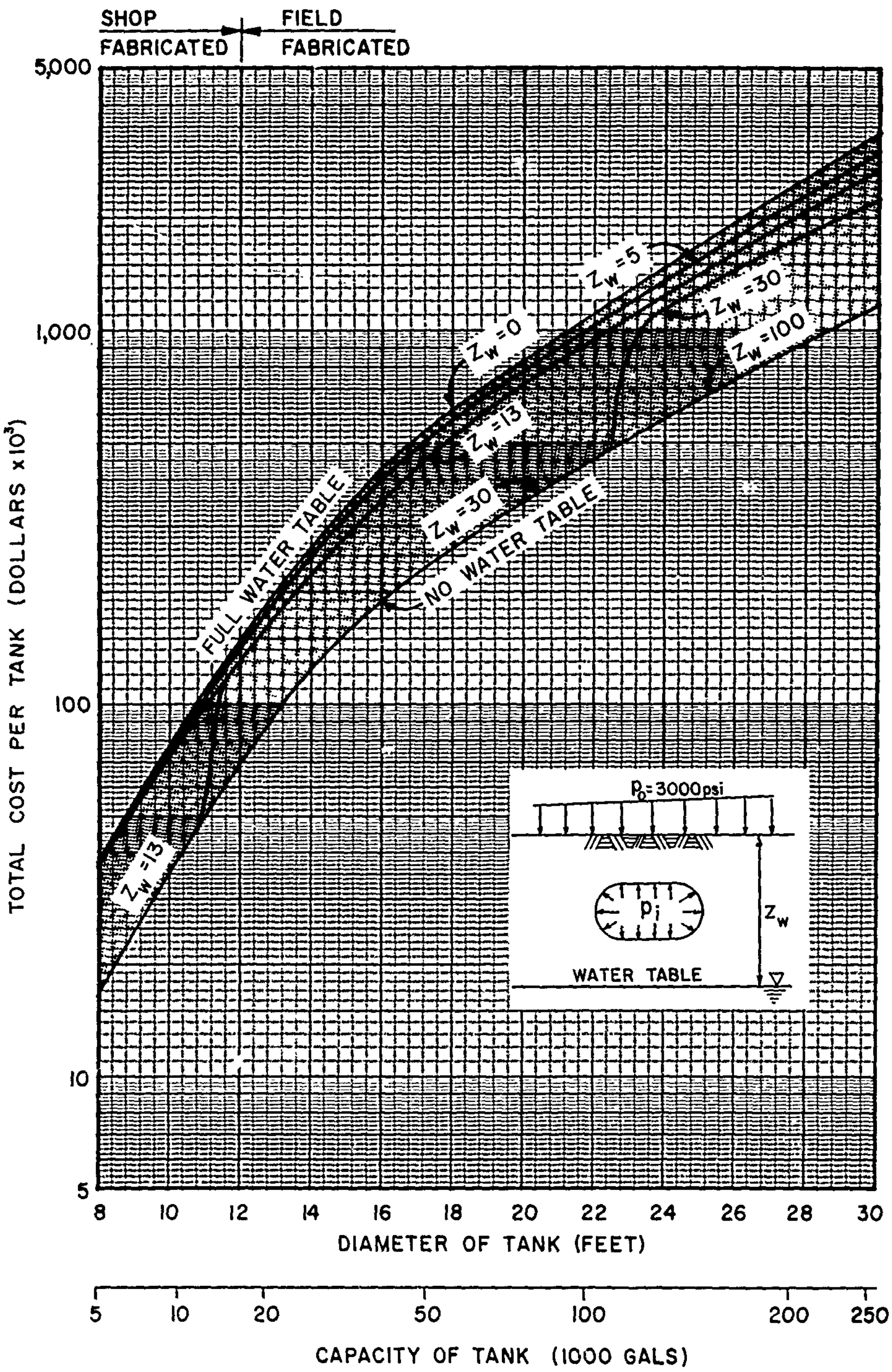


Figure 3-10 TOTAL COST PER TANK AS A FUNCTION OF WATER TABLE DEPTH AND TANK DIAMETER - CASE 2 (P_0 = 3000 psi)

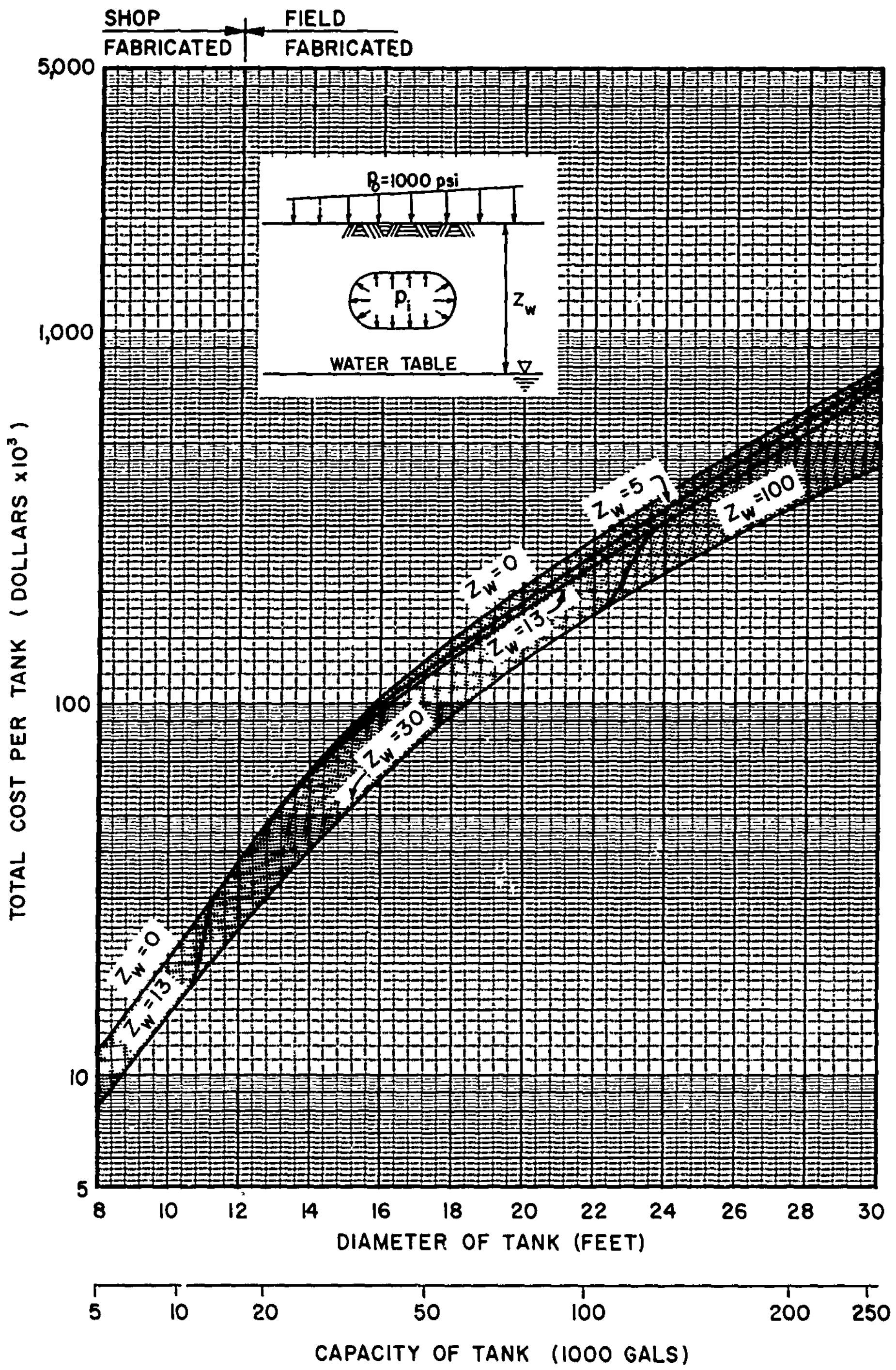


Figure 3-11 TOTAL COST PER TANK AS A FUNCTION OF WATER TABLE DEPTH AND TANK DIAMETER - CASE 2 (P_0 = 1000 psi)

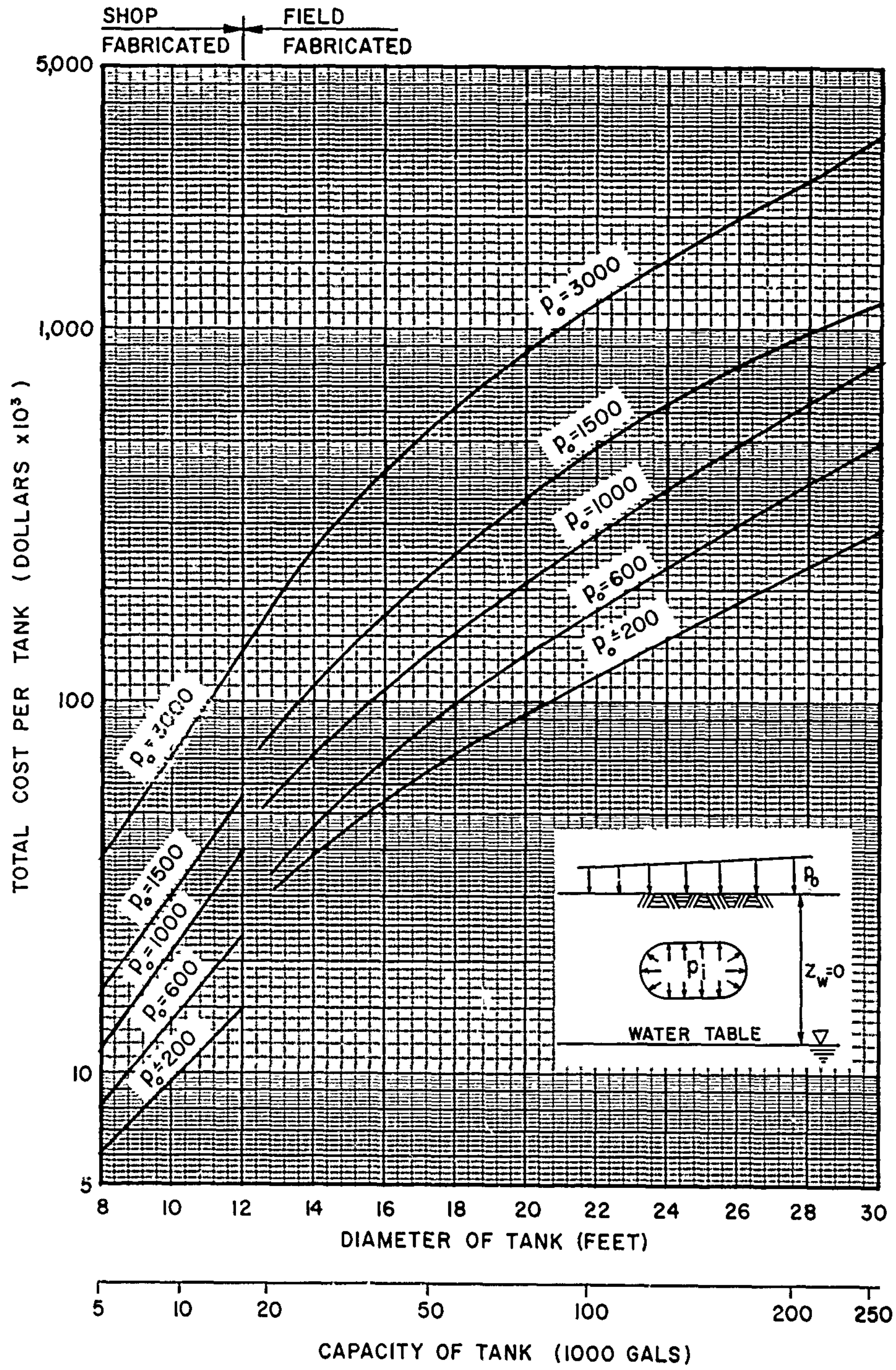


Figure 3-12 TOTAL COST PER TANK AS A FUNCTION OF OVERPRESSURE LEVEL AND TANK DIAMETER - CASE 2 ($Z_W = 0$)

An optimization curve for total tank costs under various water table conditions is presented in Figure 3-13.

The ratio of total tank cost to minimum tank cost has been normalized using the minimum cost for a tank installed at a dry site as the divisor. Thus, the plotted curves represent the relative cost between various water table depths as well as the relative cost between tank sizes at any given water depth. The optimum tank diameter is between 10 and 12 feet for all water table conditions. Thus, the presence of water table does not alter the optimum tank size, however, it is worth noting that at 1000 psi overpressure, full water table increases the total cost for shop fabricated tanks by 50% and for large diameter, field fabricated, tanks by 100% over the tank costs in dry soil conditions.

3-4.3 Case 3 - Influence of Internal Pressure on Tank Costs

To investigate the influence of internal pressurization on optimum tank design two cases were considered. One with normal internal pressure conditions and the other without internal pressure.

The resulting plate thicknesses calculated for the two cases are shown in Figure 3-14. It is evident that internal pressurization significantly reduces the required plate thickness. This is particularly true in the medium to high overpressure range.

A comparison of total tank costs is presented in Figure 3-15, for various overpressure levels. Internal pressurization reduces the total tank cost by as much as 50% in the high overpressure range. In the low overpressure range, under 200 psi, fabrication requirements dictate the minimum plate thickness rather than the external pressure loads. Similarly, the influence of internal pressurization does not effect the determination of plate thickness in this overpressure range.

For the case without internal pressure, the optimum tank size is in the 10 to 12 foot diameter range, which is the same as for the cases studied previously.

3-4.4 Case 4 - Influence of Tank Yield Strength

From the results of Case 1 it is apparent that in the low overpressure range, tank costs are controlled by

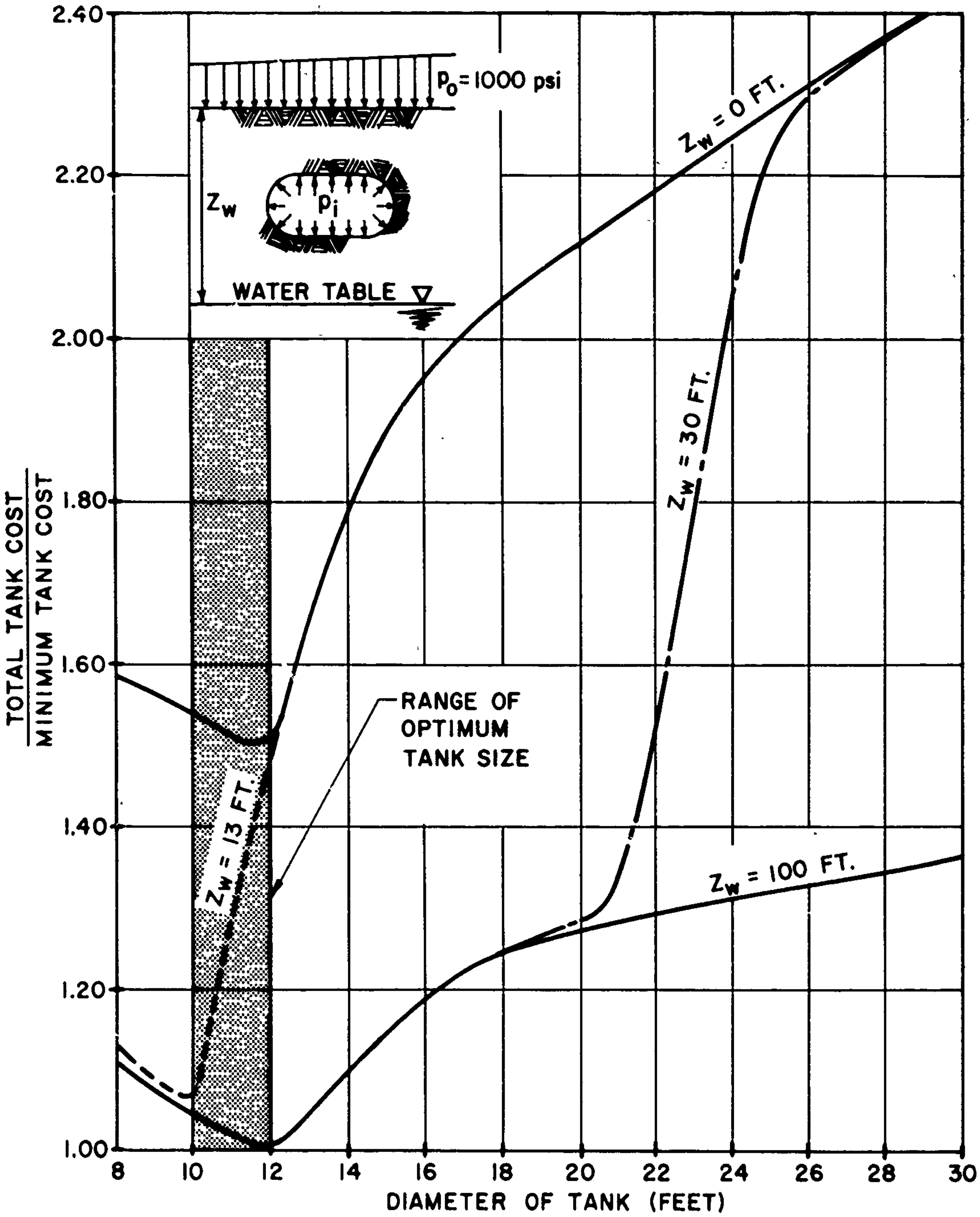


Figure 3-13 NORMALIZED TOTAL TANK COSTS AS A FUNCTION OF WATER TABLE DEPTH AND TANK DIAMETER - CASE 2 (P_0 = 1000 psi)

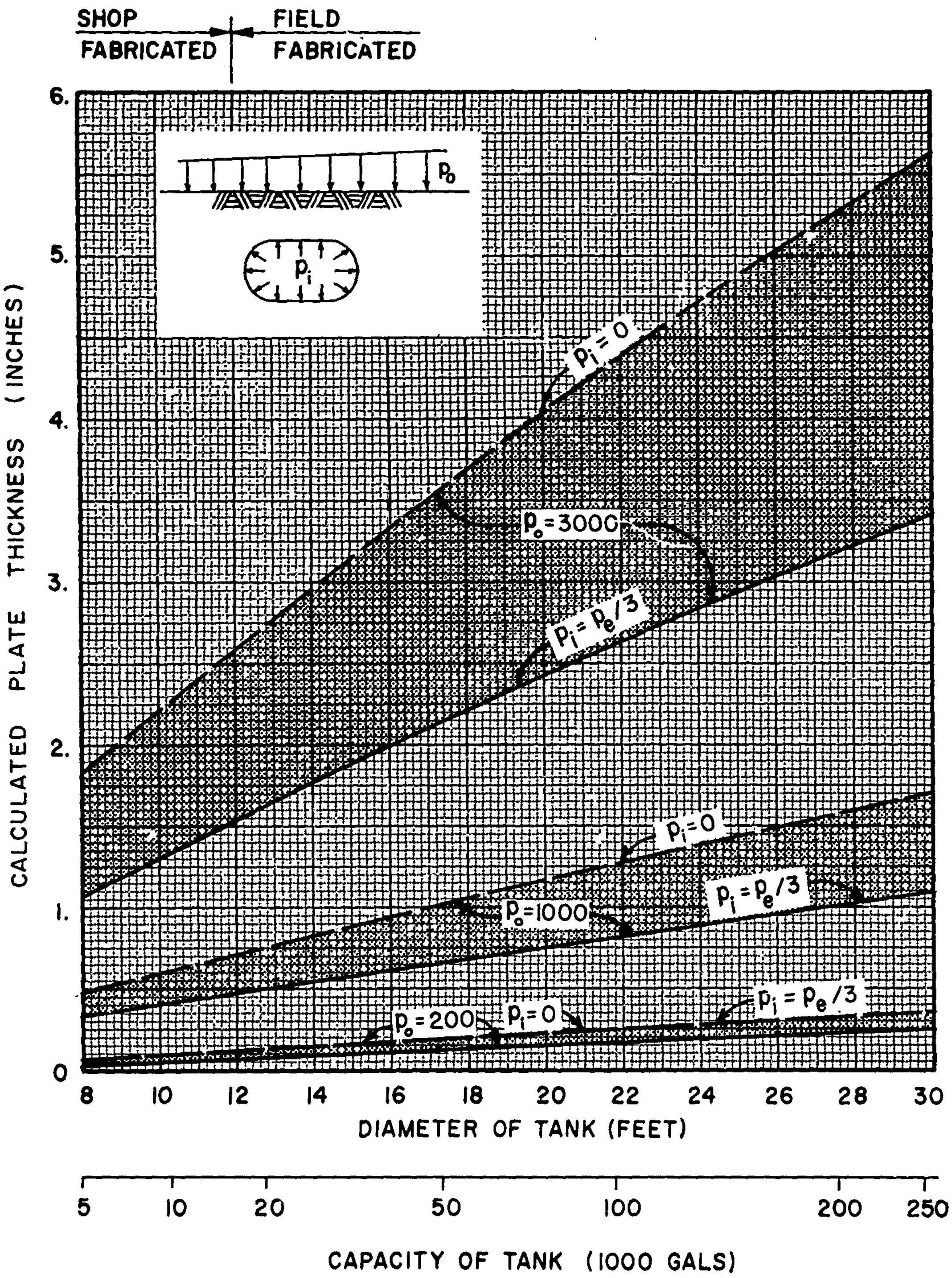


Figure 3-14 PLATE THICKNESS AS A FUNCTION OF INTERNAL PRESSURE, OVERPRESSURE AND TANK DIAMETER - CASE 3

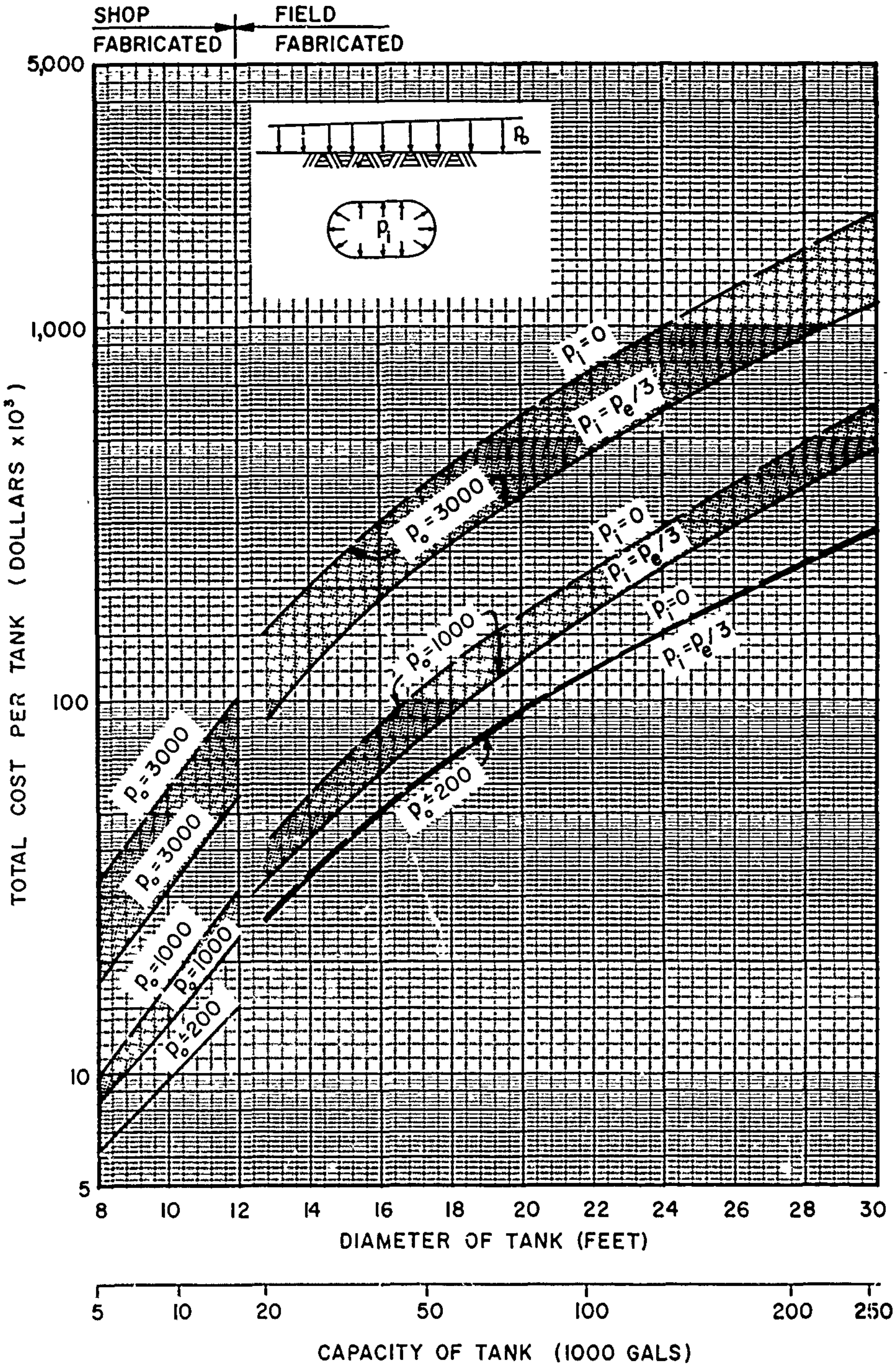


Figure 3-15 TOTAL COST PER TANK AS A FUNCTION OF INTERNAL PRESSURE, OVERPRESSURE AND TANK DIAMETER - CASE 3

minimum plate thickness requirements rather than strength. Thus, the economic advantages of using a low yield strength steel, such as A-36 with a yield stress of 36 ksi, were investigated.

Figure 3-16 presents the calculated plate thickness requirements based on design pressure criteria. At 200 psi overpressure, the calculated plate size for A-36 steel closely parallels the shop and field fabrication requirements of ¼ inch and ½ inch respectively. It is interesting to note that for the 3000 psi overpressure level, the design plate thickness for A-36 steel exceeds 6 inches. Fabrication of plates this size is costly, as can be seen by the total tank costs shown in Figure 3-17. These results further point out the fact that high yield strength steels are the most economical solution in the medium to high overpressure range. While in the low overpressure range under 200 psi, normal mild steel with a yield strength of 33 to 36 ksi is more economical.

The plot of normalized total tank costs in Figure 3-18 reveals that the optimum size for tanks shop fabricated from A-36 steel is 12 feet in diameter and for field fabrication it is 20 feet in diameter. These results apply only to the low overpressure range of 200 psi and under.

3-4.5 Case 5 - Influence of Soil Conditions on Excavation Costs

An economic study was made of the excavation costs in rock versus sand or clay soils. The variable was tank diameter or excavation volume. Water table was set well below the bottom of the excavation to exclude dewatering costs from the comparison.

The resulting excavation costs are presented in Figure 3-19. As anticipated, rock excavation costs exceed sand and clay soil excavation costs by a margin of 600% in the small tank diameter range and by 100% in the large tank diameter range.

Backfill costs have been added to the excavation costs to provide an economical comparison between the two costs. Backfill costs are a very small percentage of the total excavation cost for large diameter tanks. They do increase to significant proportions for small diameter tanks under sandy or clayey soil conditions.

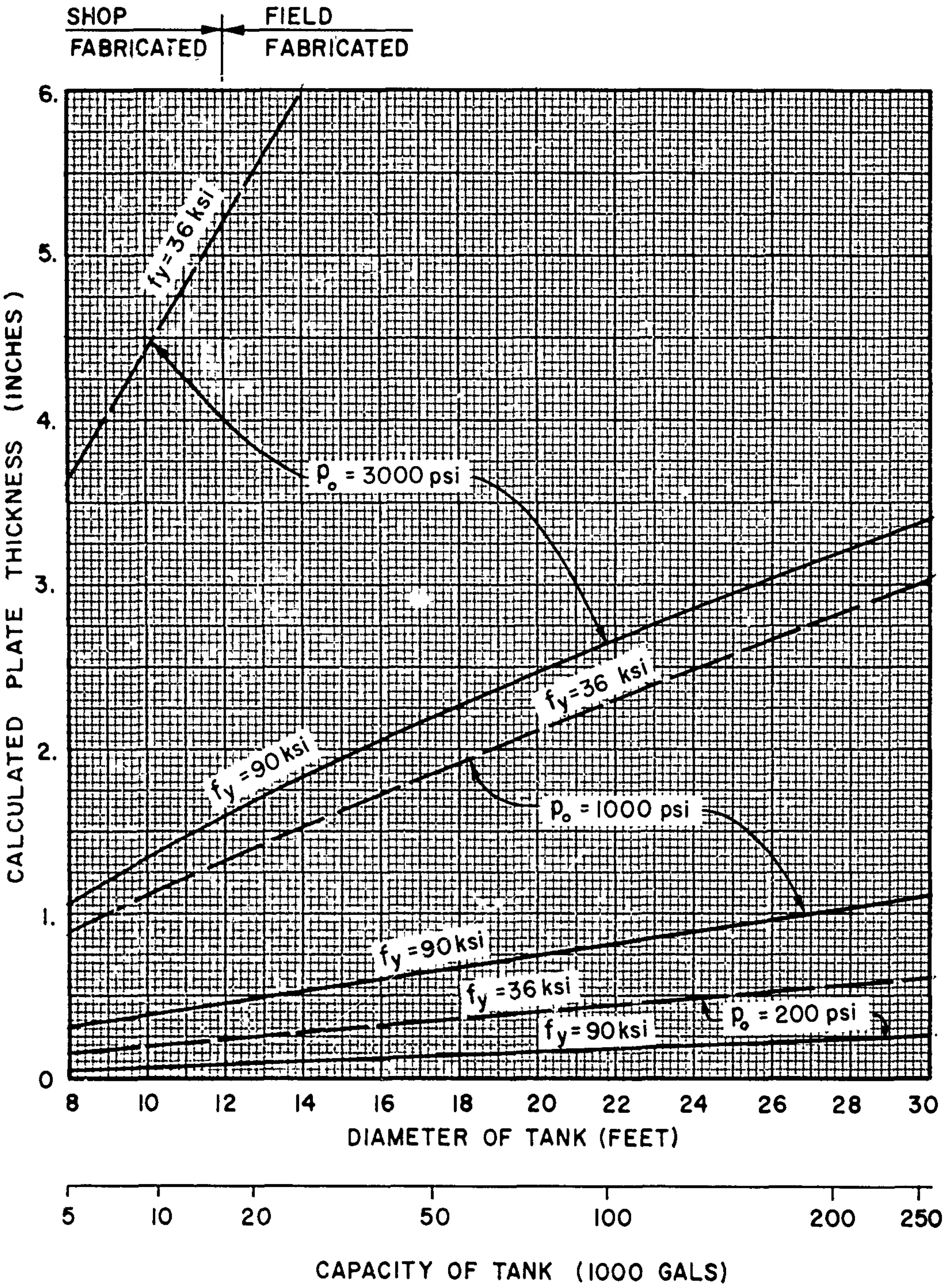


Figure 3-16 PLATE THICKNESS AS A FUNCTION OF YIELD STRESS, OVERPRESSURE LEVEL AND TANK DIAMETER - CASE 4

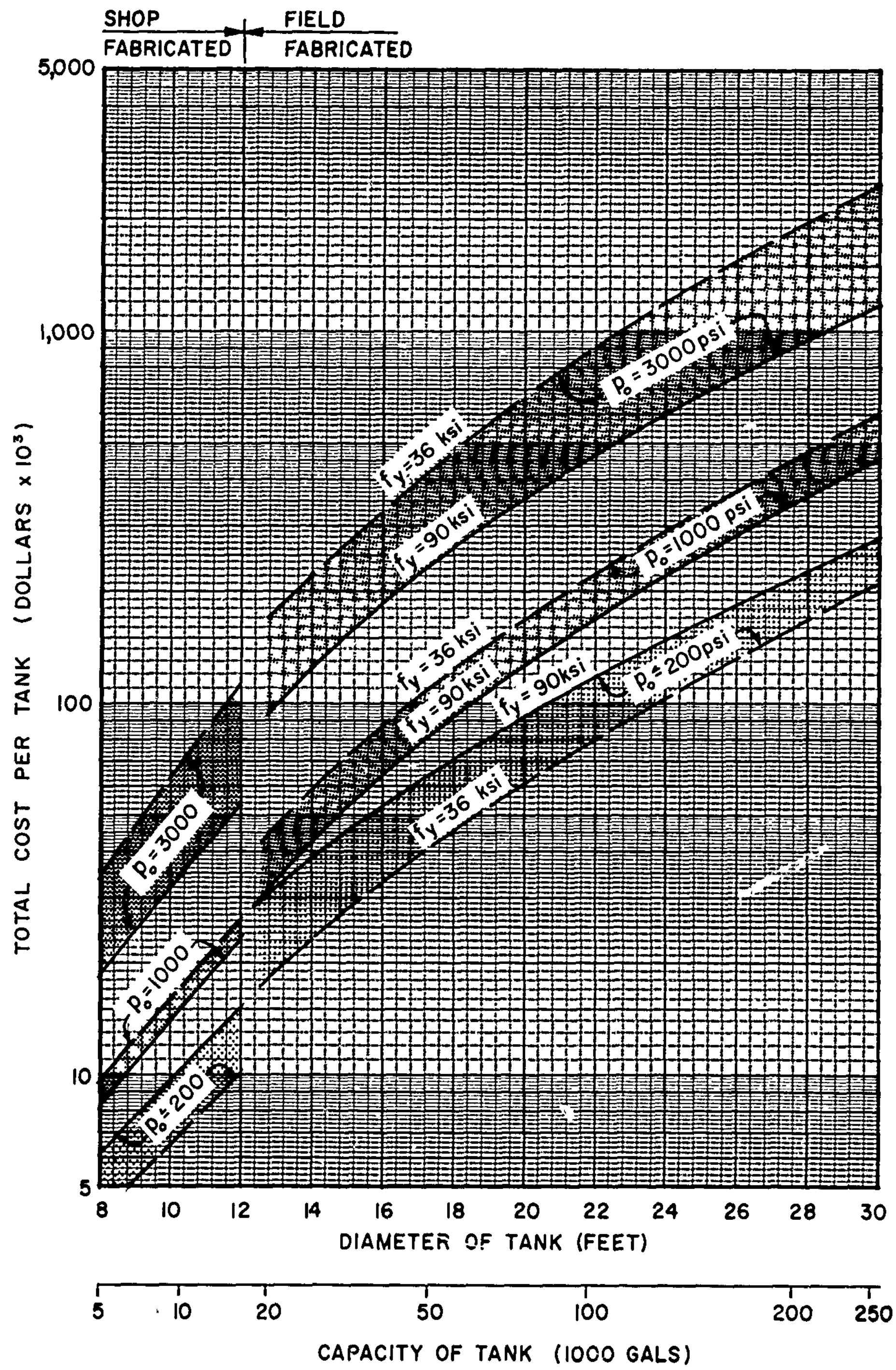


Figure 3-17 TOTAL COST PER TANK AS A FUNCTION OF YIELD STRESS, OVERPRESSURE LEVEL AND TANK DIAMETER - CASE 4

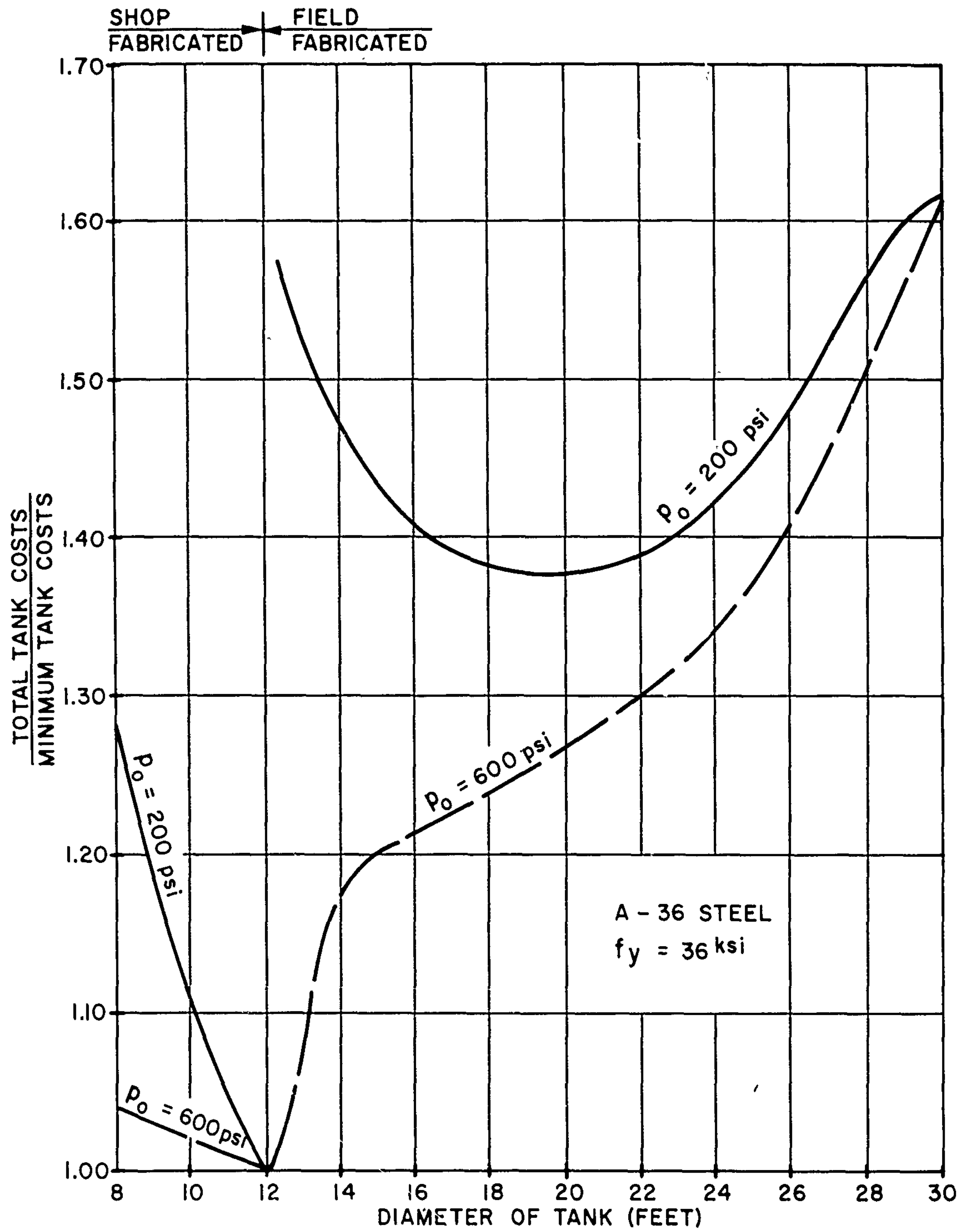


Figure 3-18 NORMALIZED TOTAL TANK COSTS AS A FUNCTION OF PEAK OVERPRESSURE LEVEL AND TANK DIAMETER - CASE 4

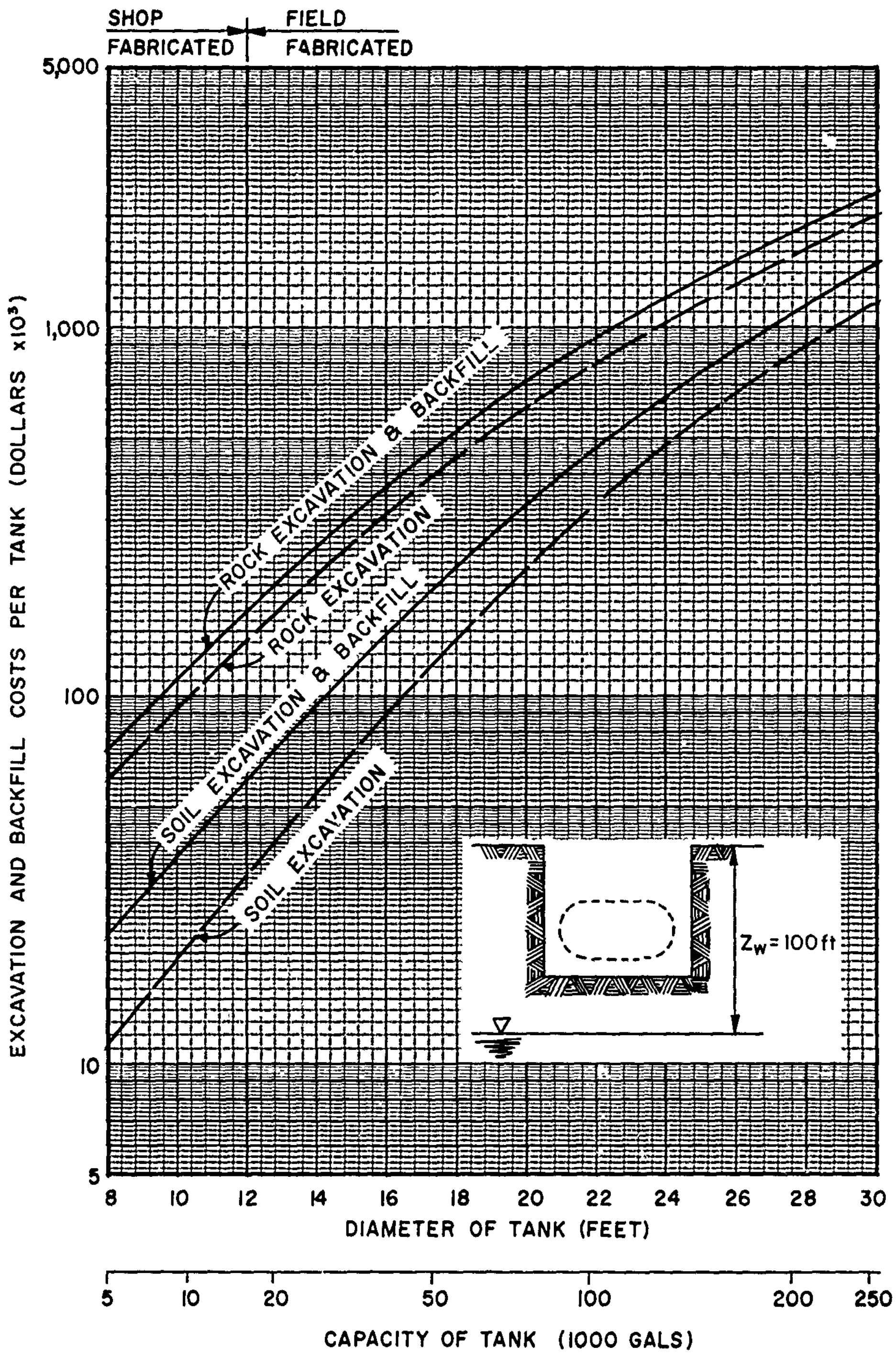


Figure 3-19 EXCAVATION AND BACKFILL COSTS FOR ROCK SITE VERSUS SAND OR CLAY SOIL CONDITIONS - CASE 5

3-5 List of Major Findings and Conclusions

From the five cases studied the following conclusions can be derived:

1. From an economical standpoint, the optimum tank size is the 10 to 12 foot diameter shop fabricated tank.

2. High water table conditions may increase the material and fabrication costs for a tank by 300%.

3. High water table in combination with 3000 psi overpressure produces design requirements which exceed the available plate size for high strength steels.

4. Internal pressurization provides a significant savings in tank costs. This savings far exceeds the initial costs of compressor and special mechanical equipment used to pressurize the tanks.

5. For overpressure levels over 200 psi, high strength steels such as U.S. Steel's T-1, or equivalent, with a 90 ksi yield strength, are more economical to use than mild steels with a yield strength of 36 ksi. Under 200 psi overpressure, fabrication restrictions on minimum plate thickness make mild steels the economic choice.

6. Rock site excavation costs exceed soft soil site costs by a minimum of 100%.

REFERENCES

1 Allgood, J.R., Ciani, J.B., and Lew, T.K., *Influence of Soil Modulus on the Behavior of Cylinders Buried in Sand,* Technical Report R-582, U.S. Naval Civil Engineering Laboratory, June 1968.

2 Allgood, J.R. and Herrmann, H.G., *Static Behavior of Buried Reinforced Concrete Model Cylinders*, Technical Report R-606, U.S. Naval Civil Engineering Laboratory, January 1969.

3 AMF Advanced Systems Laboratory, *Sanguine Facility System Study (U)*, Naval Civil Engineering Laboratory Contract Report CR 69.020, Port Hueneme, California, December 1969, (Classified).

4 Chelapati, C.V., *Critical Pressures for Radially Supported Cylinders*, Technical Note N-773, for the U.S. Naval Civil Engineering Laboratory, January 1966, (Contract NBY-32280) (AD 627082).

5 *Corrosion Prevention and Control,* NAVDOCKS MO-306, Department of the Navy, Bureau of Yards and Docks, June 1964.

6 Gates, W.E. and Takahashi, S.K., *Static and Dynamic Model Tests of Pressurized, Underground Fuel Storage Containers, Comparison of Experimental Results with Finite Element Analysis*, NCEL Technical Report (currently in preparation).

7 Gill, H.L. and True, D.G., *Active Arching of Sand During Static Loading*, Technical Note N-759, U.S. Naval Civil Engineering Laboratory, November 1966.

8 Haltiwanger, J.D. et al, *Attachments and Connections to Buried Structures*, Naval Civil Engineering Laboratory Contract Report - Contract No. NBY 32279, October 1965.

9 Hendron, A.J. and Auld, H.E., *The Effects of Soil Properties on the Attenuation of Airblast-Induced Ground Motions*, Proceeding of the International Symposium on Wave Propagation and Dynamic Properties of Earth Materials, University of New Mexico, August 1967.

10 Hendron, A.J., Gamble, W.L., Haltiwanger, J.D., and Newmark, N.M., *Design of Cylindrical Reinforced Concrete Tunnel Liners to Resist Air Overpressures*, Report to the U.S. Naval Civil Engineering Laboratory by N.M. Newmark, Consulting Engineering Services, Urbana, Illinois, June 1968.

11 Murphy, H.L., *Ground Motion Predictions for Nuclear Attack Area Studies*, Stanford Research Institute, May 1967.

12 Newmark, N.M. and Hall, W.J., *Preliminary Design Methods for Underground Protective Structures*, AFSWC-TDR-62-6, June 1962.

13 Newmark, N.M. and Haltiwanger, J.D., *Principles and Practices for Design of Hardened Structures*, Air Force Design Manual, AFSWC-TDR-62-138, December 1962.

14 Sauer, F.M., Clark, G.B., and Anderson, D.C., *Nuclear Geoplosics, Part IV, Empirical Analysis of Ground Motion and Cratering*, Stanford Research Institute, May 1964.

15 True, D.G., *Design Criteria for Flexible Utility Connections*, Naval Civil Engineering Laboratory, Technical Report R608, December 1968.

16 True, D.G., *Exploratory High-Explosive Field Tests of Flexible Utility Connections*, Naval Civil Engineering Laboratory, Technical Report R638, September 1969.

17 Wilson, E., et al, *Dynamic Analysis of Two-Dimensional Structure - Media Systems with Initial Stress and Nonhomogeneous Damping*, Naval Civil Engineering Laboratory Contract Report, CR69.019, January 1970.

APPENDIX A

COMPUTER PROGRAM FOR DESIGN AND COST OPTIMIZATION

A-1 Introduction

The design procedure for horizontally buried, internally pressurized fuel storage tanks, as outlined in Section 2-7, was coded for analysis on the IBM 1130 computer. To generate the cost optimization curves of Section 3, a series of subroutines were added to the program which calculated material quantities and costs. Thus the computer program which is presented in this Appendix will calculate both tank thickness and costs. The cost functions included in the program are those given in Section 3-2.

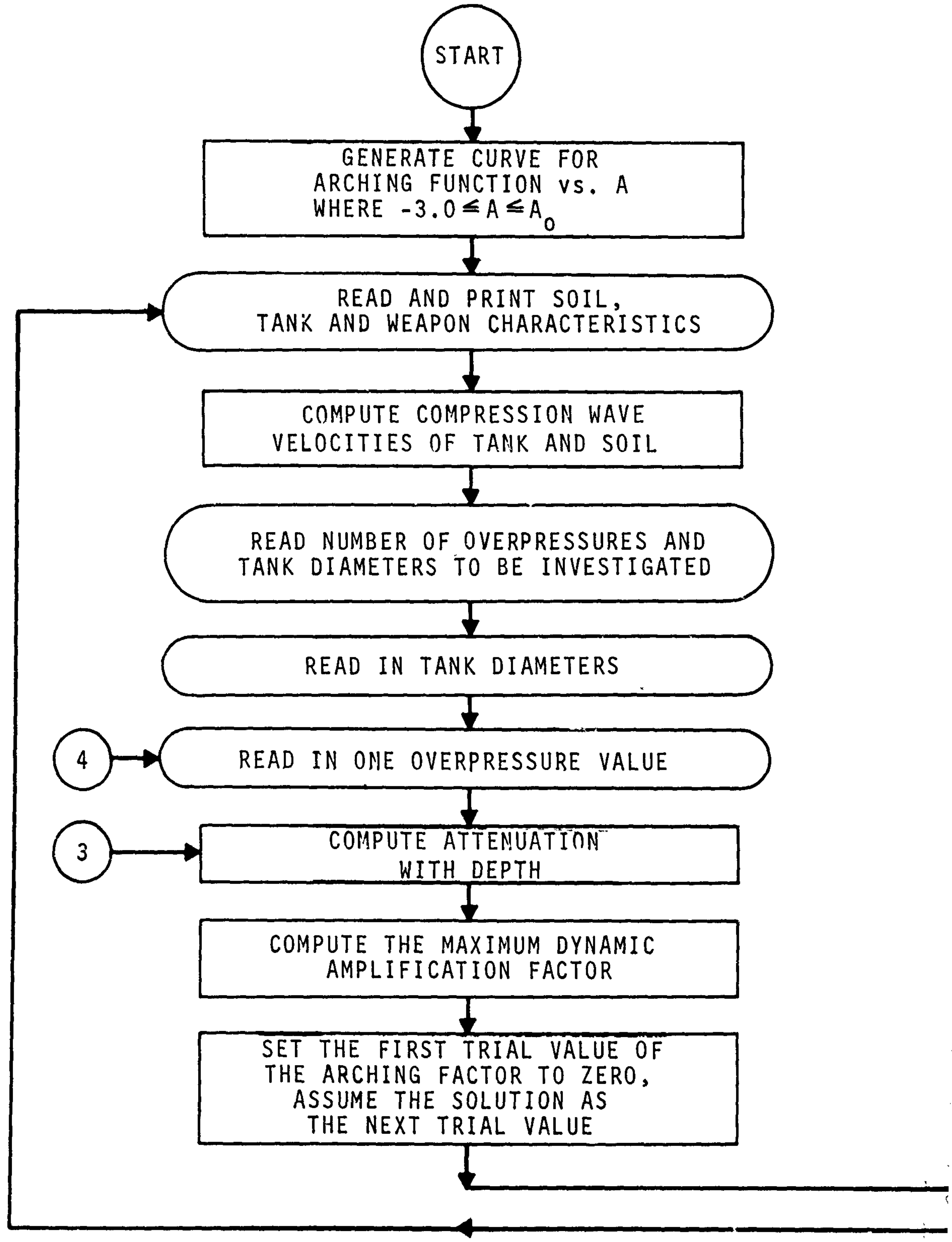
A-2 Computer Program

Figure A-1 is a flow diagram of the program logic. Following the figure is the program listing on Pages A-4 to A-15. All input data has been defined and units noted on the first sheet. Definition of terms used in the program follows the input information.

The program is dimensioned to take 15 different overpressures and 15 different tank diameters in a single run. This number may be increased by modification of the program dimension statement.

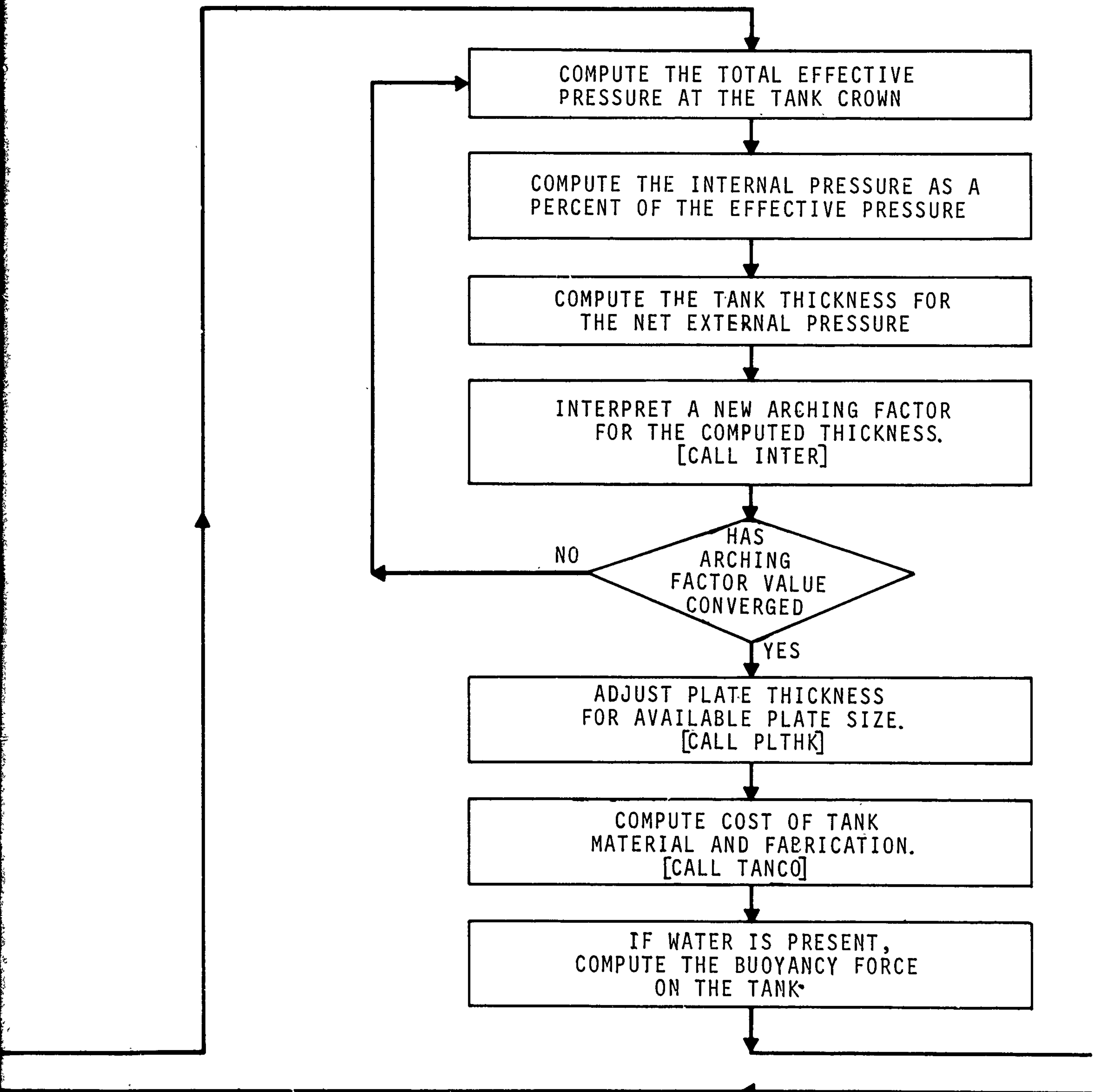
A-3 Sample Problems

The two design examples of Section 2-8 are presented as sample input-output to the computer program. Design Example No. 1 is on pages A-3 to A-4 while Design Example No. 2 is on pages A-5 to A-11.



START
GENERATE CURVE FOR ARCHING FUNCTION vs. A WHERE $-3.0 \leq A \leq A_0$
READ AND PRINT SOIL, TANK AND WEAPON CHARACTERISTICS
COMPUTE COMPRESSION WAVE VELOCITIES OF TANK AND SOIL
READ NUMBER OF OVERPRESSURES AND TANK DIAMETERS TO BE INVESTIGATED
READ IN TANK DIAMETERS
4
READ IN ONE OVERPRESSURE VALUE
3
COMPUTE ATTENUATION WITH DEPTH
COMPUTE THE MAXIMUM DYNAMIC AMPLIFICATION FACTOR
SET THE FIRST TRIAL VALUE OF THE ARCHING FACTOR TO ZERO, ASSUME THE SOLUTION AS THE NEXT TRIAL VALUE

GENERAL LOGIC DIAGRAM FOR DESIGN AND COST OPTIMIZATIO



B

ON PROGRAM (TKOPT)

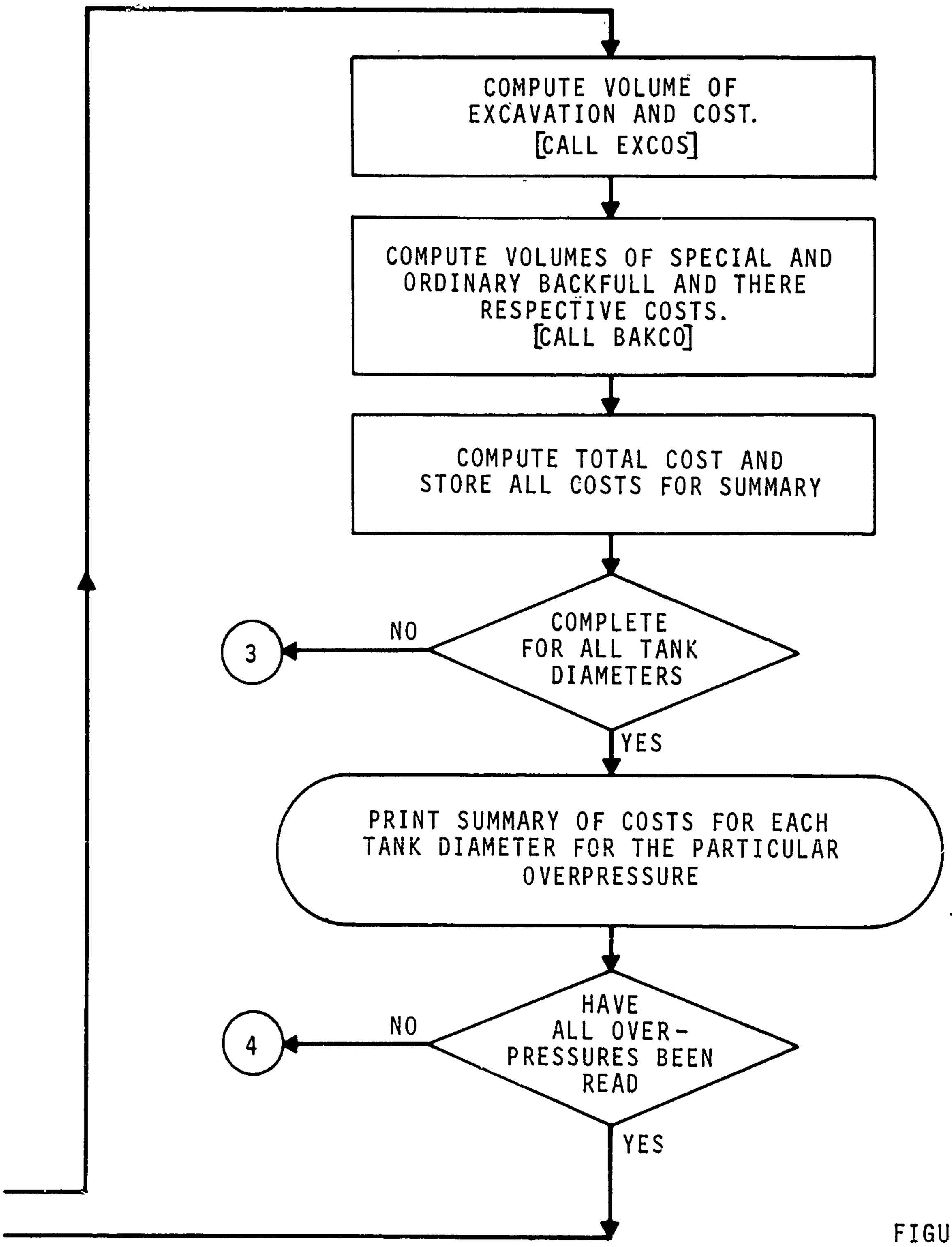
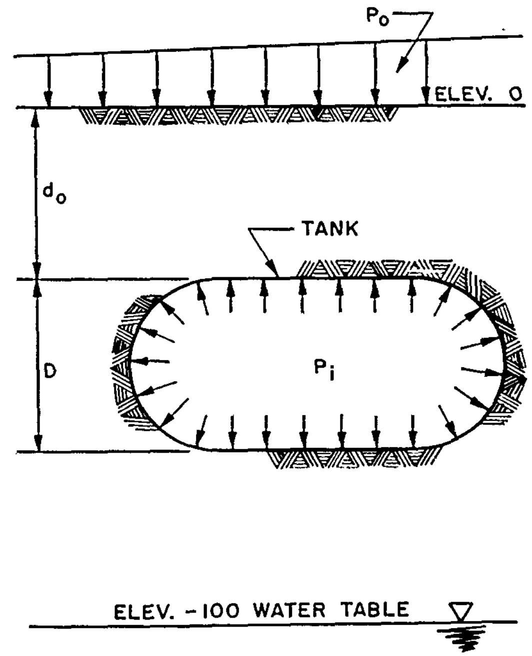


FIGURE A-1

A C

```
DESIGN AND COST OPTIMIZATION PROGRAM FOR
HORIZONTALLY BURIED FUEL STORAGE TANKS
CONSTRUCTED OF STEEL AND INTERNALLY PRESSURIZED
**************************************************

                                          PAGE-  1

DESIGN EXAMPLE NO. 1
****************************************************

INPUT DATA
***************************************************
UNIT WEIGHT OF SOIL-                 110.0 LBS/CU-FT
UNIT WEIGHT OF SATURATED SOIL-       130.0 LBS/CU-FT
ELASTIC MODULUS OF SOIL-           33000.  PSI
COEFFICIENT OF INTERNAL FRICTION-  37.00  DEG
YIELD STRESS OF TANK-              90000.  PSI
ELASTIC MODULUS OF TANK-        30000000.  PSI
UNIT WEIGHT OF TANK-                 490.0 LBS/CU-FT
POISSON'S RATIO-                   0.2500
WEAPON YIELD-                        5.00 MT
DEPTH OF WATER TABLE-              100.00 FEET
TANK LENGTH FACTOR-                  2.00
BURIAL DEPTH FACTOR-                 1.00

OVERPRESSURES( 1)               TANK DIAMETERS( 1)
-----------------               ------------------
  1500.00 PSI                         16.00 FEET
***************************************************

                                          PAGE-  2

COMPUTED CONSTANTS
***************************************************

COMPRESSION WAVE VELOCITY OF SOIL-  1180.3 FPS
COMPRESSION WAVE VELOCITY OF TANK- 18471.3 FPS
***************************************************

INTERNAL CONSTANTS
***************************************************
EXPONENTIAL N FACTOR-              0.1350
```



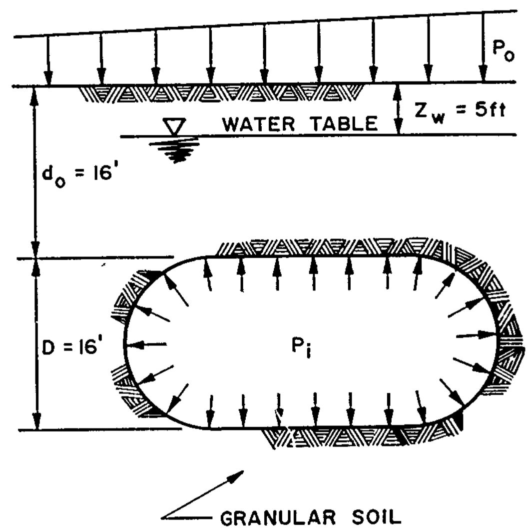
```
OVERPRESSURE        -     1500.00 PSI                                                             PAGE-  3
DIAMETER OF TANK    -       16.00 FEET
LENGTH OF TANK      -       32.00 FEET
BURIAL DEPTH        -       16.00 FEET
WATER TABLE DEPTH -       100.00 FEET
*************************************************

DYNAMIC AMPLIFICATION FUNCTION FOR-
T(INITIAL)=     1.99775            T(TOTAL)=    2.00183
FACTOR USED=    2.00183
PRESSURE AMPLIFICATION FACTOR DUE TO WATER TABLE-     1.0000
DEADLOAD AND WATER PRESSURE=       8.37 PSI           INTERNAL PRESSURE =                 523.51 PSI
CRITICAL PRESSURE =             1047.02 PSI           EFFECTIVE PRESSURE AT TANK=        1570.53 PSI
ALPHA =                         0.75887               ARCHING FACTOR =                   0.31522
CALC. TANK THICKNESS=           0.96694 IN.
WEIGHT OF TANK =                  65680. LBS.         THICKNESS USED=                    1.00000 IN.
BUOYANCY EFFECT =                     0. LBS.  (TOTAL UPLIFT FORCE)
VOLUME OF EXCAVATION       =(CY)      1121.
VOLUME OF SPECIAL BACKFILL =(CY)       515.
VOLUME OF ORDINARY BACKFILL=(CY)       407.
VOLUME OF TANK            =(CF)    5361.66 OR     40110. GALS.        TOTAL COST OF TANK = ($)   87254.73
-------------------------------------------------------------------------------------------------------------
```


SUMMARY SHEET NO. 1

```
OVERPRESSURE       = 1500.00 PSI
WATER TABLE DEPTH =  100.00 FEET
LENGTH OF TANK    =    2.00 X DIAMETER OF TANK
TANK BURIAL DEPTH =    1.00 X DIAMETER OF TANK
```

TANK DIA. (FEET)	CAPACITY (GALS)	TANK COST	EXCAV. COST	SPECIAL FILL COST	ORDINARY FILL COST	EXCAV. AND FILL COST	TOTAL COST
16.00	40110.	73342.86	9083.29	3605.23	1223.33	13911.87	87254.73

```
DESIGN AND COST OPTIMIZATION PROGRAM FOR
HORIZONTALLY BURIED FUEL STORAGE TANKS
CONSTRUCTED OF STEEL AND INTERNALLY PRESSURIZED
************************************************
                                       PAGE-  1
DESIGN EXAMPLE NO. 2
**************************************************

INPUT DATA
************************************************
UNIT WEIGHT OF SOIL-                 110.0 LBS/CU-FT
UNIT WEIGHT OF SATURATED SOIL-       130.0 LBS/CU-FT
ELASTIC MODULUS OF SOIL-           95000.  PSI
COEFFICIENT OF INTERNAL FRICTION-  41.00 DEG
YIELD STRESS OF TANK-              90000.  PSI
ELASTIC MODULUS OF TANK-        30000000.  PSI
UNIT WEIGHT OF TANK-                 490.0 LBS/CU-FT
POISSON'S RATIO-                   0.2500
WEAPON YIELD-                        5.00 MT
DEPTH OF WATER TABLE-                5.00 FEET
TANK LENGTH FACTOR-                  2.00
BURIAL DEPTH FACTOR-                 1.00

OVERPRESSURES( 1)            TANK DIAMETERS(12)
-----------------            ------------------
  1500.00 PSI                        8.00 FEET
                                    10.00 FEET
                                    12.00 FEET
                                    14.00 FEET
                                    16.00 FEET
                                    18.00 FEET
                                    20.00 FEET
                                    22.00 FEET
                                    24.00 FEET
                                    26.00 FEET
                                    28.00 FEET
                                    30.00 FEET
************************************************
COMPUTED CONSTANTS                     PAGE-  2
************************************************
COMPRESSION WAVE VELOCITY OF SOIL-  2002.6 FPS
COMPRESSION WAVE VELOCITY OF TANK- 18471.3 FPS
************************************************
INTERNAL CONSTANTS
************************************************
EXPONENTIAL N FACTOR-           0.1350
```



```
OVERPRESSURE       -    1500.00 PSI
DIAMETER OF TANK   -       8.00 FEET
LENGTH OF TANK     -      16.00 FEET
BURIAL DEPTH       -       8.00 FEET
WATER TABLE DEPTH -        5.00 FEET
*************************************************

DYNAMIC AMPLIFICATION FUNCTION FOR-
T(INITIAL)=     2.00416            T(TOTAL)=    2.00098
FACTOR USED=    2.00416
PRESSURE AMPLIFICATION FACTOR DUE TO WATER TABLE-    1.5011
DEADLOAD AND WATER PRESSURE=     4.44 PSI            INTERNAL PRESSURE =            733.18 PSI
CRITICAL PRESSURE =           1466.36 PSI            EFFECTIVE PRESSURE AT TANK=   2199.54 PSI
ALPHA =                       0.80950                ARCHING FACTOR =              0.40184
CALC. TANK THICKNESS=         0.67710 IN.
WEIGHT OF TANK =               12315. LBS.           THICKNESS USED=               0.75000 IN.
BUOYANCY EFFECT =              41820. LBS.  (TOTAL UPLIFT FORCE)
VOLUME OF EXCAVATION       =(CY)       216.
VOLUME OF SPECIAL BACKFILL =(CY)       134.
VOLUME OF ORDINARY BACKFILL=(CY)        57.
VOLUME OF TANK             =(CF)     670.20 OR      5013. GALS.        TOTAL COST OF TANK = ($)    13315.03
-----------------------------------------------------------------------------------------------------------
OVERPRESSURE       -    1500.00 PSI
DIAMETER OF TANK   -      10.00 FEET
LENGTH OF TANK     -      20.00 FEET
BURIAL DEPTH       -      10.00 FEET
WATER TABLE DEPTH -        5.00 FEET
*************************************************

DYNAMIC AMPLIFICATION FUNCTION FOR-
T(INITIAL)=     2.00348            T(TOTAL)=    2.00120
FACTOR USED=    2.00348
PRESSURE AMPLIFICATION FACTOR DUE TO WATER TABLE-    1.5011
DEADLOAD AND WATER PRESSURE=     5.87 PSI            INTERNAL PRESSURE =            733.42 PSI
CRITICAL PRESSURE =           1466.85 PSI            EFFECTIVE PRESSURE AT TANK=   2200.28 PSI
ALPHA =                       0.80950                ARCHING FACTOR =              0.40182
CALC. TANK THICKNESS=         0.84667 IN.
WEIGHT OF TANK =               22449. LBS.           THICKNESS USED=               0.87500 IN.
BUOYANCY EFFECT =              81681. LBS.  (TOTAL UPLIFT FORCE)
VOLUME OF EXCAVATION       =(CY)       354.
VOLUME OF SPECIAL BACKFILL =(CY)       198.
VOLUME OF ORDINARY BACKFILL=(CY)       107.
VOLUME OF TANK             =(CF)    1309.00 OR      9792. GALS.        TOTAL COST OF TANK = ($)    23577.65
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OVERPRESSURE      -    1500.00 PSI
DIAMETER OF TANK  -      12.00 FEET
LENGTH OF TANK    -      24.00 FEET
BURIAL DEPTH      -      12.00 FEET
WATER TABLE DEPTH -       5.00 FEET
**************************************************

DYNAMIC AMPLIFICATION FUNCTION FOR-
T(INITIAL)=    2.00217          T(TOTAL)=    2.00142
FACTOR USED=   2.00217
PRESSURE AMPLIFICATION FACTOR DUE TO WATER TABLE-    1.5011
DEADLOAD AND WATER PRESSURE=    7.30 PSI            INTERNAL PRESSURE =              733.42 PSI
CRITICAL PRESSURE =          1466.84 PSI            EFFECTIVE PRESSURE AT TANK=     2200.26 PSI
ALPHA =                      0.80950                ARCHING FACTOR =                0.40182
CALC. TANK THICKNESS=        1.01599 IN.
WEIGHT OF TANK =              41563. LBS.           THICKNESS USED=                 1.12500 IN.
BUOYANCY EFFECT =            141145. LBS.  (TOTAL UPLIFT FORCE)
VOLUME OF EXCAVATION       =(CY)     540.
VOLUME OF SPECIAL BACKFILL =(CY)     276.
VOLUME OF ORDINARY BACKFILL=(CY)     180.
VOLUME OF TANK             =(CF)   2261.95 OR     16921. GALS.        TOTAL COST OF TANK = ($)    41231.32
------------------------------------------------------------------------------------------------------------
OVERPRESSURE      -    1500.00 PSI
DIAMETER OF TANK  -      14.00 FEET
LENGTH OF TANK    -      28.00 FEET
BURIAL DEPTH      -      14.00 FEET
WATER TABLE DEPTH -       5.00 FEET
**************************************************

DYNAMIC AMPLIFICATION FUNCTION FOR-
T(INITIAL)=    2.00025          T(TOTAL)=    2.00163
FACTOR USED=   2.00163
PRESSURE AMPLIFICATION FACTOR DUE TO WATER TABLE-    1.5011
DEADLOAD AND WATER PRESSURE=    8.73 PSI            INTERNAL PRESSURE =              733.72 PSI
CRITICAL PRESSURE =          1467.45 PSI            EFFECTIVE PRESSURE AT TANK=     2201.17 PSI
ALPHA =                      0.80950                ARCHING FACTOR =                0.40180
CALC. TANK THICKNESS=        1.18581 IN.
WEIGHT OF TANK =              62858. LBS.           THICKNESS USED=                 1.25000 IN.
BUOYANCY EFFECT =            224134. LBS.  (TOTAL UPLIFT FORCE)
VOLUME OF EXCAVATION       =(CY)     780.
VOLUME OF SPECIAL BACKFILL =(CY)     370.
VOLUME OF ORDINARY BACKFILL=(CY)     277.
VOLUME OF TANK             =(CF)   3591.89 OR     26870. GALS.        TOTAL COST OF TANK = ($)    77026.18
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OVERPRESSURE       -    1500.00 PSI
DIAMETER OF TANK   -      16.00 FEET
LENGTH OF TANK     -      32.00 FEET
BURIAL DEPTH       -      16.00 FEET
WATER TABLE DEPTH -        5.00 FEET
*************************************************

DYNAMIC AMPLIFICATION FUNCTION FOR-
T(INITIAL)=     1.99775           T(TOTAL)=     2.00183
FACTOR USED=    2.00183
PRESSURE AMPLIFICATION FACTOR DUE TO WATER TABLE-     1.5011
DEADLOAD AND WATER PRESSURE=    10.16 PSI              INTERNAL PRESSURE =                 734.32 PSI
CRITICAL PRESSURE =           1468.64 PSI              EFFECTIVE PRESSURE AT TANK=       2202.97 PSI
ALPHA =                       0.80950                  ARCHING FACTOR =                  0.40176
CALC. TANK THICKNESS=         1.35632 IN.
WEIGHT OF TANK =                90310. LBS.            THICKNESS USED=                   1.37500 IN.
BUOYANCY EFFECT =              334567. LBS.  (TOTAL UPLIFT FORCE)
VOLUME OF EXCAVATION       =(CY)      1121.
VOLUME OF SPECIAL BACKFILL =(CY)       515.
VOLUME OF ORDINARY BACKFILL=(CY)       407.
VOLUME OF TANK             =(CF)    5361.66 OR     40110. GALS.         TOTAL COST OF TANK = ($)  110242.79
------------------------------------------------------------------------------------------------------------
OVERPRESSURE       -    1500.00 PSI
DIAMETER OF TANK   -      18.00 FEET
LENGTH OF TANK     -      36.00 FEET
BURIAL DEPTH       -      18.00 FEET
WATER TABLE DEPTH -        5.00 FEET
*************************************************

DYNAMIC AMPLIFICATION FUNCTION FOR-
T(INITIAL)=     1.99469           T(TOTAL)=     2.00202
FACTOR USED=    2.00202
PRESSURE AMPLIFICATION FACTOR DUE TO WATER TABLE-     1.5011
DEADLOAD AND WATER PRESSURE=    11.59 PSI              INTERNAL PRESSURE =                 734.92 PSI
CRITICAL PRESSURE =           1469.84 PSI              EFFECTIVE PRESSURE AT TANK=       2204.76 PSI
ALPHA =                       0.80950                  ARCHING FACTOR =                  0.40171
CALC. TANK THICKNESS=         1.52710 IN.
WEIGHT OF TANK =              14[illegible]471. LBS.   THICKNESS USED=                   1.75000 IN.
BUOYANCY EFFECT =              476367. LBS.  (TOTAL UPLIFT FORCE)
VOLUME OF EXCAVATION       =(CY)      1596.
VOLUME OF SPECIAL BACKFILL =(CY)       733.
VOLUME OF ORDINARY BACKFILL=(CY)       580.
VOLUME OF TANK             =(CF)    7634.08 OR     57110. GALS.         TOTAL COST OF TANK = ($)  188856.47
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OVERPRESSURE       -    1500.00 PSI
DIAMETER OF TANK   -      20.00 FEET
LENGTH OF TANK     -      40.00 FEET
BURIAL DEPTH       -      20.00 FEET
WATER TABLE DEPTH  -       5.00 FEET
*************************************************

DYNAMIC AMPLIFICATION FUNCTION FOR-
T(INITIAL)=    1.99110          T(TOTAL)=    2.00220
FACTOR USED=   2.00220
PRESSURE AMPLIFICATION FACTOR DUE TO WATER TABLE-    1.5011
DEADLOAD AND WATER PRESSURE=   13.02 PSI             INTERNAL PRESSURE =              735.51 PSI
CRITICAL PRESSURE =          1471.02 PSI             EFFECTIVE PRESSURE AT TANK=     2206.53 PSI
ALPHA =                      0.80950                 ARCHING FACTOR =                0.40167
CALC. TANK THICKNESS=        1.69815 IN.
WEIGHT OF TANK =             179594. LBS.            THICKNESS USED=                 1.75000 IN.
BUOYANCY EFFECT =            653452. LBS.  (TOTAL UPLIFT FORCE)
VOLUME OF EXCAVATION       =(CY)     2190.
VOLUME OF SPECIAL BACKFILL =(CY)     1005.
VOLUME OF ORDINARY BACKFILL=(CY)      796.
VOLUME OF TANK             =(CF)   10472.00 OR     78341. GALS.        TOTAL COST OF TANK = ($)  238523.47
----------------------------------------------------------------------------------------------------------
OVERPRESSURE       -    1500.00 PSI
DIAMETER OF TANK   -      22.00 FEET
LENGTH OF TANK     -      44.00 FEET
BURIAL DEPTH       -      22.00 FEET
WATER TABLE DEPTH  -       5.00 FEET
*************************************************

DYNAMIC AMPLIFICATION FUNCTION FOR-
T(INITIAL)=    1.98701          T(TOTAL)=    2.00238
FACTOR USED=   2.00238
PRESSURE AMPLIFICATION FACTOR DUE TO WATER TABLE-    1.5011
DEADLOAD AND WATER PRESSURE=   14.45 PSI             INTERNAL PRESSURE =              736.10 PSI
CRITICAL PRESSURE =          1472.20 PSI             EFFECTIVE PRESSURE AT TANK=     2208.30 PSI
ALPHA =                      0.80950                 ARCHING FACTOR =                0.40163
CALC. TANK THICKNESS=        1.86946 IN.
WEIGHT OF TANK =             248353. LBS.            THICKNESS USED=                 2.00000 IN.
BUOYANCY EFFECT =            869745. LBS.  (TOTAL UPLIFT FORCE)
VOLUME OF EXCAVATION       =(CY)     2915.
VOLUME OF SPECIAL BACKFILL =(CY)     1338.
VOLUME OF ORDINARY BACKFILL=(CY)     1060.
VOLUME OF TANK             =(CF)   13938.23 OR    104271. GALS.        TOTAL COST OF TANK = ($)  334939.57
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OVERPRESSURE       -     1500.00 PSI
DIAMETER OF TANK   -       24.00 FEET
LENGTH OF TANK     -       48.00 FEET
BURIAL DEPTH       -       24.00 FEET
WATER TABLE DEPTH  -        5.00 FEET
*************************************************

DYNAMIC AMPLIFICATION FUNCTION FOR-
T(INITIAL)=     1.98244           T(TOTAL)=    2.00255
FACTOR USED=    2.00255
PRESSURE AMPLIFICATION FACTOR DUE TO WATER TABLE-    1.5011
DEADLOAD AND WATER PRESSURE=     15.89 PSI            INTERNAL PRESSURE =              736.68 PSI
CRITICAL PRESSURE =            1473.37 PSI            EFFECTIVE PRESSURE AT TANK=     2210.06 PSI
ALPHA =                        0.80950                ARCHING FACTOR =                0.40159
CALC. TANK THICKNESS=          2.04104 IN.
WEIGHT OF TANK =               332506. LBS.           THICKNESS USED=                 2.25000 IN.
BUOYANCY EFFECT =             1129166. LBS.  (TOTAL UPLIFT FORCE)
VOLUME OF EXCAVATION      =(CY)     3784.
VOLUME OF SPECIAL BACKFILL =(CY)    1738.
VOLUME OF ORDINARY BACKFILL=(CY)    1376.
VOLUME OF TANK            =(CF)  18095.61 OR    135373. GALS.       TOTAL COST OF TANK = ($)  455487.19
------------------------------------------------------------------------------------------------------
OVERPRESSURE       -     1500.00 PSI
DIAMETER OF TANK   -       26.00 FEET
LENGTH OF TANK     -       52.00 FEET
BURIAL DEPTH       -       26.00 FEET
WATER TABLE DEPTH  -        5.00 FEET
*************************************************

DYNAMIC AMPLIFICATION FUNCTION FOR-
T(INITIAL)=     1.97741           T(TOTAL)=    2.00271
FACTOR USED=    2.00271
PRESSURE AMPLIFICATION FACTOR DUE TO WATER TABLE-    1.5011
DEADLOAD AND WATER PRESSURE=     17.32 PSI            INTERNAL PRESSURE =              737.27 PSI
CRITICAL PRESSURE =            1474.54 PSI            EFFECTIVE PRESSURE AT TANK=     2211.81 PSI
ALPHA =                        0.80950                ARCHING FACTOR =                0.40155
CALC. TANK THICKNESS=          2.21287 IN.
WEIGHT OF TANK =               390233. LBS.           THICKNESS USED=                 2.25000 IN.
BUOYANCY EFFECT =             1435635. LBS.  (TOTAL UPLIFT FORCE)
VOLUME OF EXCAVATION      =(CY)     4811.
VOLUME OF SPECIAL BACKFILL =(CY)    2210.
VOLUME OF ORDINARY BACKFILL=(CY)    1749.
VOLUME OF TANK            =(CF)  23006.98 OR    172115. GALS.       TOTAL COST OF TANK = ($)  546078.59
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OVERPRESSURE       -    1500.00 PSI
DIAMETER OF TANK   -      28.00 FEET
LENGTH OF TANK     -      56.00 FEET
BURIAL DEPTH       -      21.00 FEET
WATER TABLE DEPTH -        5.00 FEET
*************************************************

DYNAMIC AMPLIFICATION FUNCTION FOR-
T(INITIAL)=     1.97195            T(TOTAL)=    2.00286
FACTOR USED=    2.00285
PRESSURE AMPLIFICATION FACTOR DUE TO WATER TABLE-      1.5011
DEADLOAD AND WATER PRESSURE=    18.75 PSI              INTERNAL PRESSURE =              737.85 PSI
CRITICAL PRESSURE =           1475.70 PSI              EFFECTIVE PRESSURE AT TANK=     2213.55 PSI
ALPHA =                       0.80950                  ARCHING FACTOR =                0.40151
CALC. TANK THICKNESS=         2.38497 IN.
WEIGHT OF TANK =              502864. LBS.             THICKNESS USED=                 2.50000 IN.
BUOYANCY EFFECT =            1793074. LBS.  (TOTAL UPLIFT FORCE)
VOLUME OF EXCAVATION        =(CY)      6009.
VOLUME OF SPECIAL BACKFILL =(CY)       2760.
VOLUME OF ORDINARY BACKFILL=(CY)       2185.
VOLUME OF TANK              =(CF)  28735.16 OR    214967. GALS.       TOTAL COST OF TANK = ($)  714069.44
------------------------------------------------------------------------------------------------------
OVERPRESSURE       -    1500.00 PSI
DIAMETER OF TANK   -      30.00 FEET
LENGTH OF TANK     -      60.00 FEET
BURIAL DEPTH       -      30.00 FEET
WATER TABLE DEPTH -        5.00 FEET
*************************************************

DYNAMIC AMPLIFICATION FUNCTION FOR-
T(INITIAL)=     1.96608            T(TOTAL)=    2.00300
FACTOR USED=    2.00300
PRESSURE AMPLIFICATION FACTOR DUE TO WATER TABLE-      1.5011
DEADLOAD AND WATER PRESSURE=    20.18 PSI              INTERNAL PRESSURE =              738.42 PSI
CRITICAL PRESSURE =           1476.85 PSI              EFFECTIVE PRESSURE AT TANK=     2215.28 PSI
ALPHA =                       0.80950                  ARCHING FACTOR =                0.40146
CALC. TANK THICKNESS=         2.55732 IN.
WEIGHT OF TANK =              634994. LBS.             THICKNESS USED=                 2.75000 IN.
BUOYANCY EFFECT =            2205403. LBS.  (TOTAL UPLIFT FORCE)
VOLUME OF EXCAVATION        =(CY)      7392.
VOLUME OF SPECIAL BACKFILL =(CY)       3395.
VOLUME OF ORDINARY BACKFILL=(CY)       2688.
VOLUME OF TANK              =(CF)  35343.00 OR    264400. GALS.       TOTAL COST OF TANK = ($)  915150.93
------------------------------------------------------------------------------------------------------
```

SUMMARY SHEET NO. 1

OVERPRESSURE = 1500.00 PSI
WATER TABLE DEPTH = 5.00 FEET
LENGTH OF TANK = 2.00 X DIAMETER OF TANK
TANK BURIAL DEPTH = 1.00 X DIAMETER OF TANK

TANK DIA. (FEET)	CAPACITY (GALS)	TANK COST	EXCAV. COST	SPECIAL FILL COST	ORDINARY FILL COST	EXCAV. AND FILL COST	TOTAL COST
8.00	5013.	11083.53	1116.21	944.16	171.11	2231.49	13315.03
10.00	9792.	19830.20	2037.63	1386.25	323.55	3747.44	23577.65
12.00	16921.	35328.75	3429.00	1933.56	540.00	5902.57	41231.32
14.00	26870.	68096.15	5504.22	2594.69	831.11	8930.03	77026.18
16.00	40110.	96330.92	9083.29	3605.23	1223.33	13911.87	110242.79
18.00	57110.	167292.02	14689.38	5133.23	1741.82	21564.45	188856.47
20.00	78341.	206533.36	22559.29	7041.48	2389.33	31990.10	238523.47
22.00	104271.	289154.04	33233.12	9372.21	3180.20	45785.53	334939.57
24.00	135373.	391881.94	47308.80	12167.68	4128.76	63605.25	455487.19
26.00	172115.	459916.99	65442.09	15470.13	5249.36	86161.59	546078.59
28.00	214967.	599844.72	88346.56	19321.82	6556.33	114224.71	714069.44
30.00	264400.	766528.32	116793.60	23765.00	8064.00	148622.60	915150.93

APPENDIX B

COMPARISON OF EXPERIMENTAL RESULTS WITH EMPIRICAL EQUATIONS

B-1 Test Description

A series of static and dynamic tests were conducted on small scale stainless steel tanks, shown schematically in Figure B-1. The tanks consisted of a cylindrical section and two hemispherical end caps. The models were buried one diameter below the soil surface in a horizontal position. Internal pressurization was used to evaluate improvement in buckling resistance of the thin-walled structures.

Two static tests and two dynamic tests were conducted on identical models. The first of the static and dynamic test series were conducted at pressure levels low enough to prevent yielding and buckling of the tank. Tank stresses from these two tests (Static Test #8 and Dynamic Test #2) are presented in this appendix.

In the second static test, external pressures were increased until yield strains were reached and the tank buckled. The pressure was increased beyond the buckling point without further increase in load to the tank due to full transfer of load away from the tank through soil arching. Buckling of the tank did not produce a rupture.

Complete details of the entire test program are given in Reference 6.

B-2 Tank Stresses

For Static Test #8 an internal pressure of 100 psi was used to counterbalance an external soil surface pressure of 155 psi. The tank strains were monitored at eleven gages located around the perimeter of the cylinder at its centerline, see Figure B-1. Stresses computed from the test data are plotted in Figure B-2 on polar graph paper. The right half of the graph represents hoop stresses while the left half indicates meridional stresses. Two experimental plots are shown on each half. One, marked p_i (Exp.), is the plot of stresses induced under internal

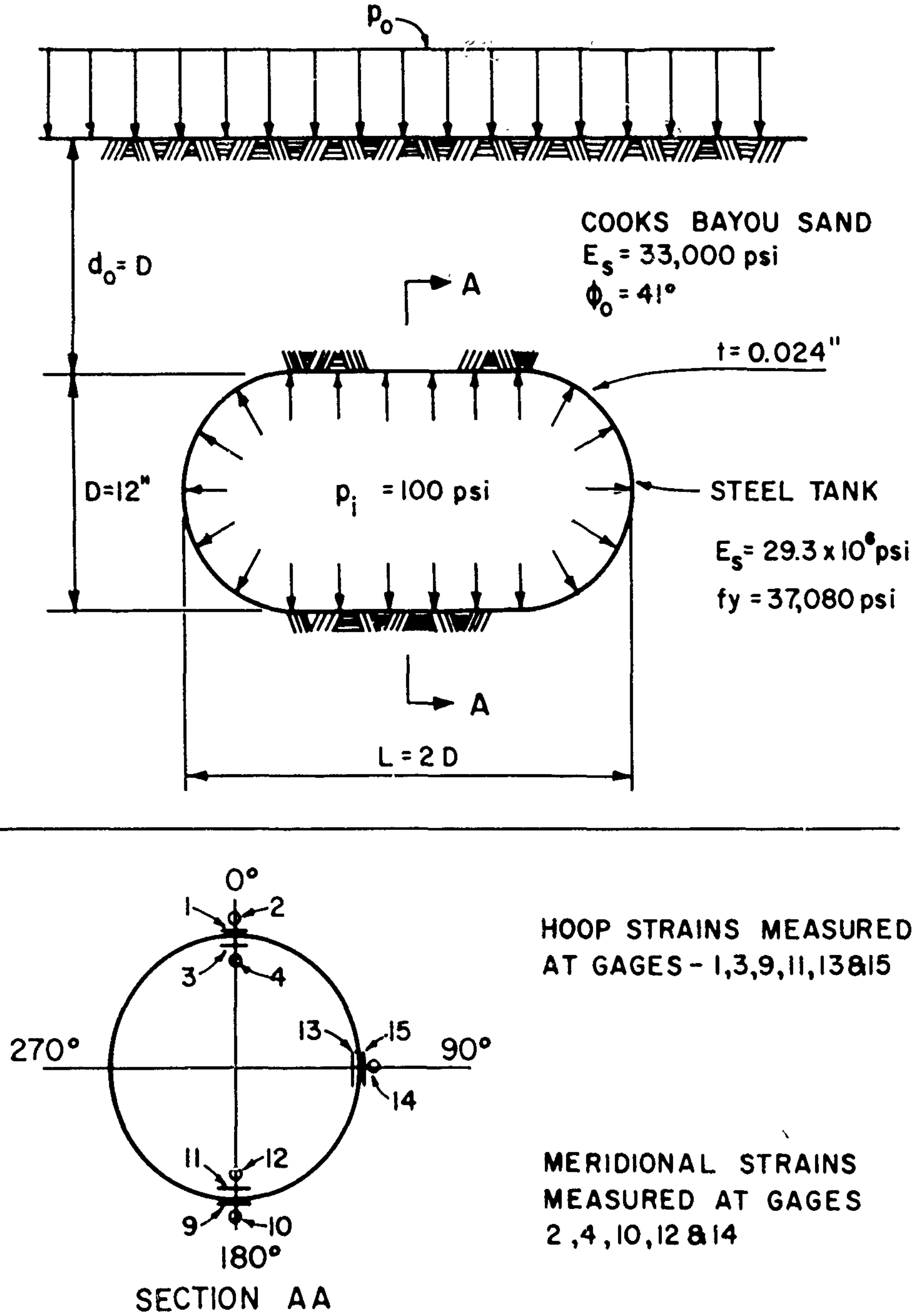


Figure B-1 MODEL TEST OF INTERNALLY PRESSURIZED BURIED FUEL TANKS - STATIC & DYNAMIC TESTS

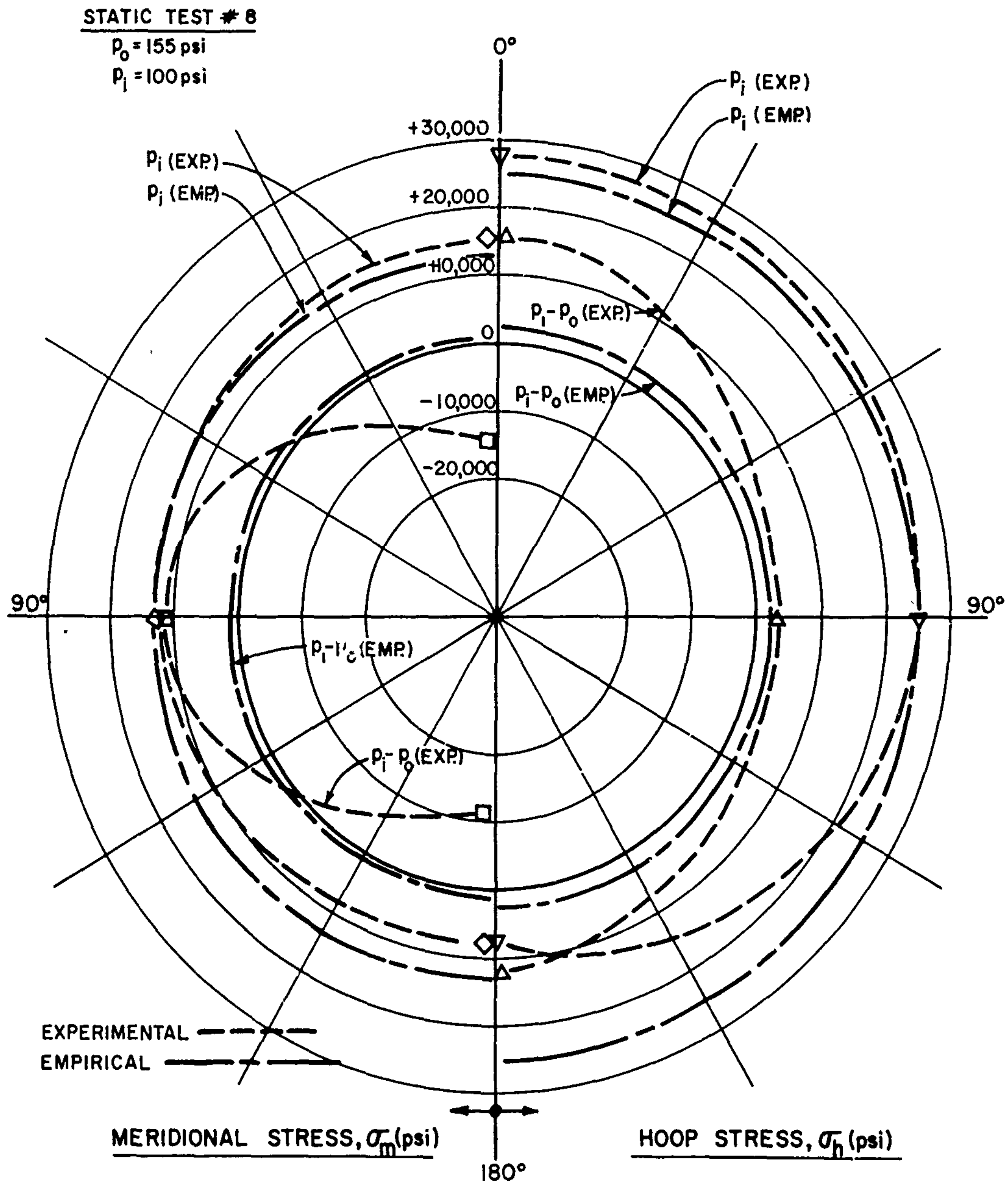


Figure B-2 COMPARISON OF TANK STRESSES FROM STATIC TEST RESULTS AND EMPIRICAL FORMULA

pressure loading (i.e. no external pressure). The second plot, marked $p_i - p_o$, (Exp.), represents the state of stress under combined external and internal pressure loading. A dashed line has been used to connect the recorded data points for purposes of readability. The actual magnitude of stresses between monitored values is unknown.

For comparison purposes, hoop and meridional stresses were computed using the empirical design formulas of Section 2. These results have been superimposed on the same graph in Figure B-2. By comparing the experimental and empirical plots it is apparent that the tank behaves as a pressure vessel under uniform internal pressure with the soil providing little or no resistance. Results from the model tests and the empirical formulas are virtually identical. The invert hoop stress measured experimentally appears to be a questionable record.

The empirical design equations of Section 2 are based on the assumption that the actual distribution of external pressures on the tank may be approximated by a uniform distribution. This assumption gives reasonable results in approximating the principle hoop stress which occurs at the spring line of the tank, see Figure B-2. The uniform pressure distribution assumption is an upper bound on all hoop compressive stresses. However, in the meridional direction this assumption underestimates the compressive stress in local regions near the crown and invert. It was in these local zones, at the junction of cylinder with hemispherical end cap that buckling of the tank occurred during the second cycle of static loading - to failure.

Results from the dynamic test are given in Figure B-3. A rectangular pressure pulse with nearly instantaneous rise time was used to generate the dynamic surface pressure, p_o, which had a peak value of 150 psi. The tank was internally pressurized to 100 psi. For comparison purposes, peak stresses were computed in the hoop and meridional directions using the empirical relationships of Section 2 .

From a comparison of Figure B-2 and B-3, it is apparent that the stress patterns for static and dynamic loading are nearly identical. The only variation between the two cases is in the magnitude of the stresses. Since internal pressures and peak external surface pressures for both tests were virtually the same in magnitude,

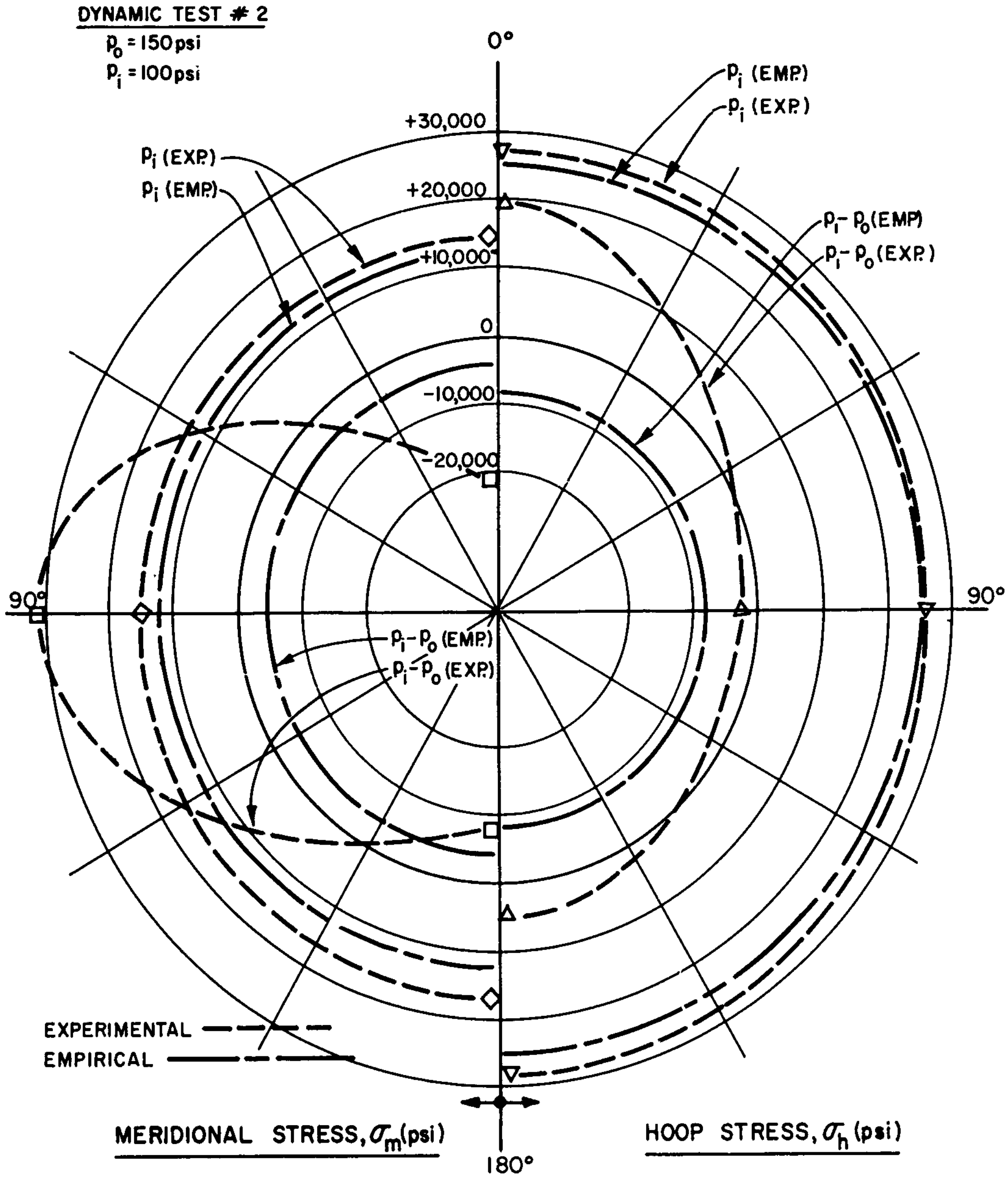


Figure B-3 COMPARISON OF TANK STRESSES FROM DYNAMIC TEST RESULTS AND EMPIRICAL FORMULA

the basic difference in the resulting stresses is due to amplification resulting from dynamically applied loads. The dynamic amplification factor, DF, in this case is 1.5.

As seen in Figure B-3, for the dynamic case, the empirical equations conservatively bound the experimental values of peak hoop stress. In the meridional direction, however, the empirical equations underestimate the dynamic test results by 100%. From a design standpoint, this is not as serious as it may appear, since hoop stresses will normally be larger in magnitude than meridional stresses. Thus, sizing of the plate thickness will be dictated primarily by the hoop stress.

B-3 Arching

From the static test results arching factors were computed which ranged from 0.32 to 0.68. For the first cycle of external pressure loading, the arching factor ranged from 0.32 to 0.39 while in the second cycle it ranged from 0.52 to 0.68. Thus, the first cycle of loading compacted the soil, so that subsequent cycles loaded a significantly stiffer soil medium. As noted in Section 2-2.6, arching is a function of the relative stiffness between the tank and the surrounding soil. As the soil increased in stiffness, more of the load was transferred away from the tank to adjacent soil by shear. Arching factors from the dynamic tests ranged in value from 0.34 to 0.37.

For comparison purposes, the arching factor was computed using the empirical equations of Section 2-2.6. The resulting arching factor, A, was 0.41, which compares quite well with the experimental values from the first cycle of static loading and both of the dynamic test cases.

B-4 Buckling

The empirically determined buckling capacity of the tank as given by Chelapati's [4] formula from Section 2-4. is

$$p_{cr} = 0.612\sqrt{\frac{E_s E_t}{(r/t)^3}} = 0.612\sqrt{\frac{(33{,}000\ \text{psi})(29.3\text{x}10^6\ \text{psi})}{(6\ \text{in}/0.024\ \text{in})^3}}$$

$$p_{cr} = 152\ \text{psi}$$

while actual buckling occurred under a net external compressive pressure of 498 psi. The tank had an internal pressure of only 13 psig at the time it buckled.

From these and other test results on vertical tanks[6] it is apparent that the Chelapati buckling equation gives conservative results for internally pressurized tanks of the general geometry tested.

APPENDIX C

RECOMMENDED SOILS INVESTIGATION

C-1 Introduction

This appendix presents a series of recommended soil investigations which are applicable to the design of underground fuel storage tanks. These recommendations were prepared by Mr. H. Klehn, a member of the firm of Dames and Moore, consulting engineers in the applied earth sciences, Los Angeles, California.

C-2 Design Requirements

The proposed facilities will consist of steel storage tanks placed below ground surface. The tanks will be designed to resist high nuclear blast pressures. They may range in diameter from 10 to 30 feet, and in length from 20 to 60 feet and be buried either in a horizontal or vertical orientation. It is estimated that the bottom of the tank will be from 20 feet to 65 feet below the existing ground surface. Subsurface materials may include dry granular soils, saturated granular soils, or relatively shallow bedrock.

The method of placement would be in open excavations with required dewatering. The soil conditions will normally be improved by overexcavation and backfilling with high quality compacted fill soil.

A comprehensive series of studies will be required to provide criteria for design of the vessels to resist the nuclear blast pressures. Under these loading conditions it will be necessary to know the properties of the subsurface soil and rock materials when subjected to dynamic forces at relatively high pressures. Of particular interest are the dynamic secant modulus, compressional velocity, and Poisson's Ratio.

C-3 Recommended Soils Engineering and Engineering Geology Investigation

C-3.1 General

Because of the close relationship of the soils engineering and engineering geology studies, these studies should be done concurrently, as part of one investigation performed by a single consulting group. The investigation would include the following elements:

1. Siting studies.

2. Geological mapping survey.

3. Subsurface exploratory drilling and sampling.

4. Geophysical survey.

5. Laboratory testing.

6. Engineering analysis and preparation of report outlining specific recommendations for use by the design engineer.

The final report of the investigation should be prepared jointly by the soils engineer and the geologist, and should contain specific recommendations along with supporting data. The results of all tests should be included in the final report.

C-3.2 <u>Site Selection</u>

The siting team should include the soils engineer and engineering geologist. Prior to making the site selections, all available geological and soils data should be reviewed. This information would form the basis of preliminary opinions concerning the probable soil and rock conditions.

During the siting team's tour of the project area, the soils engineer and the engineering geologist should advise the rest of the team concerning the following:

1. Probable thickness of overburden soils.

2. Probable depth to ground water.

3. Type of soil and bedrock.

4. Engineering characteristics of the subsurface materials.

It is anticipated that the information concerning the subsurface conditions will have a substantial influence on final site selection.

C-3.3 Geological Survey

A detailed geological survey of each of the sites will be required. The engineering geologist should have available to him topographic maps at the approximate scale of 1 inch equals 200 feet. The contour interval will depend on the site terrain; however, the interval should not exceed 5 feet in any case.

It is recommended that both black and white, and color aerial photographs be obtained specifically for this investigation. This would be stereoscopic coverage extending at least one mile beyond the site limits. The scale of the photographs will depend on the terrain and site geology; however, the direct photographic prints should be no smaller than a scale of about 1 inch equals 800 feet.

The engineering geologist should make a complete inspection of all erosional features, rock outcrops, excavation and road cuts. In addition, test pits should be excavated as required to further define the geologic conditions at the site.

The engineering geology map of each site should include the boundaries of the soil and rock types, the orientation of joints, bedding and shear zones in the rock, and the extent of all landslides, serious erosion or other areas of past or potential instability.

Following the initial geological mapping, the geologist should assist the soils engineer in selecting locations for the subsurface exploratory borings. In addition, the geophysical survey should be planned to further define the subsurface conditions. Sufficient subsurface exploration should be completed to define any suspected or known subsurface geological structure.

It should be the responsibility of the engineering geologist to prepare subsurface sections detailing the site conditions.

The geological studies should include a review of all ground water information available for the area. This description should include anticipated seasonal ground water fluctuations, as well as the conductivity of the ground water, and presence of corrosive chemicals. Sufficient subsurface exploration should be performed to completely describe the local ground water conditions.

The engineering geologist should present his opinions concerning the suitability of the site for construction of the proposed facilities, along with his evaluation of the engineering properties of the rock and soil materials.

C-3.4 Drilling and Sampling Program

The purpose of the drilling and sampling program will be to explore the subsurface conditions. During the program, sufficient core samples should be obtained for laboratory testing and classification of all soils and rock.

The soils engineer and engineering geologist should lay out the locations of all borings. During drilling, any adjustments or additions to the drilling program should be made in the field.

The depths of the borings will depend to some extent on the materials encountered. However, for planning purposes, it is expected that the borings will be in the range of 50 feet to 100 feet deep.

There are many types of drilling equipment which could be used. It should be the responsibility of the soils engineer and the engineering geologist to jointly select the proper type of drilling equipment to accomplish the work at each site.

Each piece of drilling equipment should be under the continuous supervision of either the soils engineer or the engineering geologist. This professional would be responsible for the drilling and sampling operations and would maintain an accurate log of each boring.

Several alternate methods of sampling should be available. The actual method used will depend on the type of sample desired and the soil or rock encountered.

Representative, undisturbed samples of relatively dense granular materials and firm to stiff cohesive materials can be obtained using a sampler of the type shown in Figure C-1. When a sampler of this type is driven into the soil, the number of blows required to cause the

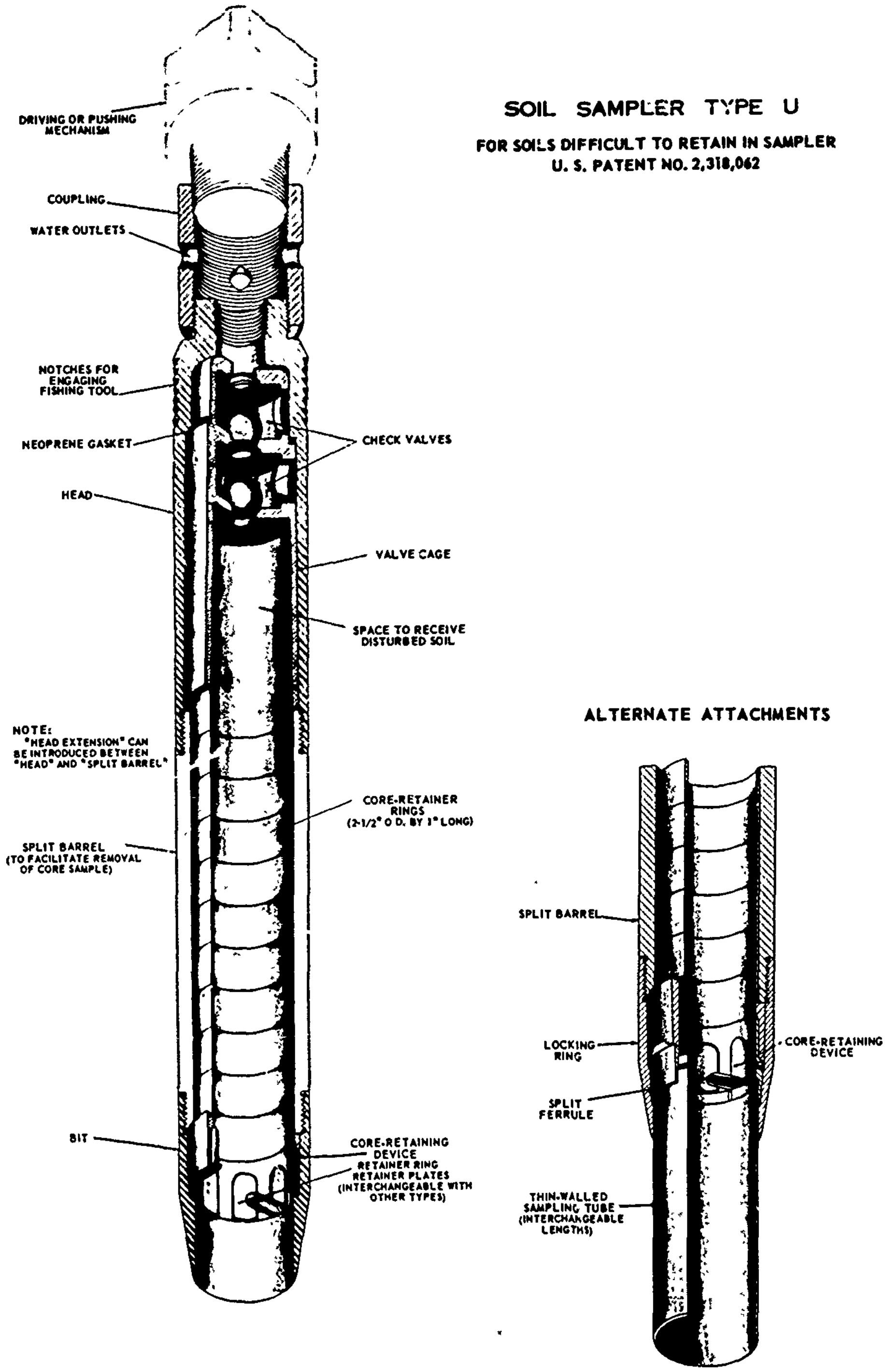


Figure C-1 SOIL SAMPLER

sampler to penetrate each foot when driven with a calibrated weight should be recorded. This driving resistance can be used to correlate samples between borings. In addition, the blow count provides a means for estimating the relative density of the soils, soil strength and other soil properties.

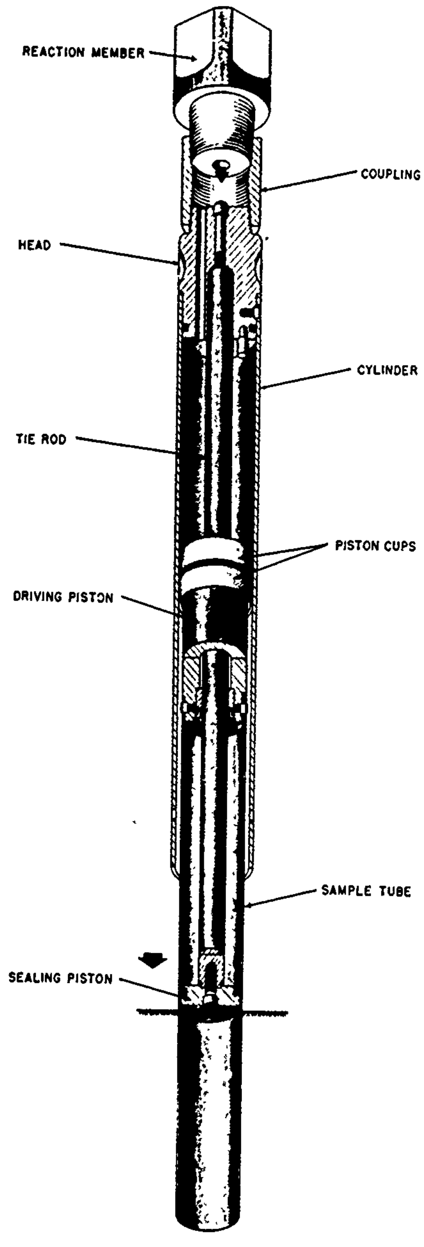
Where soft or sensitive soils are encountered, a piston or Shelby tube sampler, of the type shown on Figure C-2, should be utilized. This type of sampler has the advantage of a bit with a thin cutting edge which results in less sample disturbance during the sampling operations. This type of sampler also should be used when attempting to obtain accurate density samples of loose granular materials for the purpose of evaluating the relative density.

Depending on the type of soils, it may be desirable to perform field tests during the drilling program. Field tests provide a means of evaluating the in situ physical properties of the soil. This is particularly important when suitable undisturbed samples of the soil materials cannot be obtained.

At a minimum, standard penetration tests (American Society for Testing and Materials, Designation D1586-63T) should be performed in each boring at intervals of not greater than 10 feet when drilling in granular soils. In this test, a sampler similar to that shown on Figure C-1 is driven with a hammer weighing 140 pounds, dropping from a height of 30 inches. The number of blows required to cause the sampler to penetrate 1 foot can be used to estimate relative density of sandy and gravelly soils.

Another field test which may be considered is the static penetrometer test, similar to the Dutch cone penetration test. This test procedure is described in a paper by Plantema in the *Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering, 1948*. This test is a measure of the static resistance to penetration of the undisturbed soils at the bottom of the boring. A penetrometer is provided with a cone-shaped tip, and the force required to cause penetration is recorded. This test is particularly useful in cohesive soils and has been correlated with various soil properties, including the soil strength.

A third test which may be useful in measuring the in situ soil properties is the pressure meter test. This test



PISTON SAMPLER

THE DAMES & MOORE PISTON SAMPLER HAS BEEN DEVELOPED TO OBTAIN SAMPLES OF SOFT SOILS WITH A MINIMUM OF DISTURBANCE. THE MOST SIGNIFICANT FEATURES ARE THE SEALING PISTON WHICH CONFINES THE SOIL DURING SAMPLING AND THE SAMPLE TUBE WHICH HAS A WALL THICKNESS OF ONLY 0.042 INCHES.

AT THE START OF THE SAMPLING, THE LOWER END OF THE SAMPLE TUBE IS ADJACENT TO THE SEALING PISTON AT THE BOTTOM OF AN EXPLORATION TEST BORING. THE SEALING PISTON, CYLINDER, HEAD, AND REACTION MEMBER REMAIN STATIONARY DURING SAMPLING. COMPRESSED AIR, COMPRESSED NITROGEN, OR WASH WATER ARE FORCED INTO THE CYLINDER THROUGH THE SAMPLING RODS FROM THE DRILLING EQUIPMENT. THE DRIVING PISTON MOVES THE SAMPLE TUBE DOWNWARD INTO THE SOIL.

Figure C-2 PISTON SAMPLER

utilizes an inflatable tube or cylinder which is introduced into the boring. As the tube is expanded against the walls of the boring, the pressure required to cause a unit strain or deflection of the soils forming the walls is measured. This stress-strain relationship is a function of soil strength and compressibility and has been correlated with these properties.

If low strength silts or clays are encountered, the shear strength and stress-strain characteristics of these soils can be measured in situ by the vane shear test. For this test, a steel vane is pushed into the soil at the bottom of the boring, and the torque required to rotate the vane is recorded.

Where bedrock is encountered, it is recommended that samples be obtained by using NX coring tools. NX coring tools provide a clean rock core, 2-1/8 inches in diameter. This has become a standard core size, and provides rock samples which can be conveniently tested in the laboratory. A great deal of information is available from tests on NX core samples for correlation of physical properties. Utilizing cores of other diameters would influence the test values and make it difficult to access rock properties based on information available from previous work.

The rock should be continuously cored, and an effort should be made to obtain 100 percent core recovery. An accurate log of all bedrock cores should be maintained, and the cores should be photographed immediately upon extraction from the boring. The log should include the number, inclination and spacing of joints or fracture planes observed in the bedrock cores.

The RQD index should be recorded as described by Professor D.U. Deere of the University of Illinois. Professor Deere has developed this index value as a measure of the overall massiveness and competency of bedrock materials. Utilizing his method, the cumulative length of individual core pieces longer than 4 inches is reported as a percent of the total length of core run. This is referred to as the RQD index.

All soil samples and rock cores should be packaged in watertight and vapor-tight containers, and stored in a protected place prior to laboratory testing. Since many of the physical properties of soil and rock are dependent on the moisture content of the materials, every effort should be made to maintain the samples at the field moisture conditions.

During the drilling operations, it may be desirable to perform pumping tests to evaluate the permeability and hydraulic characteristics of the subsurface materials. This would be particularly important where dewatering of deep excavations is anticipated during construction of the proposed facilities. The method and duration of the pump tests should be reviewed by the design engineer prior to initiation of the tests. At least two observation wells should be placed in the vicinity of the pumped well to permit accurate measurement of the drawdown of the water level during test pumping.

The field log maintained for each boring should include the following information:

1. The date, type of drilling equipment, and supervising engineer or geologist.

2. The drilling contractor's name and address.

3. Boring locations in terms of distances or coordinates to the nearest foot in plan dimension.

4. Elevation of the ground surface at each boring location to an accuracy of 0.10 foot.

5. Type of sampling, depth, length of sample or core attempted, the percent recovery, and the RQD index for rock cores.

6. Field classification of materials drilled and sampled.

7. A visual field evaluation of the relative firmness or density of the sampled materials.

8. The loss of drilling fluid circulation if rotary-wash drilling methods are utilized.

9. Observations regarding ground water flow into holes through fractures or permeable soils.

10. Initial and static water level measurements obtained daily for a period of at least two weeks following completion of each boring.

11. Results of in situ tests.

C-3.5 Geophysical Survey

The geophysical survey will provide additional information concerning the depth and type of soil and rock beneath the sites. In addition, the velocities with which shock waves pass through the subsurface soil and rock are measured as part of the geophysical exploration program.

It is important that the uniformity of subsurface conditions be known for quite a distance surrounding the actual site to be developed. This information is needed in the finite element analysis and to estimate the approach of outrunning shock waves through the subsurface materials.

It is recommended that the geophysical program include a minimum of two seismic refraction profiles or lines at each site. Generally, these lines should be at right angles to each other, centered on the approximate center of the proposed structure. The lines should extend approximately 1,000 feet beyond the site limits.

For the finite element analysis, the seismic refraction survey should include measurement of both the compression wave and the shear wave velocities of the soil and rock present in the subsurface profile to a minimum of three times the depth of the bottom of the structures.

As part of the geophysical program, measurements of uphole velocities should be made. These measurements are useful in evaluating the dynamic properties of the subsoils in the immediate vicinity of the proposed structures. It is recommended that the uphole velocity be measured in the exploration borings which are drilled to a minimum depth of 50 feet below the proposed lowest point of excavation.

The results of the geophysical explorations should be correlated with the engineering geology survey and with the results of the drilling program. Any structural irregularities or other poorly defined subsurface conditions should be explored by drilling.

C-3.6 Laboratory Testing

Static Tests: Conventional soils engineering tests are used to define the density, moisture content, strength, compressibility and permeability of the subsurface soils. When known, these basic soil properties permit evaluation of the performance of subsurface soil and rock materials when subjected to static loads.

It is recommended that the moisture content and dry density be measured for all samples. This soil property is easily obtained and is useful in approximating soil strength and compressibility.

For granular samples, it is important to know the relative density of the soils. It is recommended, therefore, that the minimum and maximum densities of representative samples of granular soils be measured. Knowing these two values, and the in-place density of the soils, it is possible to calculate the relative density of the in situ soils. From the relative density, the compressibility of the soils under load can be estimated. It is possible also to evaluate the static subgrade modulus of these soils and the probable soil strength from the relative density. In addition, the susceptibility of saturated granular soils to liquefy under shock loading can be estimated, based on the relative density of the soil.

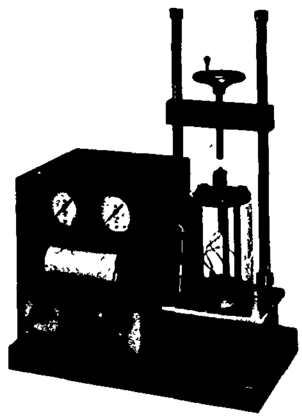
Depending on the type of soil, i.e. granular soil or cohesive soil, one or more types of strength tests should be performed. For cohesive soils it is recommended that triaxial compression tests or unconfined compression tests be performed as described in Figure C-3. If the soils are saturated, the pore water pressures should be measured during tests.

In the case of granular samples, triaxial compression tests could be utilized. Ordinarily, however, the static shearing strength can be adequately measured using direct shear tests. The method of direct shear tests is described in Figure C-4.

In addition to performing pumping tests in the field to measure the permeability and hydraulic properties of the soils, it is sometimes advisable to perform permeability tests in the laboratory. The method of performing these tests is described in Figure C-5. Basically, this test is a measure of the relative ease or difficulty with which water passes through the sample. The results of

METHODS OF PERFORMING UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS

THE SHEARING STRENGTHS OF SOILS ARE DETERMINED FROM THE RESULTS OF UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS. IN TRIAXIAL COMPRESSION TESTS THE TEST METHOD AND THE MAGNITUDE OF THE CONFINING PRESSURE ARE CHOSEN TO SIMULATE ANTICIPATED FIELD CONDITIONS.



TRIAXIAL COMPRESSION TEST UNIT

UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS ARE PERFORMED ON UNDISTURBED OR REMOLDED SAMPLES OF SOIL APPROXIMATELY SIX INCHES IN LENGTH AND TWO AND ONE-HALF INCHES IN DIAMETER. THE TESTS ARE RUN EITHER STRAIN-CONTROLLED OR STRESS-CONTROLLED. IN A STRAIN-CONTROLLED TEST THE SAMPLE IS SUBJECTED TO A CONSTANT RATE OF DEFLECTION AND THE RESULTING STRESSES ARE RECORDED. IN A STRESS-CONTROLLED TEST THE SAMPLE IS SUBJECTED TO EQUAL INCREMENTS OF LOAD WITH EACH INCREMENT BEING MAINTAINED UNTIL AN EQUILIBRIUM CONDITION WITH RESPECT TO STRAIN IS ACHIEVED.

YIELD, PEAK, OR ULTIMATE STRESSES ARE DETERMINED FROM THE STRESS-STRAIN PLOT FOR EACH SAMPLE AND THE PRINCIPAL STRESSES ARE EVALUATED. THE PRINCIPAL STRESSES ARE PLOTTED ON A MOHR'S CIRCLE DIAGRAM TO DETERMINE THE SHEARING STRENGTH OF THE SOIL TYPE BEING TESTED.

UNCONFINED COMPRESSION TESTS CAN BE PERFORMED ONLY ON SAMPLES WITH SUFFICIENT COHESION SO THAT THE SOIL WILL STAND AS AN UNSUPPORTED CYLINDER. THESE TESTS MAY BE RUN AT NATURAL MOISTURE CONTENT OR ON ARTIFICIALLY SATURATED SOILS.

IN A TRIAXIAL COMPRESSION TEST THE SAMPLE IS ENCASED IN A RUBBER MEMBRANE, PLACED IN A TEST CHAMBER, AND SUBJECTED TO A CONFINING PRESSURE THROUGHOUT THE DURATION OF THE TEST. NORMALLY, THIS CONFINING PRESSURE IS MAINTAINED AT A CONSTANT LEVEL, ALTHOUGH FOR SPECIAL TESTS IT MAY BE VARIED IN RELATION TO THE MEASURED STRESSES. TRIAXIAL COMPRESSION TESTS MAY BE RUN ON SOILS AT FIELD MOISTURE CONTENT OR ON ARTIFICIALLY SATURATED SAMPLES. THE TESTS ARE PERFORMED IN ONE OF THE FOLLOWING WAYS:

UNCONSOLIDATED-UNDRAINED: THE CONFINING PRESSURE IS IMPOSED ON THE SAMPLE AT THE START OF THE TEST. NO DRAINAGE IS PERMITTED AND THE STRESSES WHICH ARE MEASURED REPRESENT THE SUM OF THE INTERGRANULAR STRESSES AND PORE WATER PRESSURES.

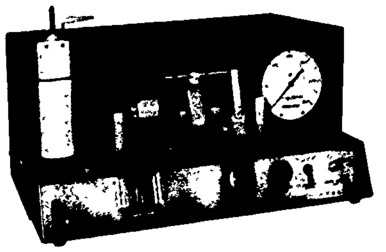
CONSOLIDATED-UNDRAINED: THE SAMPLE IS ALLOWED TO CONSOLIDATE FULLY UNDER THE APPLIED CONFINING PRESSURE PRIOR TO THE START OF THE TEST. THE VOLUME CHANGE IS DETERMINED BY MEASURING THE WATER AND/OR AIR EXPELLED DURING CONSOLIDATION. NO DRAINAGE IS PERMITTED DURING THE TEST AND THE STRESSES WHICH ARE MEASURED ARE THE SAME AS FOR THE UNCONSOLIDATED-UNDRAINED TEST.

DRAINED: THE INTERGRANULAR STRESSES IN A SAMPLE MAY BE MEASURED BY PERFORMING A DRAINED, OR SLOW, TEST. IN THIS TEST THE SAMPLE IS FULLY SATURATED AND CONSOLIDATED PRIOR TO THE START OF THE TEST. DURING THE TEST, DRAINAGE IS PERMITTED AND THE TEST IS PERFORMED AT A SLOW ENOUGH RATE TO PREVENT THE BUILDUP OF PORE WATER PRESSURES. THE RESULTING STRESSES WHICH ARE MEASURED REPRESENT ONLY THE INTERGRANULAR STRESSES. THESE TESTS ARE USUALLY PERFORMED ON SAMPLES OF GENERALLY NON-COHESIVE SOILS, ALTHOUGH THE TEST PROCEDURE IS APPLICABLE TO COHESIVE SOILS IF A SUFFICIENTLY SLOW TEST RATE IS USED.

AN ALTERNATE MEANS OF OBTAINING THE DATA RESULTING FROM THE DRAINED TEST IS TO PERFORM AN UNDRAINED TEST IN WHICH SPECIAL EQUIPMENT IS USED TO MEASURE THE PORE WATER PRESSURES. THE DIFFERENCES BETWEEN THE TOTAL STRESSES AND THE PORE WATER PRESSURES MEASURED ARE THE INTERGRANULAR STRESSES.

Figure C-3 UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS

Method Of Performing Direct Shear and Friction Tests



DIRECT SHEAR TESTING & RECORDING APPARATUS

DIRECT SHEAR TESTS ARE PERFORMED TO DETERMINE THE SHEARING STRENGTHS OF SOILS. FRICTION TESTS ARE PERFORMED TO DETERMINE THE FRICTIONAL RESISTANCES BETWEEN SOILS AND VARIOUS OTHER MATERIALS SUCH AS WOOD, STEEL, OR CONCRETE. THE TESTS ARE PERFORMED IN THE LABORATORY TO SIMULATE ANTICIPATED FIELD CONDITIONS.

EACH SAMPLE IS TESTED WITHIN THREE BRASS RINGS, TWO AND ONE-HALF INCHES IN DIAMETER AND ONE INCH IN LENGTH. UNDISTURBED SAMPLES OF IN-PLACE SOILS ARE TESTED IN RINGS TAKEN FROM THE SAMPLING DEVICE IN WHICH THE SAMPLES WERE OBTAINED. LOOSE SAMPLES OF SOILS TO BE USED IN CONSTRUCTING EARTH FILLS ARE COMPACTED IN RINGS TO PREDETERMINED CONDITIONS AND TESTED.

Direct Shear Tests

A THREE-INCH LENGTH OF THE SAMPLE IS TESTED IN DIRECT DOUBLE SHEAR. A CONSTANT PRESSURE, APPROPRIATE TO THE CONDITIONS OF THE PROBLEM FOR WHICH THE TEST IS BEING PERFORMED, IS APPLIED NORMAL TO THE ENDS OF THE SAMPLE THROUGH POROUS STONES. A SHEARING FAILURE OF THE SAMPLE IS CAUSED BY MOVING THE CENTER RING IN A DIRECTION PERPENDICULAR TO THE AXIS OF THE SAMPLE. TRANSVERSE MOVEMENT OF THE OUTER RINGS IS PREVENTED.

THE SHEARING FAILURE MAY BE ACCOMPLISHED BY APPLYING TO THE CENTER RING EITHER A CONSTANT RATE OF LOAD, A CONSTANT RATE OF DEFLECTION, OR INCREMENTS OF LOAD OR DEFLECTION. IN EACH CASE, THE SHEARING LOAD AND THE DEFLECTIONS IN BOTH THE AXIAL AND TRANSVERSE DIRECTIONS ARE RECORDED AND PLOTTED. THE SHEARING STRENGTH OF THE SOIL IS DETERMINED FROM THE RESULTING LOAD-DEFLECTION CURVES.

Friction Tests

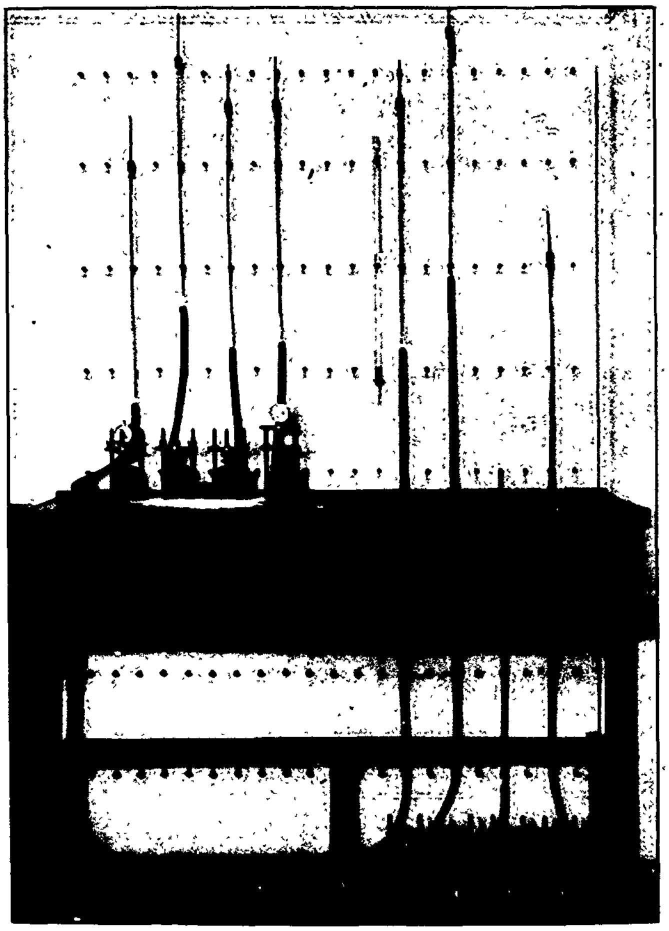
IN ORDER TO DETERMINE THE FRICTIONAL RESISTANCE BETWEEN SOIL AND THE SURFACES OF VARIOUS MATERIALS, THE CENTER RING OF SOIL IN THE DIRECT SHEAR TEST IS REPLACED BY A DISK OF THE MATERIAL TO BE TESTED. THE TEST IS THEN PERFORMED IN THE SAME MANNER AS THE DIRECT SHEAR TEST BY FORCING THE DISK OF MATERIAL FROM THE SOIL SURFACES.

Figure C-4 DIRECT SHEAR AND FRICTION TESTS

METHOD OF PERFORMING PERCOLATION TESTS

The quantity and the velocity of flow of water which will escape through an earth structure or percolate through soil are dependent upon the permeability of the earth structure or soil. The permeability of soil has often been calculated by empirical formulas but is best determined by laboratory tests, especially in the case of compacted soils.

A one-inch length of the core sample is sealed in the percolation apparatus, placed under a confining load, or surcharge pressure, and subjected to the pressure of a known head of water. The percolation rate is computed from the measurements of the volume of water which flows through the sample in a series of time intervals. These rates are usually expressed as the velocity of flow in feet per year under a hydraulic gradient of one and at a temperature of 20 degrees Centigrade. The rate so expressed may be adjusted for any set of conditions involving the same soil by employing established physical laws. Generally, the percolation rate varies over a wide range at the beginning of the test and gradually approaches equilibrium as the test progresses.



APPARATUS FOR PERFORMING PERCOLATIONS TESTS
Shows tests in progress on eight samples simultaneously.

During the performance of the test, continuous readings of the deflection of the sample are taken by means of micrometer dial gauges. The amount of compression or expansion, expressed as a percentage of the original length of the sample, is a valuable indication of the compression of the soil which will occur under the action of load or the expansion of the soil as saturation takes place.

Figure C-5 PERCOLATION TEST

the permeability tests are useful in evaluating the dewatering requirements for excavations during construction. In addition, the potential for the soils to liquefy under shock loading is a function of permeability.

Consolidation tests are a measure of the one-dimensional compressibility of the soil when completely confined. This test is performed as described in Figure C-6. Typically, this test permits evaluation of the settlement or consolidation of the soil when subjected to static loads. However, the test can be performed wherein cyclic loads are applied. In this case, it is a measure of the compressibility of the soil under confined conditions when subjected to momentary loads.

The electrical resistivity and corrosion potential of the samples of soil and rock materials should be measured. These tests are useful in anticipating the risk of deterioration of buried steel and concrete structures.

<u>Dynamic Tests</u>: Because of the requirement for the vessels to withstand shock loading, several unique laboratory tests will be required. Of primary interest are the stress-strain properties of the soils and rock under the range of dynamic loads anticipated. As stated earlier, shock loading may be as high as 3,000 pounds per square inch in the form of a single shock wave passing over the site surface.

Ideally, the dynamic tests would be performed under conditions which closely simulate anticipated field loading conditions. Pressures of 3,000 pounds per square inch, or pressures approaching this, are unusual in normal soil testing. It is believed, however, that suitable triaxial compression testing equipment could be developed to test samples at confining pressures on the order of 3,000 pounds per square inch or less, if necessary.

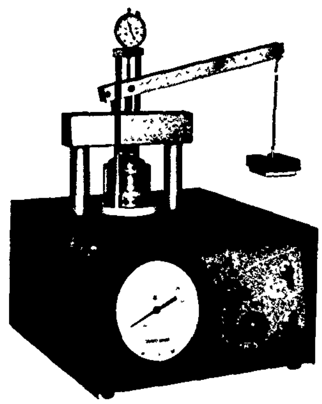
However, before these high pressure tests are performed, the more conventional dynamic testing equipment should be utilized. Tests would be performed in existing dynamic triaxial equipment which has a capability for testing in the range of 100 to 200 pounds per square inch confining pressures. The type of equipment used for these tests is shown in Figure C-7.

The test involves taking an undisturbed sample or a laboratory prepared sample of the on-site soil and applying an all around confining pressure. Typically, the

METHOD OF PERFORMING CONSOLIDATION TESTS

CONSOLIDATION TESTS ARE PERFORMED TO EVALUATE THE VOLUME CHANGES OF SOILS SUBJECTED TO INCREASED LOADS. TIME-CONSOLIDATION AND PRESSURE-CONSOLIDATION CURVES MAY BE PLOTTED FROM THE DATA OBTAINED IN THE TESTS. ENGINEERING ANALYSES BASED ON THESE CURVES PERMIT ESTIMATES TO BE MADE OF THE PROBABLE MAGNITUDE AND RATE OF SETTLEMENT OF THE TESTED SOILS UNDER APPLIED LOADS.

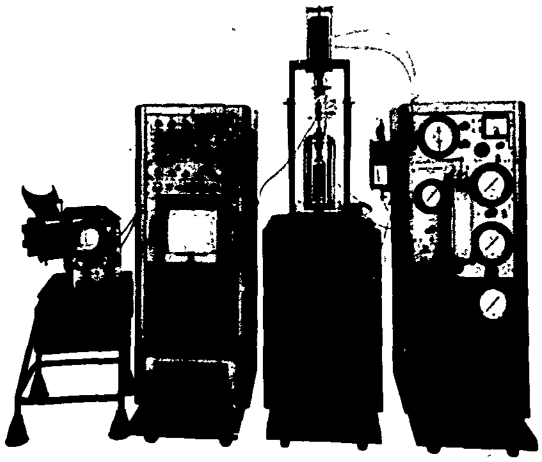
EACH SAMPLE IS TESTED WITHIN BRASS RINGS TWO AND ONE-HALF INCHES IN DIAMETER AND ONE INCH IN LENGTH. UNDISTURBED SAMPLES OF IN-PLACE SOILS ARE TESTED IN RINGS TAKEN FROM THE SAMPLING DEVICE IN WHICH THE SAMPLES WERE OBTAINED. LOOSE SAMPLES OF SOILS TO BE USED IN CONSTRUCTING EARTH FILLS ARE COMPACTED IN RINGS TO PREDETERMINED CONDITIONS AND TESTED.



DEAD LOAD-PNEUMATIC CONSOLIDOMETER

IN TESTING, THE SAMPLE IS RIGIDLY CONFINED LATERALLY BY THE BRASS RING. AXIAL LOADS ARE TRANSMITTED TO THE ENDS OF THE SAMPLE BY POROUS DISKS. THE DISKS ALLOW DRAINAGE OF THE LOADED SAMPLE. THE AXIAL COMPRESSION OR EXPANSION OF THE SAMPLE IS MEASURED BY A MICROMETER DIAL INDICATOR AT APPROPRIATE TIME INTERVALS AFTER EACH LOAD INCREMENT IS APPLIED. EACH LOAD IS ORDINARILY TWICE THE PRECEDING LOAD. THE INCREMENTS ARE SELECTED TO OBTAIN CONSOLIDATION DATA REPRESENTING THE FIELD LOADING CONDITIONS FOR WHICH THE TEST IS BEING PERFORMED. EACH LOAD INCREMENT IS ALLOWED TO ACT OVER AN INTERVAL OF TIME DEPENDENT ON THE TYPE AND EXTENT OF THE SOIL IN THE FIELD.

Figure C-6 CONSOLIDATION TESTS



PULSATING AXIAL LOAD TESTS ARE PERFORMED TO EVALUATE THE DYNAMIC PROPERTIES AND THE LIQUEFACTION POTENTIAL OF THE SOILS UNDER SIMULATED ANTICIPATED FIELD LOADING CONDITIONS.

PULSATING LOAD TESTS ARE STRESS CONTROLLED AND ARE PERFORMED ON UNDISTURBED OR RECONSTITUTED SAMPLES OF SOIL APPROXIMATELY 6 INCHES IN LENGTH AND 2½ INCHES IN DIAMETER. THE SAMPLES ARE ENCASED IN A RUBBER MEMBRANE, PLACED IN A TEST CHAMBER AND SUBJECTED TO CONFINING PRESSURE THROUGHOUT THE DURATION OF THE TEST. THE TESTS MAY BE RUN ON SOILS AT FIELD MOISTURE CONTENT OR ON ARTIFICIALLY SATURATED SAMPLES. THE TRIAXIAL EQUIPMENT ACTING THROUGH A BELLOFRAM SYSTEM APPLIES A PULSATING AXIAL LOAD. THE CYCLING SPEED OF THE LOAD CAN BE VARIED BETWEEN ½ TO 5 CYCLES PER SECOND TO SIMULATE THE FIELD LOADING FREQUENCY.

DYNAMIC PROPERTIES DETERMINATION

TO EVALUATE THE DYNAMIC PARAMETERS, THE SOIL SAMPLE IS LOADED IN CYCLIC COMPRESSION. THE LOAD AND DEFLECTION ARE RECORDED ON TWO CHANNELS OF A RECORDING OSCILLOGRAPH. BY TAPPING THE OUTPUT OF THE LOAD AND DEFLECTION TRANSDUCERS AND APPLYING THESE TO VERTICAL AND HORIZONTAL PLATES, RESPECTIVELY, OF A CATHODE RAY OSCILLOSCOPE, A HYSTERESIS LOOP IS PRODUCED. THIS LOOP IS PHOTOGRAPHED, AND THE PHOTOGRAPH IS USED TO EVALUATE THE DAMPING VALUE PRESENT. THE PROCEDURE IS REPEATED AT VARIOUS STRAIN AMPLITUDES TO EVALUATE THE DYNAMIC PROPERTIES IN THE RANGE OF INTEREST ON A PARTICULAR SAMPLE. THE LOAD AND DEFLECTION VALUES OBTAINED FROM THE OSCILLOGRAPH ARE USED TO EVALUATE THE DYNAMIC MODULI OF ELASTICITY.

LIQUEFACTION POTENTIAL

TO EVALUATE THE LIQUEFACTION POTENTIAL, THE SOIL SAMPLE IS SUBJECTED TO AXIAL CYCLIC LOADING. THE MAGNITUDE, FREQUENCY, DURATION AND SEQUENCE OF LOADING IS DETERMINED ON THE BASIS OF PAST EARTHQUAKE RECORDS. THE LOAD, DEFLECTION, AND PORE PRESSURE ARE RECORDED ON THREE CHANNELS OF A RECORDING OSCILLOGRAPH. THESE RECORDS ARE USED TO EVALUATE THE LIQUEFACTION POTENTIAL FOR THAT PARTICULAR SOIL TYPE UNDER THE TEST CONDITIONS.

Figure C-7 DYNAMIC TRIAXIAL TEST

confining pressure is equal to the existing static overburden pressure. While confined by this all around pressure, an axial deviator stress (increased axial loading beyond the all around confining pressure) is applied cyclically. The strain and increased pore pressure resulting from this instantaneous loading is measured. If desired, the loading cycles can be continued and the response of the sample observed for many cycles.

For the conditions anticipated at the site, it is believed that the strain resulting from the initial load applications would be useful in evaluating the dynamic stress-strain modulus of the materials. It is this particular item which must be evaluated for use in calculation of the depth attenuation factor, α_z, in Equation 2-1 and in dynamic finite element analysis should it be required.

C-4 Engineering Analysis and Report Preparation

During the progress of the soils engineering and engineering geology investigation, frequent meetings should be held to discuss the field and laboratory test results as they become available and to make the necessary adjustments to the program.

The results of the field exploration and laboratory testing program will provide the basis for the engineering analyses. When the analyses are complete, a final report should be prepared which includes the following information:

1. Complete description of all soils and geologic work performed at the site.

2. Plot Plan, on which are shown the locations of all borings, field tests, and geophysical profiles.

3. Specific information for each boring, including the number of samples, depths of sampling, depth of ground water, and samples selected for laboratory testing.

4. A general description of the site conditions.

5. Engineering geologic map and a description of the engineering properties of the soil and rock strata.

6. Ground water conditions at the site.

7. A tabulation of all field and laboratory test data, including the results of geophysical exploration program and laboratory testing program.

8. Results of the engineering analyses and specific recommendations concerning:

   a. Relative density, type and extent of materials to be encountered at each site.

   b. Excavation problems and recommendations for slope angles during construction.

   c. Extent of overexcavation required.

   d. Stability of proposed excavation and of fill and backfill slopes.

   e. Bearing capacity and settlement characteristics of the soils subjected to static loads.

   f. Suitability of the materials available for use as fill, backfill and subgrade support.

   g. Compaction characteristics of the individual soil types relative to their use as fill or backfill, including the properties of the fill soils following compaction.

   h. Need and method of ground water control, as well as the presence of soil conditions and water levels which necessitate dewatering with well points or other dewatering systems.

i. Substances in the ground water and soils deleterious to concrete and steel.

j. Magnitude and rate of settlement under backfill loads.

k. Design values for active and passive soil pressures at varying depths, including the effect of the water table.

l. Recommended values for static and dynamic modulus of elasticity and Poisson's ratio for use in the finite element analysis.

m. Compressional and shear wave velocity of all soils and rock present in the subsurface profiles.

The results of all tests and recommendations should be organized in a form permitting interpretation by the design engineer. The method of presentation of test data and results shall be by table, graphs or charts, depending on the desire of the design engineer.

C-5 Services During Construction

The soils engineer and engineering geologist should be retained to inspect construction operations. This inspection should be frequent enough to permit adjustments in the design if unusual or unanticipated subsurface conditions are encountered.

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13. ABSTRACT

This report presents a design procedure and cost optimization curves for cylindrical steel tanks with spherical end caps that are buried with their axis parallel to the ground surface. The fuel storage tanks are designed to resist high overpressures associated with nuclear air blast. Internal pressurization ~~has been~~ utilized to minimize material and fabrication costs and increase the buckling capacity of the tanks.

The design procedure is believed to be valid for overpressures range from 10 psi to 3000psi. The overall reasonableness of the design procedure has been confirmed by experimental tests on buried steel cylinders and pressurized tanks in granular materials. ( )

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