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EXPLOSIVE EXCAVATION TECHNOLOGY

Principal Author and Editor
Major Stanley M. Johnson

Contributing Authors

Captain Robert F. Bourque
Walter C. Day
Major Richard H. Gates
Captain Dennis R. Gilson
Richard L. Fraser
Erwin H. Kleist
Jerome E. Lattery
Edward J. Leahy
Bruce B. Redpath
Adam L. Remboldt

U. S. Army Engineer Nuclear Cratering Group
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Preface

This report was prepared by the U.S. Army Engineer Nuclear Cratering Group (NCG) to serve as a textbook for engineers in planning for the use of large explosive charges in a construction role, primarily for excavations and other civil engineering purposes. The experience of the Group in modeling nuclear cratering tests with chemical (high) explosives over the past nine years, and the more recent experiments which included objectives related to the applicability and the economy of chemical explosive excavation, are the primary foundation for the writing. The knowledge developed from these experiments and related research activities has reached a level where certain excavations achievable with large chemical explosive charges can be successfully and confidently designed. This report is written with the objective of documenting this technology and making this knowledge available for practical use. The task is in accordance with the mission of NCG to provide technical advice and assistance in the use of large explosive charges to field elements of the Corps of Engineers and to other government agencies and construction organizations. Admittedly, optimum procedures have not yet been established by field testing for the full range of possible excavation situations. The active program of research conducted by NCG under the Civil Works funded program, "Nuclear Explosives Studies for Civil Construction," is being concluded in June 1971; however, research and testing related to the explosive excavation concept will continue under a field office of the U.S. Army Engineer Waterways Experiment Station. This office will be created out of NCG and designated the U.S. Army Engineer Explosive Excavation Research Office (EERO). Improvements brought about by future research work will be documented by appropriate additions or revisions to this report.

Abstract

This report is the first comprehensive textbook on a relatively new method of construction originating from research into the large-scale use of explosives for construction purposes. The central idea is that explosives can be made to do more work for the civil engineer than just break up rock; various types of excavations and explosion-generated effects can be designed and produced safely, quickly, and in many cases more cheaply than by the use of other techniques. The overall concept, design approach and procedures, and the operational consequences of using currently available techniques are fully described. Emphasis is on the adaptability of the method and its present and future potential as a cost competitive tool in various construction roles.

The report deals with the mechanism of crater formation and the characteristics of explosion-produced craters. It covers the types of projects in which such craters have useful application, how to choose the proper explosive, how to design the charge emplacement and firing system, and how to evaluate the potential hazardous effects from detonation. The field operations associated with using the method are described and the postshot engineering considerations are discussed. An example is given to illustrate the procedure to be followed to analyze a typical excavation project.

Further research needed to increase the adaptability and competitiveness of explosive excavation is presented. Considerable ancillary information for engineers interested in the evolution of explosive excavation, case histories of projects, details relating to explosives and design procedures, and the procedures to contract for explosive excavation services is appended to the report.

Acknowledgments

This report was prepared in response to a challenge to the Nuclear Cratering Group (NCG) by BG Richard H. Groves, Deputy Director of Civil Works, Office of the Chief of Engineers, to write a comprehensive text on the use of chemical high explosives for excavation. This challenge, the stimulation that it engendered, and the continued interest of General Groves are keenly appreciated.

The guidance and encouragement of LTC Bernard C. Hughes, COL William E. Vandenberg and LTC Robert L. LaFrenz, Directors of NCG during the preparation of this report, are greatly appreciated.

This report represents the combined effort of many persons in NCG. The work of the following authors is acknowledged and sincerely appreciated: Captain Robert F. Bourque for Chapter 4 and Appendix C; Mr. Walter C. Day for Chapter 11 and Appendix A; Major Richard H. Gates for parts of Chapter 2, and Chapters 3 and 8; Mr. Richard L. Fraser for Appendix F; Captain Dennis R. Gilson for Appendixes D and E; Mr. Erwin H. Kleist for Chapters 9 and 10 and for significant contributions to Appendix E; Mr. Jerome E. Lattery for parts of Chapter 2; Mr. Edward J. Leahy for Chapter 6; Mr. Bruce B. Redpath for Chapter 5; and Mr. Adam L. Remboldt for Chapter 7. Other persons who assisted in the preparation of portions of the text include Mr. William R. Bechtell and Major Charles E. Gardner.

Finally, special thanks are due to Mrs. Helen M. Sarles for the typing of many drafts during the preparation of the report.

STANLEY M. JOHNSON
Major CE
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Contents

Figures

Figures (continued)

Figures (continued)

Tables

Tables (continued)

NCG TECHNICAL REPORT NO. 21

EXPLOSIVE EXCAVATION TECHNOLOGY

Chapter 1

Introduction

1.1 OVERALL PURPOSE AND SCOPE

This report is written to provide the civil engineer with information necessary to plan, to design, and to execute explosive excavation projects using chemical high explosives. The use of nuclear explosives for excavation projects has been previously reported by the U. S. Army Engineer Nuclear Cratering Group (NCG) in 1968.[1]

1.2 WHAT IS EXPLOSIVE EXCAVATION?

The term "explosive excavation" is a general term which in most instances implies "cratering" or the use of large explosive charges to produce an excavation by fracturing and ejecting large volumes of earth or rock in a construction application. The terms "quarrying" and "mounding" as used herein describe variations of explosive excavation in which large concentrated charges are used to fracture and to loosen rock material with little or no ejection of material.

Although a precise charge weight limit is not prescribed, it is generally conceived that explosive charge sizes will range from about one ton to several hundreds of tons. Based on the types of projects studied to date, it is expected that by far the most common sizes to be employed will be in the range from 1 to 50 tons. A single detonation most likely will involve more than a single charge.

In general, explosive excavation involves charges buried at depths ranging from the surface down to a point at which, upon detonation, there is little visible surface evidence of the detonation. Like the charge weight criteria, depth of burial limits are not precisely defined. As a rule of thumb, it is convenient to think of explosive excavation with chemical explosions in terms of tens of tons of explosives emplaced tens of feet in depth. The depth of primary interest for this report is termed the "optimum depth of burial"; i.e., the depth, measured from ground surface to the charge, at which the detonation will excavate the largest net volume of material. Design techniques which use other depths of burial are under development.

Explosive excavation is an alternate method to conventional blasting and hauling for moving soil and rock on construction jobs. Similarly, explosive quarrying and mounding are alternatives to the conventional quarrying and rock blasting operations conducted as a necessary part of many construction projects. The use of large concentrated charges offers substantial potential benefits when excavation and, in certain cases, when blasting or rock removal comprise major project activities. The principal potential advantages are speed of construction and economy.

Explosive excavation is a construction tool of considerable versatility. Even at the present state of the technology, it is characterized by emplacement and firing techniques which can be designed to obtain excavation configurations suitable for a wide range of projects. Further research is extending these design techniques to cover wider applications. Some projects which may not be feasible by other means may be accomplished through the use or adaptation of explosive excavation techniques.

Explosive excavation does not stop with detonations that produce huge craters or mounds of broken rock but includes provisions for postshot operations as necessary to deliver a useful engineering excavation. Due consideration is given to facilitating follow-on construction operations. The explosive excavation method embraces all of the engineering and the operations to fit the explosively produced crater, quarry, or zone of broken rock into the overall construction project.

1.3 BASIS FOR REPORT

Since 1962 the U.S. Army Engineer Nuclear Cratering Group has executed extensive laboratory and field cratering tests using chemical explosives (Appendix A). Initially, these were primarily modeling experiments for nuclear tests conducted to fulfill a part of the Corps' obligations in a joint Corps of Engineers—Atomic Energy Commission program to investigate the use of nuclear explosives for large-scale excavations. Since 1969 the objectives of this research have been broadened to include investigations into the practical aspects of using chemical explosives in similar but smaller construction roles. A number of experiments conducted under this program have dramatically illustrated the excavation potential of large charges of chemical explosives (for examples, see Appendix B). The technical knowledge and experience accumulated in conducting this research provide an initial level of understanding of the cratering process sufficient to establish procedures whereby explosive cratering can be successfully used as a competitive excavation method under certain conditions. This report is primarily concerned with these state-of-the-art procedures. Additional research is needed to increase the applicability of the explosive excavation method, to test and to prove new techniques, and to extend the method's competitiveness to more materials and situations.

1.4 PRACTICAL APPLICATIONS

The full range and variety of applications appear to be limited only by the imagination. Studies, augmented by laboratory and field experiments, indicate the following to be the most attractive practical projects at this time: canals, waterway connections, harbors, channel deepening and widening, and highway and railway cuts. More limited studies indicate the following projects as potential applications for explosive excavation techniques: quarries, expedient blasted-into-place) dams, and overburden removal. It is with such projects in mind that this report has been written. Applications such as these usually require considerable excavation and are frequently found in areas sufficiently remote to allow large detonations.

1.5 PREVIEW OF CONTENTS

To the engineer there are practical questions concerning the use of explosive excavation which must be satisfactorily answered before it is accepted and put to wide use. These include the effectiveness, the versatility, the simplicity, the safety, and the economy of the method. This report anticipates these questions and is devoted to providing answers. The central question which forms the theme for the first edition is this: What is involved in explosive excavation and how can it be used on construction projects?

The first nine chapters are designed to present a complete picture of explosive excavation in a relatively brief textbook form following the general chronological order of project development. The information presented is basic to clear understanding of all aspects of explosive excavation. Where appropriate, recommended procedures for using this information to accomplish excavation projects are stated. Chapter 10 goes further to illustrate, by example, how the information in the text may be applied to analyze projects for the practicability of using explosive excavation techniques. Chapter 11 deals with current and future research in the area of explosive excavation. Six appendixes provide selected additional information which, although considered to be extremely important and relevant to explosive cratering, was believed to be primarily ancillary to the main text, too specialized in nature or too detailed to be integrated into the chapters.

REFERENCE

1. LTC B. C. Hughes, Nuclear Construction Engineering Technology, U. S. Army Engineer Nuclear Cratering Group, Livermore, Calif., Rept. NCG-TR-2, September 1968.

Chapter 2

Crater Formation and Properties of Craters

2.1 SCOPE

This chapter briefly explains the concept of explosive cratering, introduces nomenclature, and describes the nature of an explosively produced crater. The information presented here provides the necessary general background for the development in subsequent chapters.

Two sections are devoted to the mechanics of crater formation and the methods used to predict the dimensions of craters. The discussion of crater formation is nonmathematical and provides an overview of the physical events accompanying an underground explosion. It explains how the crater and the surrounding zones of disturbance are formed. Empirical scaling is the method for predicting crater dimensions used throughout the text. It is the simplest of the methods in current use and most immediately applicable to practical engineering situations. Other methods, requiring complex calculations based on the conservation laws of mass, momentum, and energy, are used at the present time primarily as research tools. The presentation of empirical scaling includes experimentally developed curves relating crater dimensions to the depth of burial of the charges. The curves indicate the expected performance of a 1-ton charge of TNT in each of three geologic media. These results may be extended to other charge weights by the scaling rules, and they may be extended to other explosives and media by other considerations, including subjective judgment. The results from single-charge detonations can be used to predict the results from the detonation of rows of charges by the application of additional empirical rules.

The final section describes the nature of the fragmented and the fractured material found in and around an explosively produced crater. Included here is a general discussion of those material properties which characterize the crater and which are of primary concern in looking to the engineering behavior of the crater and the functional purpose to be served by the excavation.

2.2 CRATER FORMATION

a. Crater Description and Nomenclature

A crater consists of three concentric zones known as the apparent crater, the true crater, and the rupture zone. These are illustrated by Fig. 1, a cross section of a typical crater in rock.

The apparent crater is the net excavated volume below the original ground surface. Its radius, depth, and volume are the first criteria for the engineering design of an explosive excavation. Its cross section has been found to be best approximated by a hyperbola whose asymptotes are parallel to the crater slopes near the original ground surface. Its depth is somewhat less than the charge depth of burial except for charges buried at shallow depths.

The raised rim, or lip, surrounding the crater consists of uplifted material (upthrust) overlain with fragmented material which has been ejected from the crater. The fragmented material, ejecta, covers the original ground surface out to a distance approximately equal to three times the crater radius from surface ground zero (SGZ).

The true crater is the excavation which would exist if none of the ejected material fell back into the crater. The cross section of the crater has been found to be best approximated by a parabola. During crater formation the true crater is partially filled with fallback material to form the apparent crater. Thus the size and shape of the true crater are not easily discerned.

Surrounding the true crater is a rupture zone. Within this zone material has been displaced slightly upward and outward and has more fractures than in the natural state. The actual interface between the true crater and the rupture zone is poorly defined. It is thought to be more a transition region than a definite boundary. The outer limit of the rupture zone is also poorly defined. The blast-induced fractures diminish in number with distance from the detonation point until there are so few as to be indistinguishable from naturally occurring fractures.

In quantitative discussions of crater characteristics it is necessary to have in mind the nomenclature which describes features of the crater and the notation used to represent measurable parameters. Figure 1 shows the preferred crater nomenclature. Figure 2 shows the notations used for craters produced by single

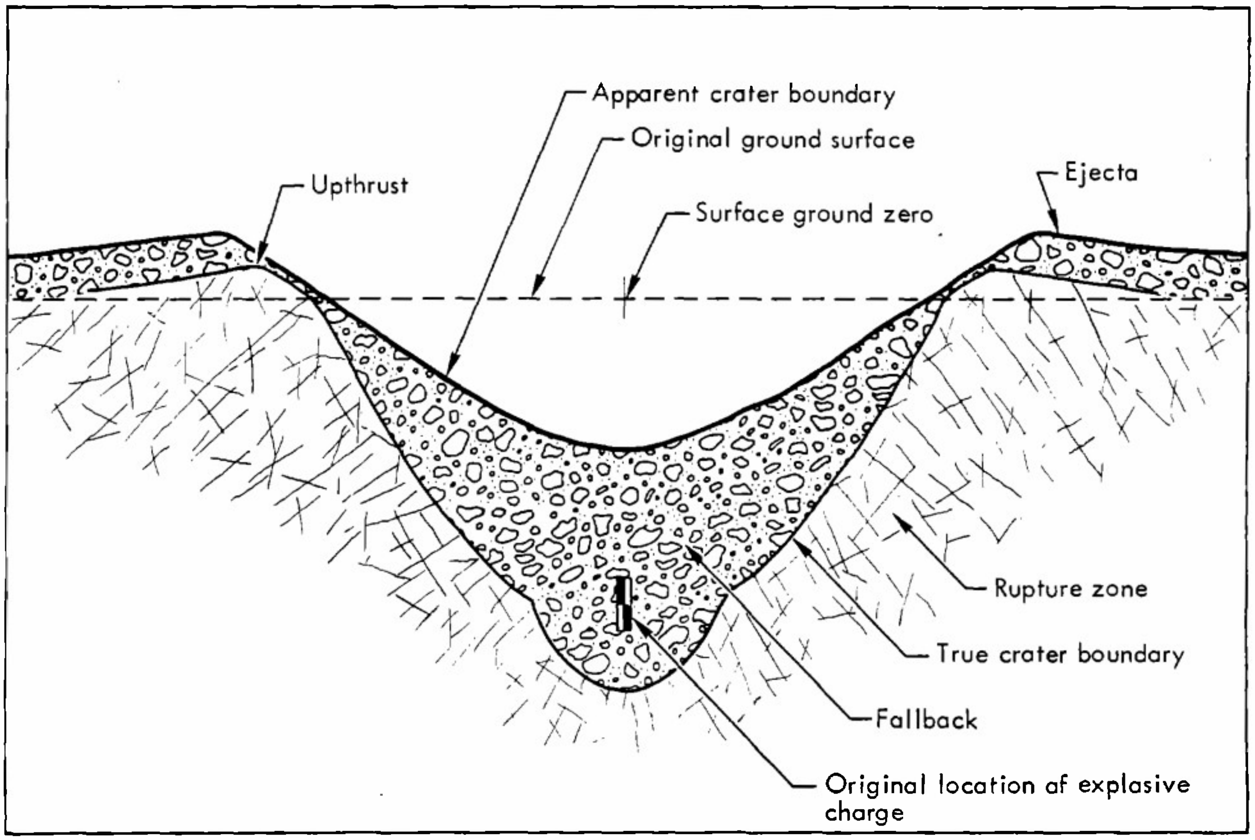
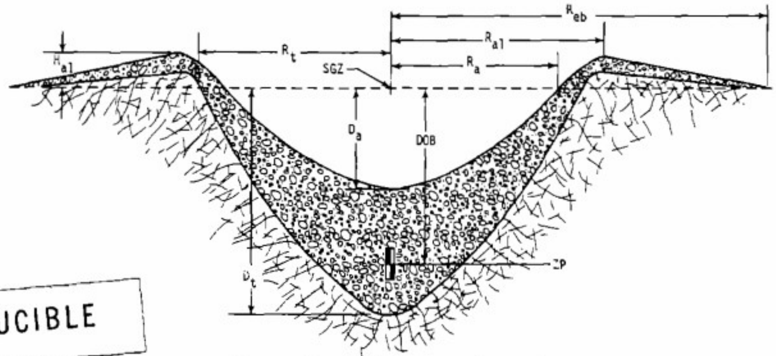
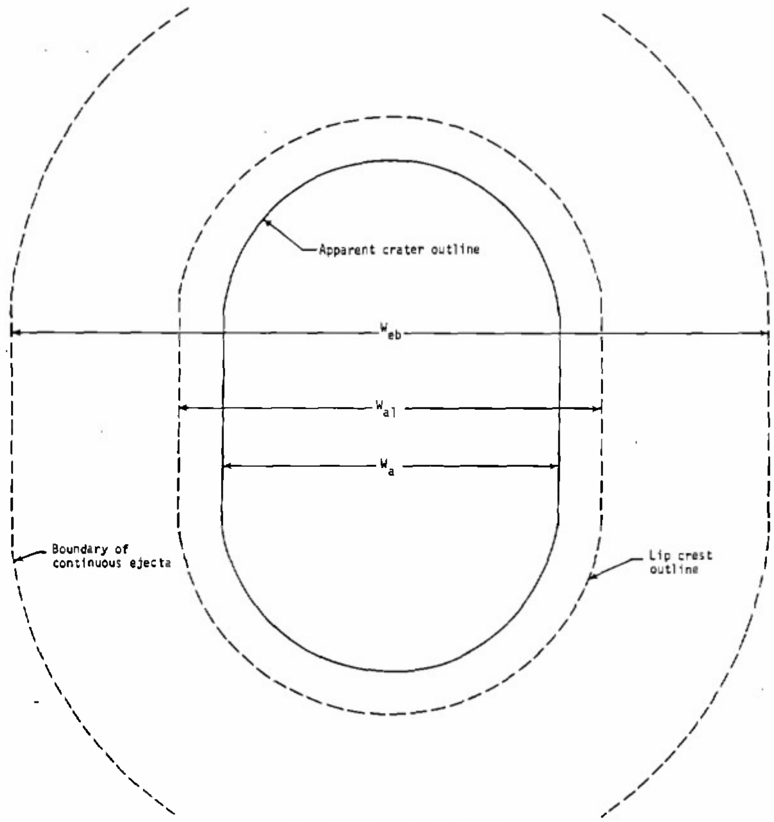


Fig. 1. Cross section of typical crater in rock, showing nomenclature.



Cross section of a single-or row-charge crater



Plan view of a row-charge crater

Notations which apply only to single charge craters

- R_a - Radius of apparent crater measured at original ground surface
- R_t - Radius of true crater measured at original ground surface
- R_{al} Radius to apparent lip crest
- R_{eb} - Radius to outer boundary of continuous ejecta
- D_a - Maximum depth of apparent crater below and normal to original ground surface
- D_t - Maximum depth of true crater below and normal to original ground surface

Notations which apply only to row-charge craters

- W_a - Width of apparent linear crater measured at original ground surface
- W_t - Width of true linear crater measured at original ground surface
- W_{al} - Width between apparent lip crests across the crater
- W_{eb} - Width between the outer boundaries of continuous ejecta across the crater
- D_{ar} - Depth of apparent row crater
- D_{tr} - Depth of true row crater

Notations and definitions which apply to both single and row-charge craters

- H_{al} - Height of apparent crater lip crest above and normal to original ground surface
- V_a - Volume of apparent crater below original ground surface
- V_{al} - Volume of apparent lip
- V_t - Volume of true crater below original ground surface
- DOB - Depth of Burial
- ZP - Zero Point-effective center of explosion
- SGZ - Surface Ground Zero (point on surface vertically above ZP)
- NSP - Nearest Surface Point (point on original ground surface nearest ZP - same as SGZ for horizontal surface)

Fig. 2. Crater notations.

charges and for craters produced by a single row of several charges. As will be seen later, these notations can be adapted for use with multiple row-charge craters.

In addition to the nomenclature and notations in Figs. 1 and 2, the following definitions will apply in discussions of the material properties of the media affected by a cratering detonation:

Rubble: The material comprising the fallback and ejecta.

Rupture zone: The zone of blast-induced fractures and displacements from the true crater boundary outward to the relatively undisturbed in situ material.

Bulking Factor (BF): The ratio of in situ or preshot bulk density to postshot bulk density. Bulking factor is used on conventional construction to determine cut, haul, and fill requirements.

Effective Porosity (n_e): The ratio of the volume of interconnected voids and fractures in a rock mass to the total volume of the same rock mass. The unconnected voids dispersed in the intact rock are not considered, consequently effective porosity of a mass is less than the total porosity (ratio of void volume to total volume) of the same mass.

In order to discuss crater formation in various geologic media, it has been found desirable to develop a system of media classification which is useful in the analysis of explosive excavation projects. Such a system, developed by the Nuclear Cratering Group, is shown in Table 1. The terminology and definitions shown in Table 1 will be used throughout this report when reference is made to media.

b. Basic Cratering Mechanism

An underground explosion fragments and ejects material by the combination of a strong compressional wave and sustained high pressures of the product gases. The key events in crater formation are illustrated in Fig. 3 for a charge buried at optimum depth in rock, and are described as follows:

(1) Within milliseconds after the detonation—see Fig. 3(a), the compressional wave has propagated a distance equal to the charge depth of burial. Behind the wavefront particle motion is radial and a state of compression exists.

(2) Surface spalling takes place—see Fig. 3(b). Because compressional stress cannot exist across the free surface, the particles move upward with a velocity equal to the sum of the wave particle velocity and the velocity imparted by the release of compressional stresses. Because the rock has little strength in tension, the upward-moving particles will pull free with considerable velocity. When a surface layer has spalled off, material beneath it experiences the conditions which had existed a moment before at the original ground surface. Spalling continues, caused by a rarefaction or tensile wave which propagates downward.

Spalling is not confined to the area immediately above the charge. At some critical distance from the charge a surface layer is lifted at low velocity only to fall back to its original position at a later time. The critical distance is the crater radius.

(3) Material above the charge fails—see Fig. 3(c)—due to the combined effects of the downward-moving rarefaction and

Table 1. Media classification for explosive excavation.[a]

A. Primary Classification
I. Media
A. Common excavation
1. Soil
2. Clay shale
B. Rock excavation (generally requires drilling and blasting to excavate)[1]
1. Weak rock (<4,000-psi unconfined compressive strength)
2. Intermediate strength rock (4,000 to 16,000-psi unconfined compressive strength)
3. High strength rock (greater than 16,000-psi unconfined compressive strength)
II. Lithology (for rock excavation) or soil classification (for common excavation) and geologic structures
III. Degree of saturation
A. Dry (<50% saturated)
B. Wet (50% ≤ % saturated ≤ 90%)
C. Saturated (>90% saturated)
IV. Joint spacing[2]
A. Very close (less than 2 in.)
B. Close (3 in. to 1 ft)
C. Moderately close (1 to 3 ft)
D. Wide (3 to 10 ft)
E. Very wide (greater than 10 ft)
V. Thickness of bedding[2]
A. Very thin (less than 2 in.)
B. Thin (2 in. to 1 ft)
C. Medium (1 to 3 ft)
D. Thick (3 to 10 ft)
E. Very thick (greater than 10 ft)
B. Secondary Classification
VI. Seismic velocity (compressional wave, V_p)
VII. Unconfined compressive strength values
VIII. Mass density
IX. Modulus of elasticity, E (taken tangent at 50% yield)
X. Abrasion

[a]Classification will be as refined as needed for the intended use.

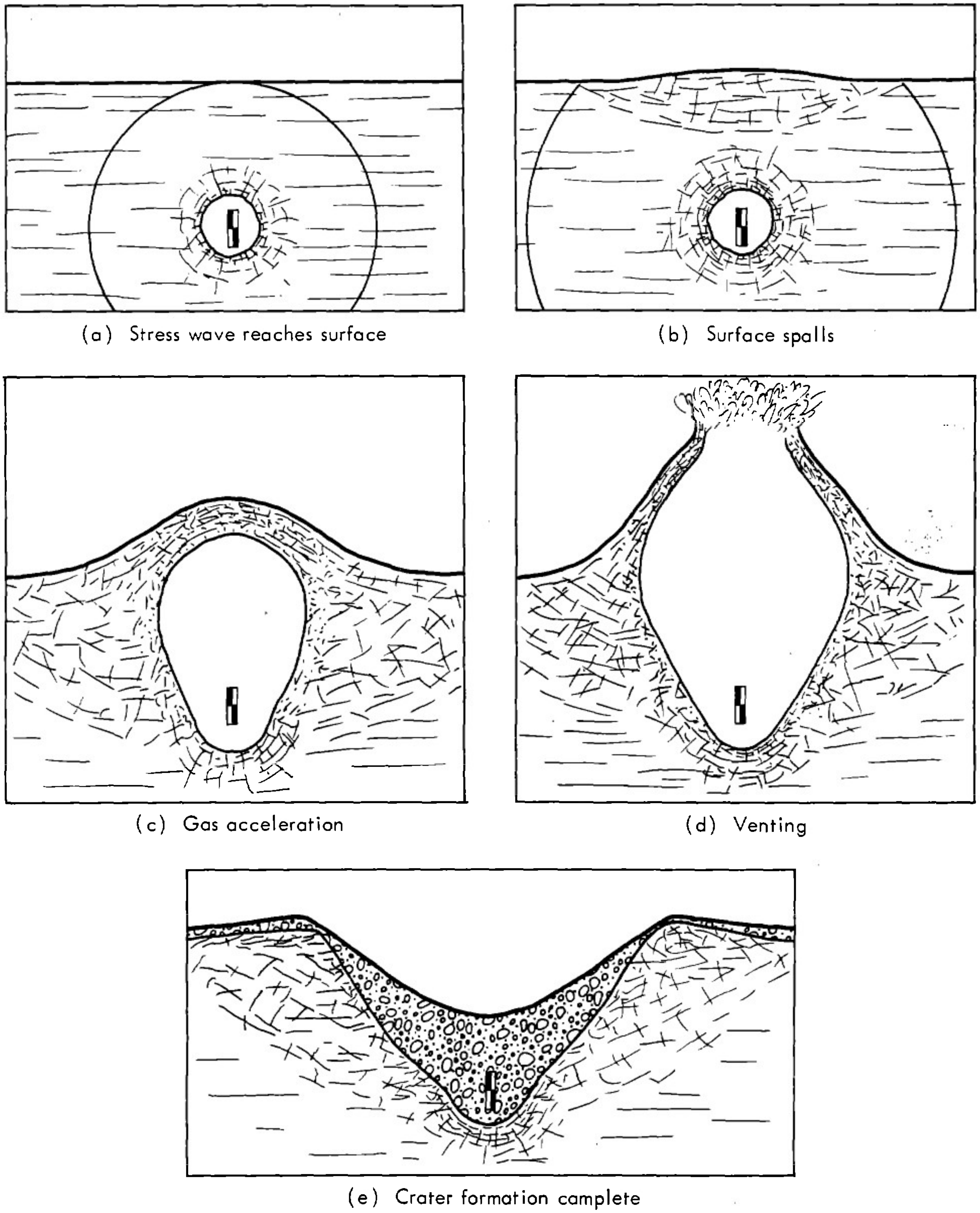


Fig. 3. Key events in crater formation.

the sustained gas pressure beneath it. A recompaction of the material propagates upward through the previously fractured and spalled surface layers to produce a second surface acceleration known as "gas acceleration." The cavity, filled with the high pressure gases produced by the explosive, expands preferentially toward the surface because the stresses on its upper boundary are relieved by the rarefaction returning from the surface.

(4) The mound of fragmented material has risen a distance approximately equal to the charge depth of burial—Fig. 3(d). The material is no longer cohesive enough to contain the explosion gases, and venting occurs. As the gas pressure is released the fragments assume free ballistic trajectories.

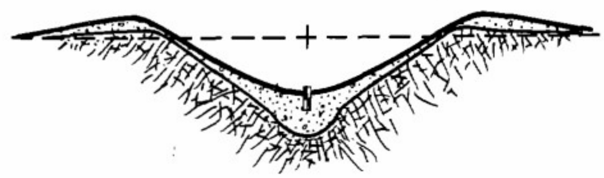
(5) Crater formation is complete—Fig. 3(e). The apparent crater depth has been determined by the amount of material which has fallen back into the initial void.

In materials other than rock, the mechanism of crater formation is substantially the same as for rock. The initial compressional wave and sustained gas pressure from the explosion are both essential to crater formation. But in weak porous material, such as dry soil, the initial wave is severely attenuated. Here the push from sustained gas pressure is the predominant mechanism for cratering. In another material—saturated clay shale—the initial wave is by far the predominant cratering mechanism. The shale is compressible but transmits the initial wave with little energy loss. This characteristic is combined with very low strength and a cratering explosion in this material spalls a deep surface layer at high velocity. The second surface acceleration due to sustained gas pressure is practically nonexistent.

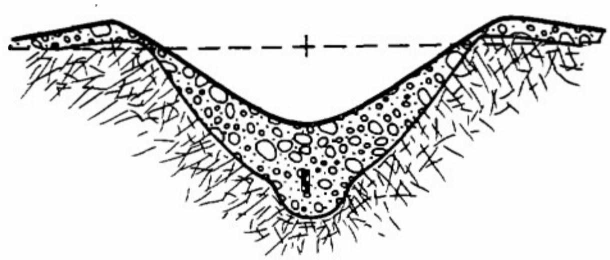
c. Effects from Varying Charge Depth of Burial

The foregoing discussion considered a charge buried at optimum depth for crater volume. The explosion produced a crater of large volume by fragmenting a large quantity of material and imparting sufficient velocity to most of that material to eject it from the true crater. An explosion at shallower depth will fragment relatively little material but will eject the material at high velocity. Conversely, a deep explosion will fragment a large quantity of material but will fail to eject it to form an apparent crater.

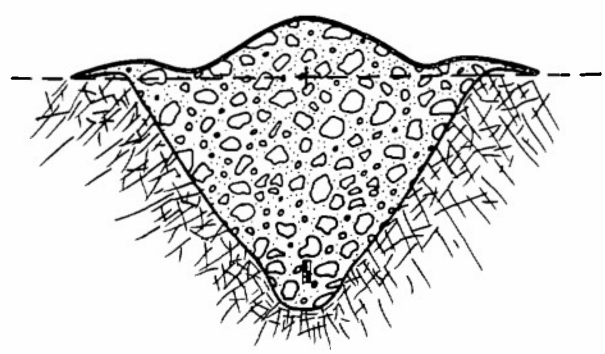
Characteristic features of craters formed by explosions at shallow, optimum, and deep depths of burial in rock are shown in Fig. 4.



(a) Shallow burial — 8 ft for 1 ton of TNT or equivalent



(b) Optimum burial — 18 ft for 1 ton of TNT or equivalent



(c) Deep burial — 28 ft for 1 ton of TNT or equivalent

Fig. 4. Crater profiles for various depths of burial.

(1) A crater formed by a shallow detonation—Fig. 4(a)—has less fallback material than a crater formed by a charge at optimum depth of burial. Its depth is equal to or greater than the charge depth of burial. It is shallow in relation to its radius, has a low lip, and has a relatively small rupture zone.

(2) The crater produced at optimum depth of burial—Fig. 4(b)—is partially filled with fallback. Usually the interior surface of the apparent crater is an uninterrupted slope of fallback material from the lip crest to the bottom. The lip height and extent of the rupture zone are intermediate between those for shallow and deep burials.

(3) The crater formed by a deeply buried explosion—Fig. 4(c)—is filled with fallback. If the fragmented material bulks so as to occupy a volume greater than its original volume, the true crater may be overfilled to form a mound. The rupture zone is relatively extensive.

2.3 PREDICTION OF CRATER GEOMETRY

Of the several means for predicting crater size and geometry, empirical scaling is the most practical for engineering. It offers accuracy comparable to that of elaborate computational methods while requiring only a fraction of the time and effort. In situations in which several charges are used to form a row-charge crater, empirical rules are the sole means for predicting the results.

Since the apparent crater forms the useful excavation, its size is of first importance. Other crater dimensions, such as lip height, are secondary but may also be important for evaluating the entire crater for certain engineering applications.

Although there may be few applications for single-charge craters, the capability to predict their size is essential. This capability forms the basis for predicting the size of row-charge craters.

a. Empirical Scaling of Crater Radius and Depth

The fundamental parameter in scaling is a quantity which represents the ability of an explosive charge to produce a crater. This quantity is not heat release, maximum pressure, or mass, but is some factor which takes into account all properties of the explosive and the medium. As indicated in the description of crater formation, the initial shock and the pressure after some gas expansion are both important properties when an explosive is detonated in rock. In weak materials the gas pressure may be more important than the initial shock. To avoid complications in the scaling of crater dimensions, it is presumed that the explosive is TNT, with its particular set of characteristics, and that the proper scaling parameter is charge weight. If another explosive is used, its effectiveness relative to TNT, discussed in Chapter 4, may be introduced as an adjustment to the computation.

In charge-weight scaling, crater dimensions are known for a reference charge weight and are multiplied by a scaling factor to predict the results for other weights. The scaling factor may be the cube root or the fourth root of the ratio of the charge weights; or it may be a fractional exponent lying between the cube and fourth roots. The reference charge in this discussion and subsequent development is one ton of TNT.

Cube root scaling may be derived from dimensional analysis by neglecting the effects of gravity and dissipative conditions, such as friction. With this form of scaling a crater dimension, radius for example, would scale as follows:

$$R_a = r_a \; (Y/Y_0)^{1/3}, \qquad (1)$$

where r_a is the crater radius for a charge having the weight Y_0 (which is one ton), and R_a is the crater radius for a charge of Y tons. The crater depth and the charge burial depth would scale similarly.

Cube root scaling gives reasonably accurate scaling of those crater dimensions and explosion effects which are little influenced by gravity. It will also scale all effects from small-scale explosions in low strength materials. However, it fails to scale accurately the crater dimensions, depth in particular, for explosions of more than a few tons. Cube root energy scaling will be used in this report for certain analyses.

Another form of scaling which may be derived by dimensional analysis and from principles of similitude is fourth root scaling. Although it is probably the fundamental form of scaling, a number of similarity conditions it requires cannot be met by explosions of less than several thousand tons.

Empirical scaling has been developed to provide a reliable scaling rule over the range of charge weights most often used in practical engineering situations. In this form of scaling the scale factor for crater dimensions is the ratio of the charge weights raised to an exponent which is intermediate between the cube root and the fourth root. A commonly accepted value for the empirical charge weight scaling exponent, 0.3, is used here. Another value, 1/3.4, may be found in the literature. The two exponents are so nearly equal that predictions of crater dimensions would differ by only a few percent if one were used in place of the other. Extensive cratering tests have led to the conclusion that the empirical scaling exponent, 0.3, can be confidently applied at depths of burial near optimum. At very shallow or deep depths of burial there would be less confidence in using this particular value.

To apply empirical scaling it is necessary to ascertain the crater dimensions for the reference charge weight in the medium being considered, and multiply them by the scale factor. Usually, the depth of burial, apparent crater radius, and apparent crater depth are the only variables considered in scaling. Other crater dimensions, such as lip height, are expressed as some multiple of crater radius, crater depth, or depth of burial.

Using a reference charge of one ton the scaling factor, P, is determined very simply:

$$P = Y^{0.3}, \qquad (2)$$

where Y is the weight in tons of the charge the crater dimensions of which are to be computed.

The charge depth of burial (DOB), the apparent crater radius (R_a), and the apparent crater depth (D_a) would then be computed from the relationships:

$$DOB = P \; (dob), \qquad (3)$$

$$R_a = Pr_a, \qquad (4)$$

$$D_a = Pd_a, \qquad (5)$$

where dob, r_a, and d_a are the depth of burial and the crater dimensions for a 1-ton charge of TNT.

Often in practice the charge weight required to produce a crater of specified dimension is the quantity to computed. In this case the specified carter dimension is divided by the corresponding crater dimension for a 1-ton charge to obtain the scale factor P. Then the unkown charge weight, Y, may be computed by the simple relation:

$$Y = P^{3.33}. \qquad (6)$$

In this form the relationship between charge weight and scale factor clearly indicates that great quantities of explosive are necessary to produce large craters. In general, the linear dimensions of a single-charge crater are doubled when the charge weight is increased by a factor of ten. Crater volume, being proportional to the cube of linear dimensions,

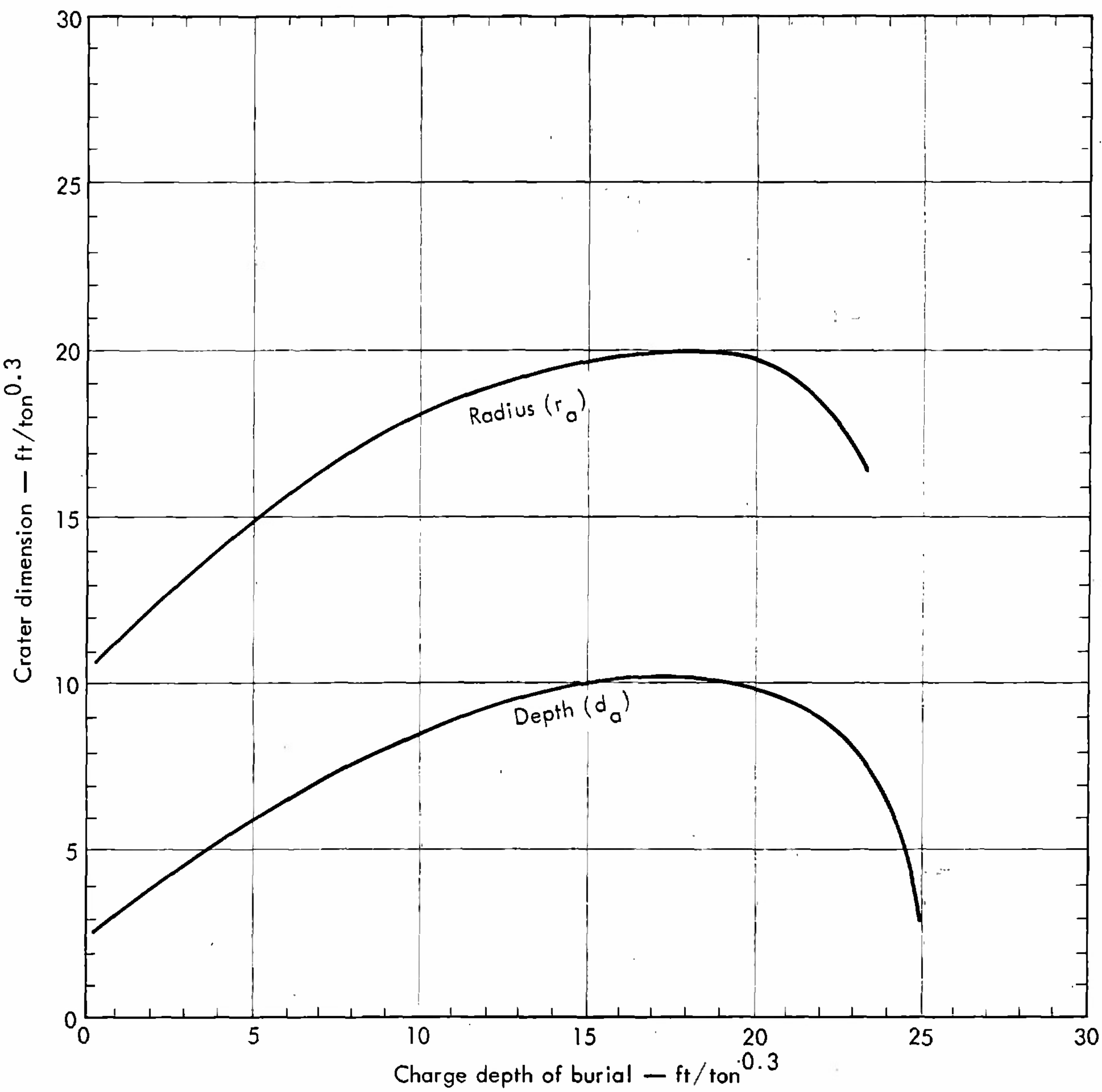


Fig. 5. Crater dimensions scaled to 1-ton charges of TNT or equivalent buried in dry rock.

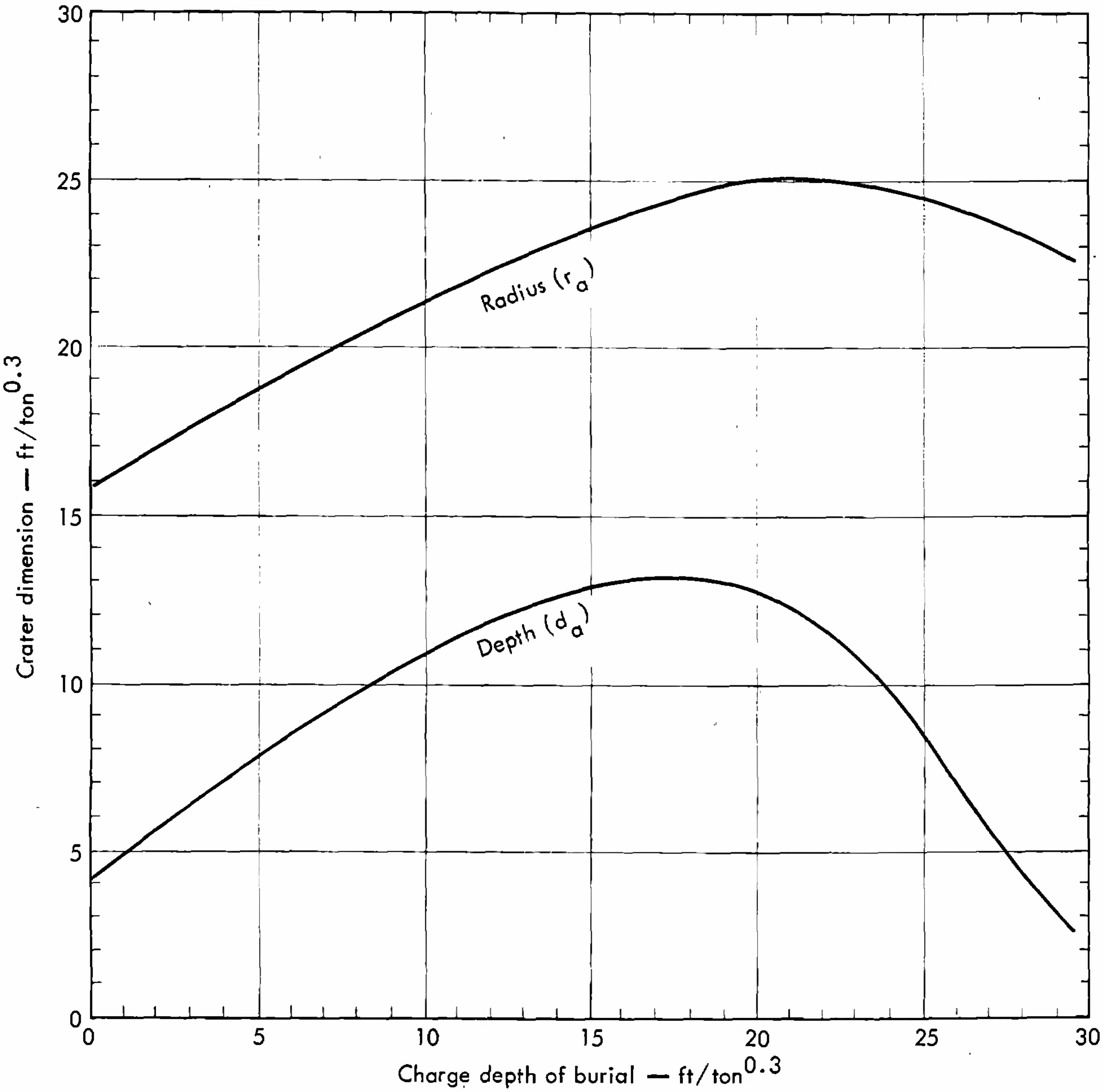


Fig. 6. Crater dimensions scaled to 1-ton charges of TNT or equivalent buried in dry soil.

increases by a factor of eight when the charge weight is increased tenfold.

The crater dimensions scaled to 1-ton TNT charges in dry rock, dry soil, and saturated clay shale are shown in Figs. 5, 6, and 7, respectively. The curves for crater radius and depth are based on data from experiments involving charge weights from 0.25 to 500 tons, with charge weights from 0.5 to 20 tons being most common.

The cratering curves for dry rock are based primarily on data from experiments in basalt, a high strength rock, with some verification from experiments in rhyolite. The curves for dry soil apply to desert alluvium, loess, dry sand, and materials of similar physical properties. They also apply to certain low strength sandstones. The curves for saturated clay shale are also reasonably valid for saturated sand.

The crater curves for dry rock and saturated clay shale may be regarded as the lower and upper limits, respectively, for crater dimensions in materials not mentioned above.

For maximum efficiency, a cratering charge is ordinarily buried at a depth which will assure the greatest apparent crater volume. In the three materials considered here the optimum depths of burial and the resulting crater dimensions are listed in Table 2 for the range of typical charge sizes. The burial depths and dimensions for the 10- and 50-ton charges are simply those for the 1-ton charge multiplied by $10^{0.3}$ and $50^{0.3}$, respectively.

b. Supplemental Crater Parameters

Although the apparent crater radius and depth are the first criteria for explosive excavation design, parameters which

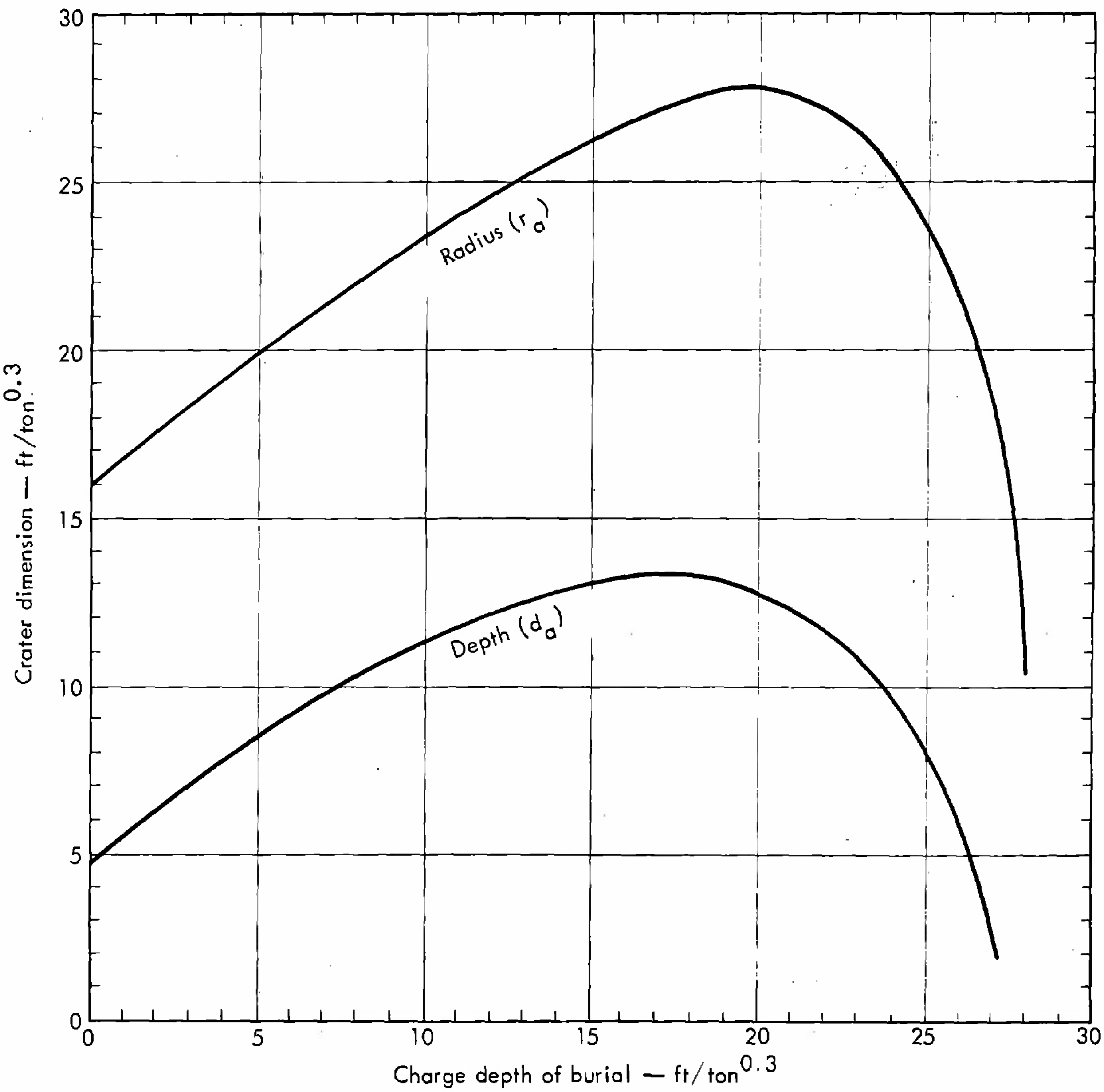


Fig. 7. Crater dimensions scaled to 1-ton charges of TNT or equivalent buried in saturated clay shale.

Table 2. Single-charge crater parameters for optimum depth of burial.

Material	Charge size (tons)								
	1	10	50	1	10	50	1	10	50
	Depth of burial (ft)			Crater radius (ft)			Crater depth (ft)		
Dry rock	18	36	58	20	40	65	10	20	32
Dry soil	20	40	65	25	50	81	12	24	39
Saturated clay shale	18	36	58	27	54	87	13	26	42

Table 3. Supplemental single-charge crater parameters.

Parameter	Dry rock	Dry soil	Saturated clay shale
Lip crest radius (R_{al})	1.2 R_a	1.2 R_a	1.4 R_a
Lip height (H_{al})	0.25 D_a	0.15 D_a	0.45 D_a
Radius of continuous ejecta (R_{eb})	3.0 R_a	2.2 R_a	3.5 R_a
Radius of rupture zone at surface	4.4 R_a	—	4.0 R_a
Radius of rupture zone at charge elevation	1.1 R_a	—	2.0 R_a
True crater radius (R_t)	1.0 R_a	1.0 R_a	1.1 R_a

describe the crater lip, the extent of the ejecta, and the extent of the rupture zone are useful. In Table 3 these parameters are given in terms of crater radius, or crater depth, and apply to craters produced by charges detonated at optimum depth. With the exception of lip crest radius, these parameters may vary over a considerable range. The lip height, for example, may vary by a factor of two around the perimeter of a typical crater. Comparable variation may be expected for the radius of continuous ejecta and the size of the rupture zone.

c. Row-Charge Cratering

In many applications it is necessary to use an array of several charges to produce a crater of suitable geometry. The most simple and most common array is a row of charges. The charges, usually five or more in number, are buried along the alinement of the desired crater with a horizontal spacing approximately equal to the crater radius for a single charge having the weight of one row-charge member. A properly designed row of charges will excavate a trench having a smooth and uniform cross section even though the charges are separated by some distance. Furthermore, the excavated volume per ton of explosive in the row is greater than for a single charge in the same material. The following discussion of row charges is background for the more detailed design procedures presented in Chapter 5.

Factors which influence the size and geometry of row-charge craters include those involved in single-charge cratering with the added factors of charge spacing and time delay, if any, between detonations of adjacent charges. Although the depth of burial of a single charge may be varied over a wide range to alter certain crater characteristics, the depth of burial for row-charge craters is restricted to a narrow range near the optimum for crater volume because of limited experimental data. There are as yet no reliable design procedures for row charges having depths of burial other than optimum.

In the discussion of single-charge craters it was noted that crater dimensions and depths of burial for any size charge may be scaled from the crater dimensions and depth of burial for a 1-ton charge. Row crater dimensions and charge depths of burial may also be scaled. The manner in which this can be done will now be described and the scaling parameter developed.

A characteristic of row craters is that their width (W_a) and depth (D_{ar}) are generally larger than the diameter ($2R_a$) and depth (D_a) of a single-charge crater excavated by a charge equal in weight to one of the charges in the row. This increase in dimensions is called enhancement, and the size of a row crater can be expressed in terms of single crater dimensions by means of an enhancement factor (the scaling factor). Because the enhancement of row crater dimensions increases as the charge spacing is decreased, the size of a row crater can be altered by changing the layout of the charges as well as their weights. The characteristic of enhancement is used in the design of row charges.

As previously mentioned, the excavated volume per ton of explosive in a row of charges is greater than for a single charge in the same material. This fact can be stated by the following equation[3]:

$$A_r e^2 S = KV_a, \tag{7}$$

where

A_r = cross-section area of optimum single-charge crater or unenhanced row-charge crater

e = enhancement of row crater dimension relative to single-crater dimension

S = spacing between charges in row

K = ratio of volume excavated by charge in a row to V_a

V_a = volume of the optimum single-charge crater

The terms on the left hand side of Eq. (7) simply represent the volume excavated by each charge in the row. The enhancement factor, e, is squared because the width and depth of the row crater are enhanced by equal amounts. Equation (7) can be rewritten

$$e = \left(\frac{KV_a}{A_r S}\right)^{1/2} \tag{8}$$

to show that the enhancement of row crater dimensions is inversely proportional to the square root of the charge spacing.

Further, Eq. (7) may be written as

$$e^2 = \frac{KV_a}{A_r R_a (S/R_a)}, \tag{9}$$

in which charge spacing is now expressed in terms of the optimum single-charge crater radius, this being the most convenient method of expressing row-charge spacing. Now, it has been found that the dimensionless quantity, $V_a/A_r R_a$, which appears on the right-hand side of Eq. (9) has a value of approximately 1.1 for an extremely wide range of crater geometries. This factor can be considered a constant, and so Eq. (9) can be rewritten again as

$$e^2 = \frac{1.1\ K}{S/R_a}. \tag{10}$$

The factor K, which can be thought of as the efficiency of a charge in a row compared to a single charge, has been determined from several field tests. The best current estimate for the value of K is approximately 1.3, which means that a row charge is about 30% more efficient than a single charge. Putting this value into Eq. (10) results in the following approximation:

$$e^2 \simeq \frac{1.4}{S/R_a}. \tag{11}$$

Equation (11) relates the enhancement of row-charge crater dimensions to the charge spacing in the row. It can be seen that a spacing of 1.4 R_a will result in a row crater with no enhancement. Equation (11) is relied upon in Chapter 5 to design row-charge craters.

An important adjunct of the concept of enhancement is the fact that the depth of burial of charges in a row must be the optimum single-charge depth increased by the amount of enhancement. Enhancement is discussed in greater detail in Chapter 5.

The nominal length of a row crater formed by N charges is S(N + 1). The length of the crater segment having uniform cross section is equal to the distance between the first and last charges, or S(N - 1). If the distance between the first and last charges is less than twice the crater width, the crater will not be linear but will resemble a single-charge crater. For charge spacings of about one crater radius, a minimum of five charges is needed to assure that the distance between the first and last charges is approximately twice the crater width.

In theory there is no reason why the charge weight and spacing cannot be simultaneously decreased to the point where adjacent charges are in physical contact. In practice, however, it appears that certain end effects occur when a row of many small charges is substituted for one having a few large charges. These end effects are recognized by a reduction in depth over the end charges and the consequent flattening of slopes qualitatively illustrated in Figs. 8(a) through 8(c).

It is not necessary that the charge weight and spacing between charges within the row charge be uniform. In fact, there are many applications in which the charge weight and spacing must be varied to produce a uniform cut through varying terrain. The design procedure for such applications is detailed in Chapter 5.

In the preceding discussion it was presumed that all charges in the row were detonated simultaneously. If time delays are used to reduce ground shock

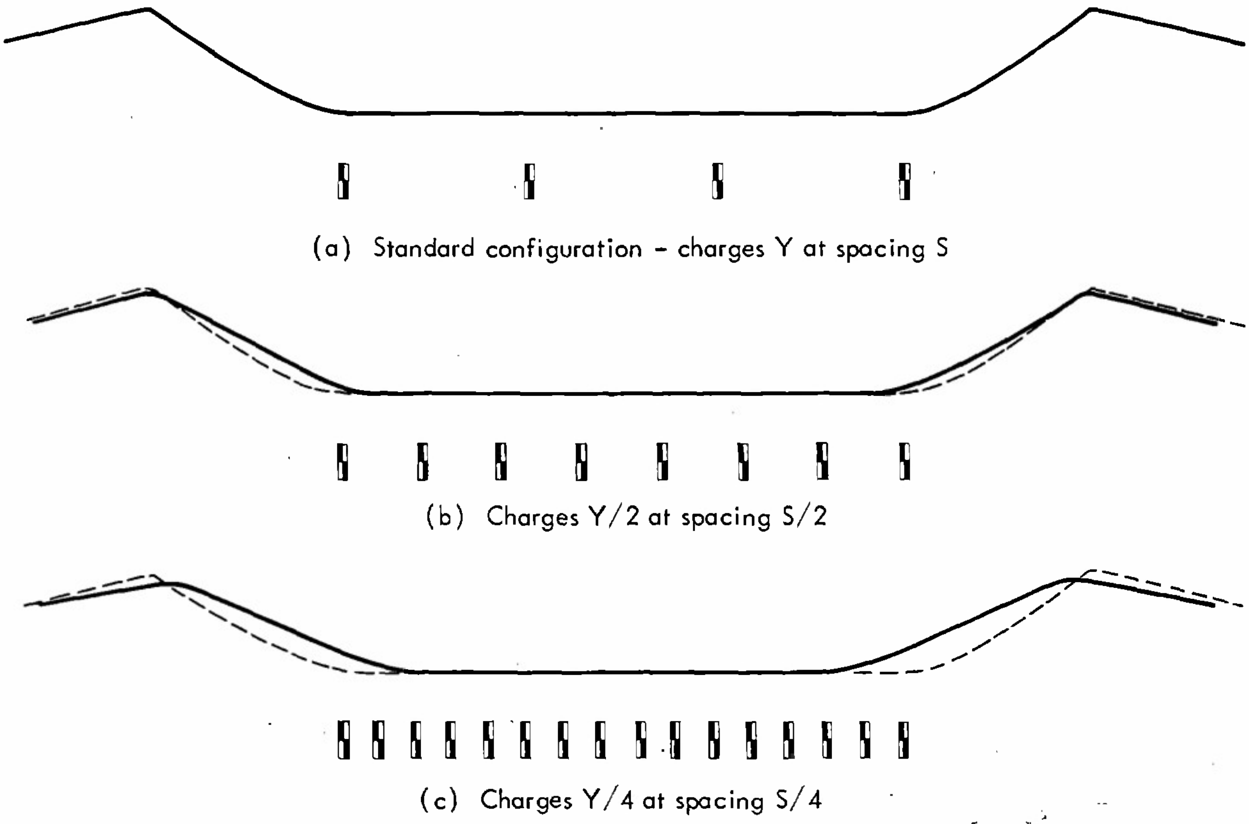


Fig. 8. Effects of charge size and spacing on row-crater end slopes (not to scale).

or airblast from the detonation, a reduction in crater volume results. The magnitude of this reduction will be influenced by the delay interval that is used. Experiments have indicated that time delay firings of charges in a row will tend to decrease the depth of the row crater. As the delay time is increased the decrease in crater depth approaches a limit as the delay is made indefinitely long. For long delays the crater depth is approximately half that for the same row of charges simultaneously detonated. The crater width is reduced only slightly. Criteria for use in designing delay intervals are given in Chapter 5.

d. Multiple Row-Charge Cratering

In applications in which crater width is more important than depth, it may be advantageous to use two parallel rows of charges. Such a charge configuration is detonated with a delay between the rows and can be designed to produce a crater typically one and one-half times as wide as the crater from either row of charges acting alone. The crater depth will be approximately the same as for a single row-charge crater. The design of two parallel rows is discussed in detail in Chapter 5. Investigations into the procedures for designing excavations using three rows of charges have just been initiated at the time of this writing.

e. Cratering in Media Overlain by Water

In some applications of explosive excavation, especially those related to navigation improvement, the rock to be excavated lies under water. The water overburden has a pronounced influence on crater characteristics. The inrush of displaced water after the detonation redistributes ejecta and may wash in material which would otherwise remain in place.

There are no reliable scaling relationships for predicting the size and geometry of underwater craters. Although the crater radius scales well when the water depth is a fixed fraction of the total charge depth of burial, the crater depth may not scale similarly.

To make use of underwater cratering on a practical basis it is necessary to perform experiments to determine crater geometry under the particular conditions at the site. As a first approximation, the underwater crater radius is assumed to be equal to the radius for a land crater in similar material and the depth is half that of the land crater. In determining charge depth of burial, the water layer may be regarded as a layer of the bottom material having a thickness equal to one-half the water depth.

2.4 POSTSHOT MATERIAL PROPERTIES

Equally as important as an understanding of the cratering mechanism and applicable scaling laws is an engineering knowledge of the nature of the excavation and of the materials comprising the various crater zones. The properties of these materials to a large extent determine the serviceability and practicality of the excavation. It is highly desirable to be able to predict postshot material properties and future engineering behavior before the detonation. The site data discussed in Chapter 3, which are collected during the project planning phase, are helpful in this regard. The media being dealt with range from soil through clay shale to competent rock. Depending upon the use of the information, the properties of intact specimens or the properties of the mass of material, or both, are measured. Seismic characteristics, porosity, angle of repose, various moduli, density, and the common index properties are examples. These material properties must be used to determine relevant engineering properties such as permeability, compressibility, and strength. The engineering behavior of the excavation, including seepage, settlement, and slope stability, can be predicted by proper evaluation of these engineering properties. It is the predicted engineering behavior of the resulting excavation that is used in conjunction with design techniques to forecast the capability of explosive excavation to meet project needs, and to estimate the scope of complementary construction activities.

a. Properties of Crater Rubble

The material comprising the fallback and ejecta has been fractured by the explosion, broken into particles of various sizes, lifted into the air or sloughed down, and redeposited in a somewhat predictable pattern. This is crater rubble. If the detonation is in rock, the rubble is more or less loose blocks of material, the faces of which are preexisting discontinuities or blast-induced fractures.

The size distribution of ejecta and fallback particles is a function of the natural material characteristics. The size distribution is also influenced by the

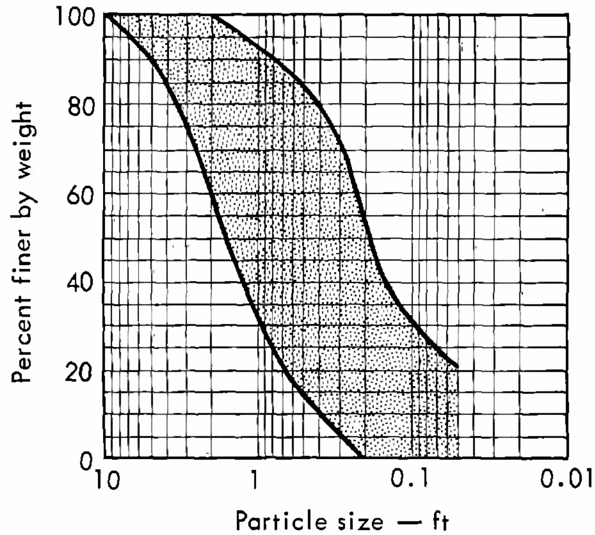


Fig. 9. General range for rubble gradation curves.

type of explosive and the charge depth of burial.[4] It should be noted that subsequent weathering may cause the particle size distribution of clay shales to be altered significantly. Figure 9 gives a general range for rubble gradation. The two curves shown are limits within which the particle size distribution curves, as determined by sieving for all available test craters, were found to lie. Material properties investigations made to date at rock crater sites indicate that gradation of the rubble is related to the preshot fracture pattern and to the type of material.[1] Figure 10 shows both a gradation curve developed by sieving and a curve developed from preshot in situ borehole photography data for the Pre-Schooner Delta crater.* The use of preshot borehole photography data as the basis for the prediction of final rubble size has met with moderate success.[5]

The bulking factor (BF) of rubble for several craters has been determined by measuring the bulk density of the material before and after a shot. The results of these investigations indicate that the bulking factor will be between 1.1 and 1.6 for craters from charges at optimum depths of burial.[6]

The porosities of rubble have been evaluated at a number of experimental craters. Total porosity is the sum of the initial porosity and Δn, the increase caused by the bulking. This increased porosity is related to the bulking factor (BF) by the expression:

$$\Delta n = 1 - \frac{1}{BF}. \qquad (12)$$

Table 4 gives the increased porosities for the range of bulking factors found to be typical of craters in rock. On the basis of the data from experiments, it appears that basalt fallback materials have total porosities of 30 to 62%, including initial porosities. These values are similar to those for structures such as rock-fill dams.[7]*

*See Table 17.

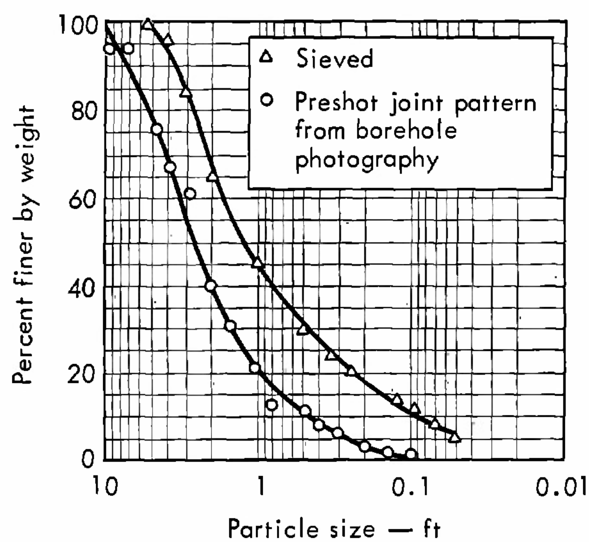


Fig. 10. Gradation curves for Pre-Schooner Delta crater rubble.

*Tables A1 and A2 in Appendix A list all major cratering experiments.

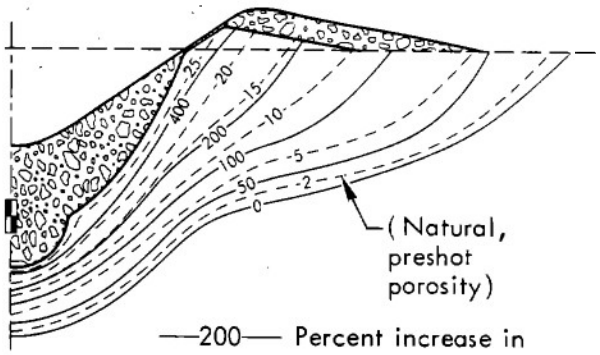


Fig. 11. Representative values of blast-induced fracturing and porosity in rupture zone (adapted from Ref. 6).

b. Properties of Rupture Zone

Deformations occur in the rupture zone in the form of blast-induced fractures, opening of existing fractures, and shearing action, accompanied by significant displacements. These deformations have been visually noted in the lip upthrust, which is that portion of the crater rupture zone above the original preshot ground surface. Exploratory drilling in the rupture zone below the preshot ground surface has provided information on the increase in the intensity of fracturing. In the portion of the rupture zone near the true crater boundary, fracturing increases in intensity by several hundred percent as compared to the preshot in situ fracturing (Fig. 11). The majority of blast-induced fractures are alined in a direction nearly parallel to natural fractures.

The effective porosity of the materials in the rupture zone is increased significantly as a result of the cratering detonation. This increase is due primarily to the opening of both natural and blast-induced fractures. Investigations of several craters in basalt, having a preshot effective porosity of approximately 2%, have indicated blast-induced porosities of as high as 25% near the true crater boundary.[6] Figure 11 schematically illustrates the decrease in blast-induced fracturing and effective porosity with distance from the true crater boundary.

2.5 SUMMARY

Empirical data have been used effectively to develop scaling laws which predict the size and shape of explosively produced craters to an acceptable degree of accuracy. This information is used in Chapter 5 as the basis for the design of explosive excavations. The critical properties of the crater materials have been discussed to provide a better understanding of their nature. Although a zone of rubble and a rupture zone are a characteristic consequence of an explosive excavation detonation, the properties of the material comprising these zones are measured by commonly used engineering methods. The collection of field data and the relationships between these material properties and the engineering behavior of the crater are discussed in Chapter 8.

Table 4. Increased porosities as determined from bulking factors.

Bulking factor	Increased porosity, Δn (%)
1.1	9
1.2	17
1.3	23
1.4	29
1.5	33
1.6	38

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Chapter 3

Applications

3.1 SCOPE

The use of explosive excavation techniques are potentially favorable for projects in remote areas where large quantities of rock must be excavated, speed of construction is essential, and weather and terrain prohibit or severely restrict the prolonged use of heavy equipment. These types of projects have stimulated and guided the development of explosive excavation technology. This chapter first discusses broad project considerations which are important during feasibility studies and field investigations leading to an explosive excavation design. Next, the types of projects which lend themselves to explosive excavation techniques are introduced and some of the relevant criteria for each type of project are discussed. Many of the points mentioned briefly in this chapter are developed in considerable detail in later chapters.

3.2 GENERAL CONSIDERATIONS

The project design establishes criteria which must be satisfied regardless of the method used for construction. These project criteria are principal constraints in the development of an explosive excavation design just as they are for conventional excavation. A measure of the suitability and the economy of explosive techniques for certain projects is the degree to which the explosive excavation can be made to match the necessary project dimensions, and usually these dimensions have been established by consideration of conventional methods only. Explosive excavation designs can be varied to achieve a variety of excavation configurations; however, excavations which best utilize the typical hyperbolic cross section of a crater are best-suited to explosive excavation at the present state of the technology. As an additional point deserving special emphasis, explosive excavation can be used alone, but it most likely will be used in conjunction with conventional means to meet project needs.

For preliminary estimates, sufficient data are needed to reasonably determine explosive charge requirements and positioning, to predict explosion effects, and to develop cost estimates. Assumptions may be necessary if data cannot be obtained from site reconnaissance and reference material. The need for additional site data, research, and calibration detonations to verify assumptions and to refine preliminary designs, safety, and cost estimates should be identified in the feasibility estimate.

The criteria pertaining to the portions of the project considered for accomplishment by explosive excavation should be determined as specifically as possible. Other project criteria may have an indirect bearing on the explosive excavation and should be developed in general terms.

Topography has a major influence on required explosive charge size and excavation design. The initial design most likely will be prepared from available topographic information but will require refinement when detailed topography is obtained at a later stage. Water depth information is important in underwater applications and in harbor excavation siting. Water depth will have a significant bearing on the quantities and the cost of excavation and breakwater construction. Currents, tides, and waves will influence field procedures, the cost of emplacement construction, and the quantities and cost of detonation construction.

The geologic composition and the structure of the medium in the area of a contemplated explosive excavation will have a pronounced influence on emplacement construction, cratering results, and overall design for the construction project. The media should be classified in as much detail as possible using Table 1 as a guide for the properties of interest. For the preliminary feasibility estimate, unless otherwise obvious, it may be assumed that structural discontinuities or faults are not present; however, it must be recognized that variations in the medium, stratification, and bedding could affect the cratering mechanism. The in situ joint and fracture pattern will affect the particle gradation of the crater rubble. Low strength rocks and deep soil deposits might develop long-term slope stability problems. Groundwater information is important as a part of the explosive selection process. If the explosive charges are to be placed in water, the more costly water-resistant slurries will be required. The seismic velocity of the medium is important for evaluation of ground shock effects.

Ground shock and airblast effects could influence the safety and economic feasibility of explosive excavation on a given project. Usually, potential damage to structures is the major concern. The distribution of dwellings and other structures should be determined by range from the contemplated explosive excavation site. At the preliminary feasibility estimate stage, the distribution could be determined from the population dispersion and the average number of people per dwelling or structure. Airblast damage predictions can be made for standard atmospheric conditions and revised later on the basis of temperature inversion and velocities of winds aloft. Other potential safety problems should be identified in the feasibility estimate.

The environmental and ecological effects of the proposed project must be analyzed. Appropriate State and Federal Agencies should be contacted early in the planning stages to discuss the effects of the project on natural shore processes, marine and wildlife habitats, forest and woodlands, and the hydrologic cycle. Measures for reducing adverse effects on the environment should be considered and incorporated in the design as appropriate. The environmental considerations arising from the detonation of large charges are discussed in Section 6.8 of Chapter 6.

The remainder of this chapter explores possible applications, both tested and conceptual, which are or appear to be feasible with chemical explosive excavation technology. In each section the primary considerations for developing a design for the application are briefly stated. These considerations and data concerning the site media collected according to Table 1 are used by the engineer to develop preliminary designs of the crater configuration best suited to the application. Specific projects which illustrate the application are included where possible to give the "feel" of the size of excavation which can be accomplished. The relationship between project dimensions and charge size will be developed in Chapter 5. Safety considerations and the detailed safety analysis, to which every design must be subjected, are discussed in Chapter 6.

3.3 CANALS*

Canal construction by use of explosive excavation techniques has been the subject of extensive studies and investigations, and the stimulus for many tests conducted by the Corps of Engineers and the Atomic Energy Commission since 1965. While these studies have concentrated on the feasibility of using nuclear explosives for the very large excavations required for a sea-level canal in the Central American Isthmus, they suggest that it is practical to consider the use of chemical explosives for canal construction. For smaller canals the use of chemical explosive charges may prove to be a desirable choice.

A canal constructed by the explosive excavation method would be formed by a linear crater designed to use one or more rows of charges. The charges might be detonated all at once, in an ordered sequence, or the length could be divided into short segments and each segment detonated independently of the others. The typical crater cross section could be used with little or no modification. The design would ordinarily specify a certain width of excavation at a specified depth. The overdepth in the center of the crater would reduce future maintenance costs. Simple projects of this type can be accomplished by current techniques.

*Limited testing of concept.

Important factors which will have an influence on the explosive excavation design include: the route of the canal, the engineering properties of the site medium, and such engineering requirements as length, width, and depth (navigation prism), and side slopes of the canal.

An excellent example of a canal excavated by chemical explosives is the Pre-Gondola series of row craters at Fort Peck, Montana.[1] These craters were produced as a part of an experimental cratering program carried out by the Nuclear Cratering Group and are discussed in detail in Appendix B. Figure 12 gives a good indication of the size of the completed canal, its total length measuring 1370 ft. The crater width at water level averages 150 ft, and the water depth at centerline averages 26 ft.

3.4 WATERCOURSES*

A watercourse is defined as a canal, channel, or ditch for the specific purpose of moving water from one geographic area to another. Examples of watercourses include irrigation and drainage ditches, floodways, and spillways.

The shape of explosively formed row craters is well-suited to some applications of this type. The craters used for a watercourse would be similar to those

*Limited testing of concept.

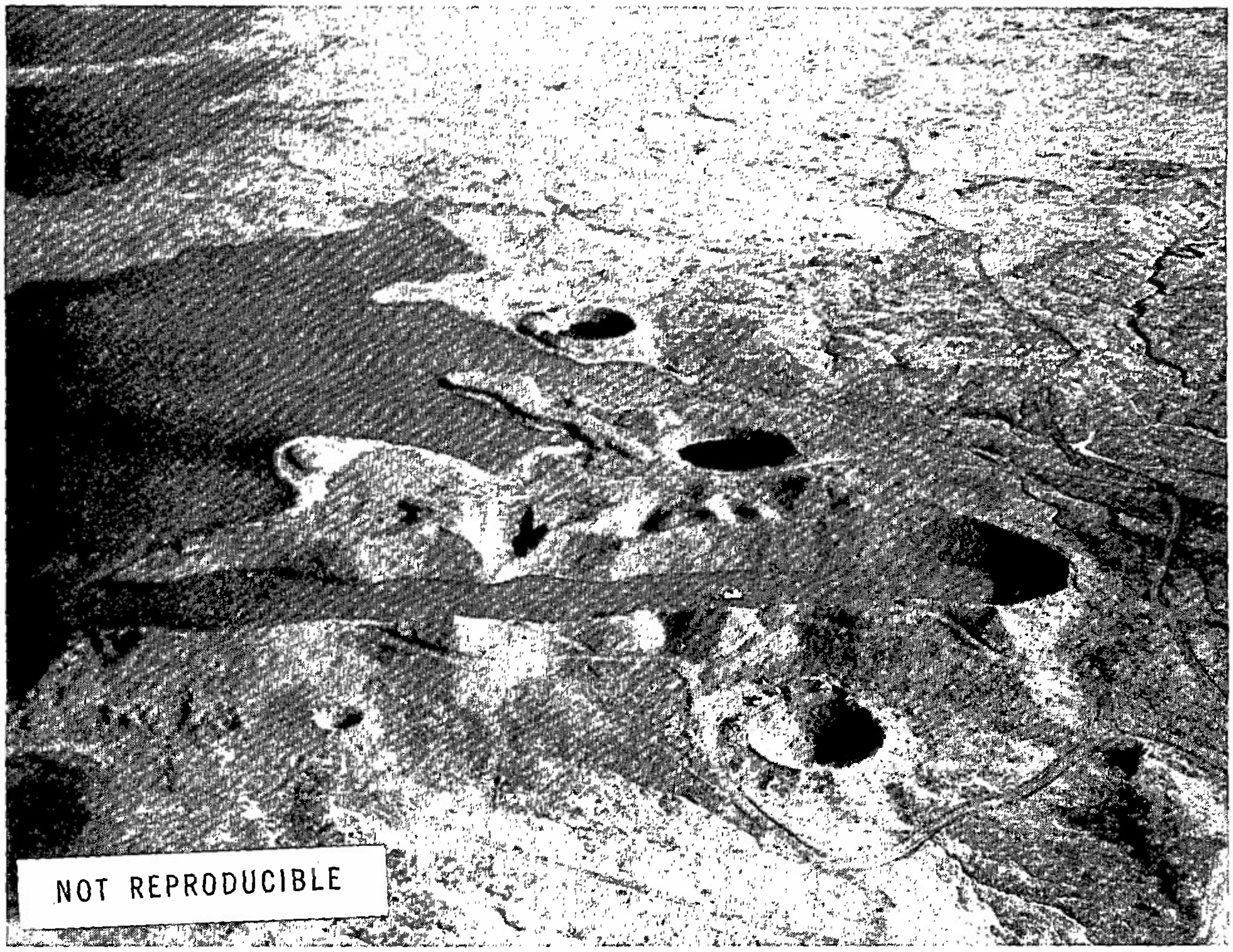

Fig. 12. The 1370-ft Pre-Gondola "canal" (boat is 42-ft tugboat).

used for a canal. The requirement for a navigation prism would ordinarily not be required.

The primary design consideration in these applications is the discharge capacity of the watercourse which is established by project criteria and is reflected in the cross section, depth, and gradient of the channel. Side slopes must be stable. Other factors important in the design include those discussed for a canal.

Spillways or connecting channels associated with dam and lake projects are typical types of watercourses which have been studied and found to be amenable to explosive excavation technology. Simple projects of this type can be accomplished using design criteria presented in this report.

3.5 HARBORS*

The construction of a harbor for small to medium size boats, on land adjacent to water or offshore in shallow water, is within the capability of chemical explosive excavation technology. A harbor complex will normally consist of a mooring basin, a turning basin, and an inlet channel. The major considerations in harbor excava-

*Limited testing of concept.

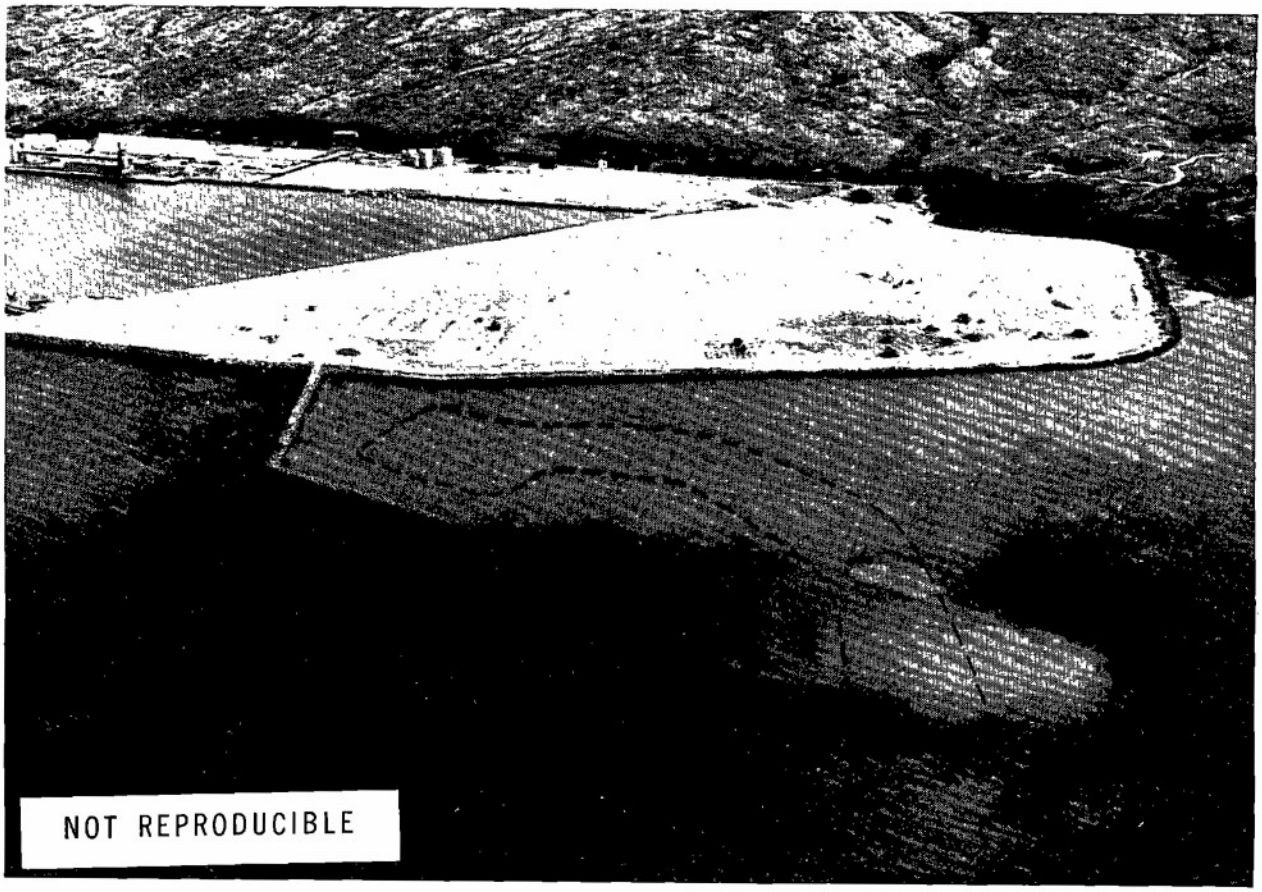

Fig. 13. Aerial view of Project Tugboat site, Kawaihae Bay, Hawaii, showing areas excavated with explosives.

tions are the size and depth of the basin and channel. The harbor must be large and deep enough to accommodate the boats expected to use the facility. In certain regions the mean tidal range will be an important factor in fixing the required depth.

The size of the harbor basin will dictate the use of single or multiple rows of charges for the excavation design. Although not yet verified by tests, it appears reasonable that the excavation can be designed such that the crater lips produced by large detonations in rock could serve as a rudimentary and perhaps adequate breakwater for some projects. Some harbor projects could be accomplished with current techniques; others might involve conceptual designs which would require further testing.

An example of a certain type of harbor capable of being produced by explosive excavation is illustrated by Project Tugboat, a small boat harbor excavated offshore in coral material at Kawaihae, Hawaii, by the Nuclear Cratering Group in April and May 1970. Figure 13 is an aerial view of the harbor site. The design requirement was for an entrance channel 12 ft deep, 120 ft wide connected to a berthing area 12 ft deep and 240 by 240 ft. (refer to Fig. 13). The coral, a low strength porous rock, was under 6 ft of water. The excavation design used a row of eight 10-ton charges for the channel and a square array of four 10-ton charges for the berthing basin. The results proved this design to be conservative. Project Tugboat is described in detail in Appendix B.

3.6 CHANNEL IMPROVEMENTS*

Explosive excavation shows great promise as a means of widening and deepening existing river and intracoastal waterway channels and of removing navigational hazards such as rock outcrops and shoals. The depth and width of the navigation prism are the principal criteria affecting an explosive excavation design. These criteria, in turn, are dependent on the vessels using the waterways and the traffic plan, and will be influenced by local site conditions to include meterologic, hydrologic, and geologic data—in the same manner as harbors. Single-row craters may be used in some instances, but it is expected that most projects of this type will be underwater rock removal projects requiring multiple rows of charges fired with delays between rows. Some additional testing is needed to prove the concept.

The application of explosive excavation to projects of these types is expected to result in considerable dollar savings over conventional means. Furthermore, difficult blasting and local site conditions may make explosive excavation the only feasible method of undertaking certain projects of this nature. For these reasons the use of multiple rows of charges for underwater rock removal is under active investigation at the time of this writing.

An example of a large chemical explosive detonation for obstacle removal is Ripple Rock.[2] This project was accomplished by the E. I. du Pont de Nemours and Company, under contract to the Federal Department of Public Works in Canada. Prior to the blast, Ripple Rock was a steeply peaked ridge situated in the middle of the 2500 ft wide Seymour Narrows about 110 mi north of Vancouver, British Columbia (Fig. 14). Seymour

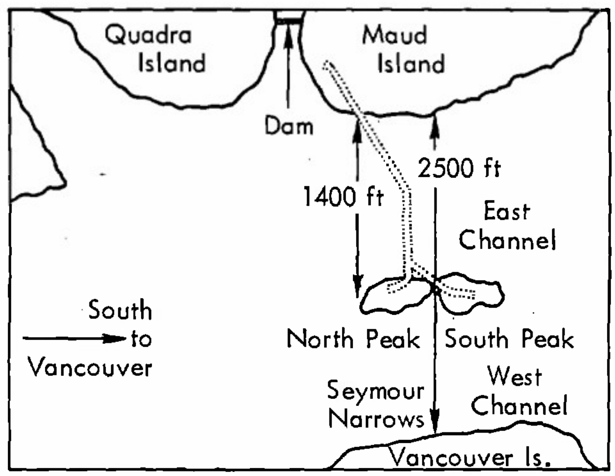

Fig. 14. Location of Ripple Rock in Seymour Narrows (from Ref. 2).

Narrows is part of the main shipping channel known as the Inside Passage to Alaska. Before the elimination of Ripple Rock, only ships capable of overcoming a 12- to 15-knot current and having good maneuverability could pass the rock, except at slack tide.

Early efforts to remove portions of the rock with small-scale blasting had been unsuccessful. The rock consisted of relatively sound volcanics, basalt, and andesite which contained no large void

*Very limited testing of concept.

spaces. The plan for demolition called for charging "coyote tunnels"* dug parallel to the long axis of the rock and arranged to follow the cross-sectional shape. The "coyote tunnels" were reached by an access tunnel 120 ft below the surface. Preparations for the blast required 30 mo. Approximately 1378 tons of "Nitramex" 2H, a high density blasting agent, were used. The blast successfully cleared the channel.

3.7 HIGHWAY AND RAILWAY CUTS†

The principal criteria for the development of an explosive excavation design for highway or railroad cuts are the required width of the cut at the finish elevation and the location of the groundwater table. The required-width-at-specified-depth criterion is somewhat analogous to the navigation prism requirement for canals. The water content of the subgrade is important in this case because it will influence the stability of the highway or railway. Proper drainage measures must be provided throughout construction. A thorough analysis of surface runoff patterns, percolation, seepage, and storage in the vicinity of the project site is important.

Highway and railway cuts may be obtained with single or multiple row-charge craters. The fallback and ejecta are potential sources of subgrade material. If the rubble material is of suitable quality, the costs of producing, hauling, and placing aggregate from another site can be eliminated.

An experimental demonstration project of a railway cut was tested in December 1970 by the Nuclear Cratering Group at the Trinidad Dam and Lake Project in Colorado. The explosive excavation design was prepared in accordance with procedures in Chapter 5 of this report. The design used two parallel rows of charges. One row contained eighteen 1-ton charges which were detonated simultaneously followed by the second row of twelve 2-ton and two 1-ton charges simultaneously detonated 150 msec later. Approximately 18,000 yd^3 of sandstone-shale were excavated leaving a broad relatively flat-bottomed crater over 400 ft long and up to 30 ft deep virtually coincident with the predicted size and shape. Figure 15 shows a typical cross section achieved by the detonation. This experiment was an excellent illustration of the capability of explosive excavation for projects of this type. It is discussed in detail in Appendix B.

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*In some operations the most economical blasting results can be obtained by use of a large concentrated charge, or several charges, properly located in one or more small tunnels driven in the rock formation. "Coyote tunnels" are usually horizontal and about 4 or 5 ft in cross section, which is just large enough to provide working space. Tunnel driving is accomplished by conventional means applicable to small headings. The simplest "coyote" layout consists of a main stem or adit perpendicular to the hillside with a single wing or crosscut at the back end driven at 90 deg to the left and right. Many modifications of this arrangement can be used, depending on the particular requirements.

"Coyote" blasting is a form of explosive excavation technology resembling explosive quarrying. The difference lies in the method of design and consequent operational procedures. When charges are designed and emplaced according to the procedures based on systematic studies and testing as presented in this report, the term explosive excavation or explosive quarrying is preferred.

†Limited testing of concept.

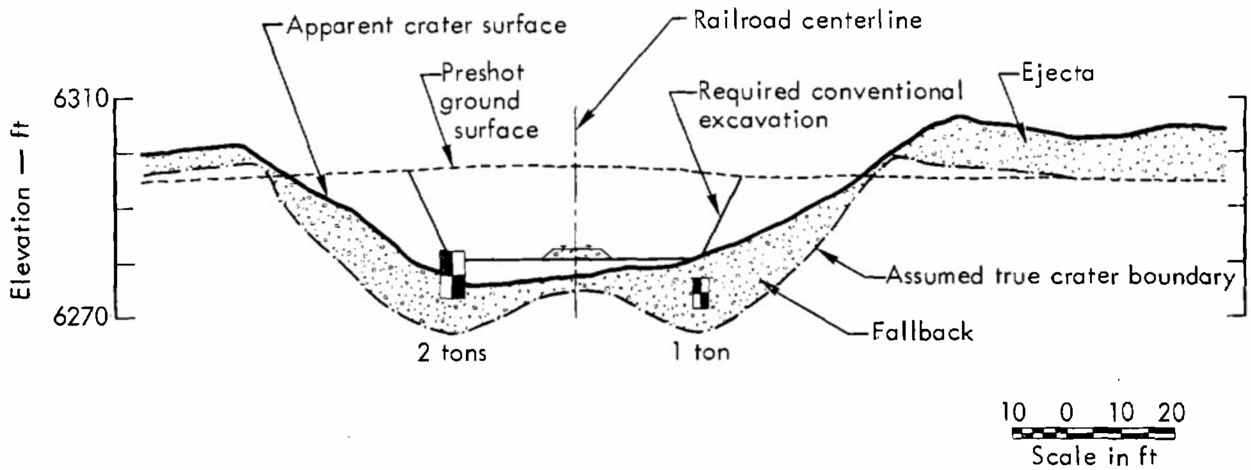

Fig. 15. Typical cross section of Project Trinidad railway cut (at station 94+60).

3.8 QUARRIES AND ROCK FRACTURING*

The conventional method of quarrying is to detonate rows of small column charges. The result is the production of rock in relatively small batches. The technique proposed here is to detonate concentrated charges of tens of tons below the optimum depth of burial for cratering. A significant quantity of fractured rock can be produced without ejecting the material great distances. The size of the charges, the depths of burial, and the natural joint and fracture pattern will influence the particle size distribution of the aggregate, and the quantity obtained from such detonations. Recovery of the material would be facilitated if the detonations were in a hillside sloping 30 deg or more. Testing is required to prove the concept.

A variation of explosive quarrying is the "bulking" or "mounding" of rock with charges detonated at or near quarrying depth. The broken material is removed from the true crater by conventional means. This approach appears to be more economical than cratering when the material excavated must be used for fill. For "mounding" detonations, it may be possible to control the limits of fracturing by certain conventional blasting techniques, such as presplitting or preshearing. Testing is required to prove this concept.

An example of a quarrying application used in Australia was reported by the Engineering News Record in July 1970.[3] According to the report, 500 tons of explosives were used to produce 1.5 million yd^3 of fill for the Ord River Dam. As in the Ripple Rock project, the "coyote tunnel"* technique was used with a single charge designed to shatter the quartzite

*Primarily conceptual applications.

*The "coyote tunnel" technique is particularly adaptable to quarries and heavy sidehill cuts where conventional drilling methods are impractical because of high costs. The method is generally limited to conditions in which the desired degree of fragmentation can be obtained simply by displacing a large mass of material. In many cases, "coyote" blasting is ideal for the production of large rock such as jetty stone or riprap.

into usable sizes for rock fill. The explosive energy went into fragmentation, barely lifting the mountain, with talus sliding down the slope into a prepared area ready to load out.

3.9 EXPEDIENT (BLASTED-INTO-PLACE) DAMS*

Construction of blasted-into-place dams with explosives is limited by topographic and geologic factors. An ideal site where this concept may be applied is a deep, narrow river canyon. Charges are emplaced and detonated in one or both canyon walls in such a manner that materials are ejected across the canyon and the stream is blocked. Dams constructed by this technique might serve as cofferdams to divert streams during construction of the major structure. Testing is required to prove the concept.

An example of the application of high explosives to create a dam across a narrow canyon in the Soviet Union was reported by the Engineering News Record in May 1968.[4] The report indicated that the detonation of 2,000 tons of explosive emplaced in one canyon wall along the Vakhsh River ejected 2.6 million yd^3 of material across the river, depositing the material in the form of an embankment which successfully blocked the river.

3.10 OVERBURDEN REMOVAL*

Overburden stripping with single or multiple rows of explosive charges has great potential for exposing mineral deposits and quarry rocks. By careful control in the positioning and detonation of the charges, deep deposits of ore or rock can be exposed quickly with a possible savings in time and labor over conventional means. The depth of the deposit, the surface relief, and the geologic characteristics of the overburden are the important considerations in determining the excavation design. A time delay between the firing of rows of charges may prove to be advantageous in this concept. Testing is required to develop the technique.

3.11 SUMMARY

The types of projects presented in this chapter include those which have been tried and proved and those which are still conceptual. Keeping in mind that the idea of explosive excavation and the techniques for applying it to various projects are new, it is recommended that <u>all</u> large excavation projects be evaluated for accomplishment by both conventional and explosive means. If both techniques are considered suitable, then the decision as to which to use will usually be decided by cost analyses. For certain projects in some areas explosive excavation may be the only logical choice because of the inadequacy of or exceptional difficulties in applying conventional techniques.

Table 5 is a listing of the project and site data needed to develop preliminary designs and to analyze the feasibility of using explosive excavation on the types of projects discussed in this chapter. With few exceptions, the information required to evaluate a project is the same as that required when the excavation is to be accomplished by conventional methods. As a result, little alteration of data

*Primarily conceptual application.

Table 5. Project criteria and site data required to develop preliminary designs and to analyze feasibility of using explosive excavation techniques.

Criteria or data required	Application: Canals	Watercourses	Harbors	Channel deepening, widening; obstacle removal	Highway, railway cuts	Quarries	Expedient dams	Overburden removal
A. Project Criteria								
Project width at project depth	x		x	x	x			
Alinement limitations	x	x	x	x	x			
Gradient limitations		x			x			
Subgrade specifications		x			x		x	
Cross-section area requirements		x					x	
Plan dimensions			x	x			x	x
Volume requirements						x	x	
Gradation limitations						x	x	
B. Site Data								
Project topography	x	x	x	x	x	x	x	x
Hydrographic survey data	x	x	x	x			x	
Currents	x	x	x	x			x	
Tides	x		x	x				
Wave statistics	x		x	x				
General regional geology	x	x	x	x	x	x	x	x
Local geology (as necessary)								
Type of material	x	x	x	x	x	x	x	x
Density	x	x	x	x	x	x	x	x
Moisture content	x	x	x	x	x	x	x	x
Strength	x	x	x	x	x	x	x	x
Seismic velocity	x	x	x	x	x	x	x	x
Bedding attitudes	x	x	x	x	x	x	x	x
Structural discontinuities	x	x	x	x	x	x	x	
Joint and fracture pattern	x	x	x	x	x	x	x	
Water table	x	x	x		x	x	x	x
Nearby structures survey	x	x	x	x	x	x	x	x
Temperature inversion statistics	x	x	x	x	x		x	x
Wind aloft velocity statistics	x	x	x	x	x		x	x

collection programs is required to incorporate a study of the feasibility of using the explosive excavation method.

REFERENCES

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2. C. H. Noren, "The Ripple Rock Blast," Bulletin Tech. Series No. 97, University of Missouri School of Mines and Metallurgy, Columbia, Mo., 1959.
3. Engineering News Record, p. 20, July 9, 1970.
4. "Russians Fire Off an Instant Rockfill Dam," Engineering News Record, May 30, 1968.

Chapter 4

Explosives

4.1 SCOPE

This chapter discusses the chemical explosives (high explosives and blasting agents) which are most likely to be used as large cratering charges for explosive excavation projects. All common commercially available types of cratering explosives are listed and discussed as to their properties, cratering effectiveness, energy content, and cost. Techniques for the initiation of large charges are discussed. Guidelines are provided for selecting cratering explosives. Discussions on energy equivalence and on the total cost of emplaced charges are included. Procedures for selection based on a cost analysis are presented in Chapter 9. Appendix C contains additional basic information about explosives and the detonation process, classification and shipment regulations, and methods of verifying explosive properties of interest for cratering applications.

4.2 CRATERING EXPLOSIVES

a. High Explosives

Common pure high explosives (HE) and certain high explosive mixtures are listed in Appendix C. Because most of these explosives are relatively expensive and have pressures too high to be useful as cratering explosives, they are not discussed here. Only TNT and nitromethane have been extensively used as cratering charges. TNT continues to be used as a reference explosive; however, it is becoming increasingly difficult to obtain commercially. Nitromethane, because it is a low-viscosity liquid, cannot be used unless it is contained in leak-tight canisters.

b. Nitrate-Based Explosives

Ammonium nitrate (AN) is currently the least expensive primary ingredient for cratering explosives. A common fertilizer, AN can be made to detonate when properly confined and strongly initiated. In dry powder form AN is susceptible to spontaneous deflagration* and possible detonation if stored in large quantities. For explosive applications, ammonium nitrate prills, that is, pea-sized pellets with very high porosity, are usually employed because they are easier to handle and are more stable.

Pure ammonium nitrate contains an excess of oxygen. When detonated, it produces N_2, H_2O, and O_2 and liberates 327 cal/g.[1] Oxygen balance is achieved by adding 5.5% fuel oil which is readily absorbed by the prills. Upon detonation it produces H_2O and N_2 and liberates 890 cal/g, or approximately three times as much energy as pure AN. A mixture of 94.5% ammonium nitrate prills and 5.5% fuel oil is termed "ANFO" (ammonium nitrate—fuel oil) and is the least expensive cratering explosive currently available. Numerous firms manufacture ANFO commercially at prices ranging from 2 to 10 cts/lb. The cratering characteristics of ANFO are very similar to

*See Appendix C for a discussion of deflagration.

TNT and 60% gelatin dynamite.[2] The cost of ANFO, however, is less than a third as much.

ANFO has two major drawbacks: (1) its low density (0.8 to 1.0 g/cm^3), which results in higher costs in emplacing the explosive below ground than other explosives which achieve comparable performance with a smaller volume, and (2) its hydroscopy—ANFO readily absorbs moisture which rapidly increases its decomposition rate and destroys its capacity to detonate (for example, with no water ANFO will detonate satisfactorily in a 1-in. steel tube, but with 5% water, the critical diameter for this degree of confinement is increased to about 4 in.)[3]; consequently, if groundwater is present, ANFO must be placed in sealed packages, thereby raising emplacement costs.

Mixtures of ammonium nitrate powder and high explosives, usually TNT, have been used for years as inexpensive oxygen-balanced explosives. Such mixtures are termed "amatols." An amatol containing 80% AN and 20% TNT is almost perfectly oxygen-balanced. Amatol-water mixtures are slurry explosives and are discussed in Section 4.2d. Properties of AN, ANFO, and some amatols are listed in Table 6.

c. Metallized Explosives

The addition of powdered aluminum to ammonium nitrate ideally produces H_2O, Al_2O_3, and N_2 when detonated and liberates up to 1975 cal/g or about twice the energy release of ANFO.[1] An oxygen-balanced mixture of AN and Al corresponds to 18.4% aluminum by weight.

Table 6. Properties of AN, ANFO, and amatols.[3,4]

Composition	Density (g/cm^3)	Detonation pressure (kbar)	Detonation velocity (m/sec)	Heat of detonation (cal/g)
AN powder	1.07	44	4100	327
Prilled AN	0.81	33	4100	327
98% prilled AN 2% fuel oil	0.80	40	4100	570
94.5% prilled AN 5.5% fuel oil	0.93	60	4560	890
90% prilled AN 10% fuel oil	0.80	45	4100	760
80% fine AN 20% TNT	1.46	88	5100	950
50% fine AN[a] 50% TNT	1.55	139	7000	960
50% coarse AN[a] 50% TNT	1.0	56	5500	900

[a]Energy release is a function of density as well as composition. The composition of the detonation products depends upon the temperature and pressure of the reaction. For example, TNT at low density produces large amounts of CO but little CO_2 and free carbon. At high density, less CO is produced but more CO_2 and C. In the latter case, more energy is also released.[4]

More aluminum can be added if additional oxygen is available. Water is sometimes added to provide extra oxygen. Dry mixtures of ammonium nitrate and aluminum are called "ammonols."

In practice, the aluminum reaction is very complex and does not proceed directly to Al_2O_3. Usually Al_2O is initially formed in the reaction zone behind the detonation wave and the more stable Al_2O_3 formed farther back in the detonation products. The time required for the aluminum reaction depends upon the surface-to-volume ratio of the aluminum particles or flakes. A low ratio (i.e., large particles) can actually reduce detonation pressure as the aluminum initially acts somewhat as a diluent. Since the reaction continues well behind the detonation front, relatively high pressures can be maintained for some distance, the result being a detonation wave with low peak pressure but a comparatively flat pressure profile behind it. Once transmitted into the medium, such waves decay less rapidly than the spiked shocks produced by nonmetallized explosives so that their influence can be felt over a greater distance.

While the 18.4% aluminum figure for ammonol is the theoretical value for oxygen balance, the aluminum particles will in fact contain surface oxides. Depending upon size distribution and quality, anywhere from 20 to 25% aluminum is actually required to achieve oxygen balance with dry ammonium nitrate.[3]

An important effect of the aluminum reaction is to increase greatly the gas bubble energy.[5,6] The result is a reduction of the ratio of shock to gas bubble energy—an effect which is beneficial for cratering. This effect, as it occurs in water, is shown in Fig. 16 and is discussed further in Appendix C.

Aluminum and high explosive mixtures have been used in military applications for many years. A mixture of TNT and aluminum is called "tritonal." Properties of some ammonols and tritonals are listed in Table 7.

d. Slurry Explosives and Blasting Agents

The various nitrate-based explosives previously discussed, (AN, ANFO, amatol, ammonol, etc.) typically suffer from one defect or another such as poor water resistance, excessive sensitivity, or poor storageability. Their desirability as cratering explosives is thus impaired. Through the addition of water, stabilizing agents, and gelling agents, not only are most of these problems overcome but handling of the explosive is simplified. Such mixtures are called slurry explosives when they contain high explosive ingredients; they are called slurry blasting agents when they do not contain such ingredients.

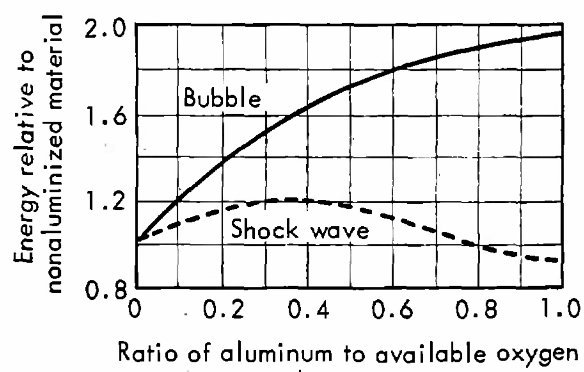

Fig. 16. Effect of aluminum on underwater shock and bubble energy[6] (calculation of aluminum-to-available-oxygen ratio is given in Appendix C).

Table 7. Properties of dry aluminized explosives.[3,4]

Composition	Density (g/cm^3)	Detonation pressure (kbar)	Detonation velocity (m/sec)	Heat of detonation (cal/g)
Ammonols:				
90% AN 10% AL	1.28	115	6100	990
80% AN 20% AL	1.27	130	6400	1470
70% AN 30% AL	1.27	111	5800	1370
Tritonals:				
80% TNT[a] 20% AL	1.42	85	4930	1100
80% TNT[a] 20% AL	1.79	125	7020	1750

[a]See footnote a in Table 6.

In recent years, slurries have become widely used in mining and blasting operations. They have the advantages of high energy release, ease of shipping, storage and emplacement, and relatively low cost.

The chemically active ingredients in most slurry explosives are AN, TNT, and water, while in slurry blasting agents they are AN, water, and sometimes sodium nitrate or aluminum.

The gelling agent in slurries serves two purposes: (1) it insures a homogeneous mixture and prevents settling of components, and (2) it facilitates handling. The slurry and gelling agent can be mixed while the explosive is being pumped into the emplacement cavity where the slurry cures to a rubbery or jelly-like solid which is water resistant. Perfect coupling with the surrounding medium is thus assured, and void spaces within the explosive are minimized. Most slurries are heavier than water (1.2 to 1.9 g/cm^3) and, being highly water-resistant, may be emplaced under water.

Typical slurries contain 40 to 75% AN, 15 to 25% water, 1 to 5% stabilizing and gelling agent, and the rest made up of aluminum, high explosive, or both. Some properties are given in Table 8. Cratering performances, cost, and other data are given in Table 9 with similar information for other explosives.

Ammonium nitrate slurries are frequently shipped and stored in plastic bags. They can be stored in most containers, including aluminum or steel (unless HNO_3 is a constituent); however, because the slurries are new, the effects of long-term storage are not well-known. Slurry blasting agents are very insensitive to flame and normally cannot be detonated with a No. 8 blasting cap. Their compressibility is low and thus they can be used under hydrostatic loads normally encountered in cratering applications. Unconfined

Table 8. Properties of selected slurry explosives and blasting agents.[4,7]

No.[a]	Consistency	Density (g/cm^3)	Detonation pressure (kbar)	Detonation velocity (m/sec)	Heat of detonation (cal/g)	Percent aluminum	Contains HE?
1	Liquid	1.40	99	5850	750	0	Yes
2	Liquid	1.40	104	6050	730	0	Yes
3	Gel	1.30	60	4300	750	2	No
4	Gel	1.33	66	4500	1110	8	No
5	Gel	1.20	85	5700	1450	20	No
6	Gel	1.50	81	5000	1950	35	No
7	Gel	1.65	80	5200	2050	—[b]	Yes
8	Gel	1.95	89	5500	2250	—[b]	Yes

[a]Brand names have been omitted.

[b]Aluminized, but percentage unknown.

critical diameters are usually about 3 in.

e. Aluminized Slurries

The addition of large quantities of aluminum to slurry blasting agents produces a cratering explosive with very high energy release at moderate detonation pressures. The presence of aluminum lowers peak pressures but provides higher sustained gas pressures during the expansion. Energy release per unit weight can exceed twice that of ANFO, and these slurries can excavate up to 80% more volume per unit weight than ANFO or TNT.[7] However, energy release per unit cost is about the same as nonaluminized slurries and approximately one-third that of ANFO. Nevertheless, aluminized slurries are often the best choice when emplacement costs overshadow explosives cost.

Aluminized slurries possess the same shipping, handling, and storage advantages as nonmetallized slurries. They may be plant-mixed or mixed on site.

Properties of some commercially available aluminized slurries are given in Table 8. Cratering properties, costs, and other data are given in Table 9.

4.3 INITIATING TECHNIQUES

Cratering charges are generally initiated using conventional electric blasting caps or exploding bridge wire (EBW) detonators fired from a voltage source designed to suit the application. Electric caps normally contain lead azide plus a few grains of RDX, PETN, or tetryl as primary boosters. The major primary high explosives used in initiating devices are listed and discussed in Appendix C.

Since most cratering explosives are relatively insensitive and cannot be detonated by blasting caps alone, secondary booster explosives are also employed. A suitable secondary booster is an explosive with a high detonation pressure and an energy release of at least 1000 cal/g. Common booster explosives are 50/50 pentolite, composition C-4, composition B,

Table 9. Measured properties and calculated parameters of representative cratering explosives.

Explosive	Detonation pressure (kbar)	Bulk specific gravity	Detonation velocity (m/sec)	Impedance (m/sec)	Heat of detonation (cal/g)	Nominal cost ($/lb)	Energy/cost (Mcal/$)	Excavated volume relative to equal weight of TNT[a]
ANFO	60	0.93	4560	4240	890	0.06 ± 0.04	6.75	1.0-1.1
AN slurry	104	1.40	6050	8470	730	0.15 ± 0.05	2.22	1.0-1.2
AN slurry (2% Al)[b]	60	1.30	4300	5590	750	0.08 ± 0.05	3.41	1.0-1.2
AN slurry (8% Al)[b]	66	1.33	4500	5990	1110	0.13 ± 0.05	2.52	1.2-1.4
AN slurry (20% Al)[b]	85	1.20	5700	6840	1450	0.20 ± 0.07	2.19	1.5-1.7
AN slurry (35% Al)[b]	81	1.50	5000	7500	1950	0.25 ± 0.10	2.52	1.6-1.8
TNT	220	1.64	6930	11360	1102	0.25 ± 0.05	2.00	1.00
Nitromethane	125	1.13	6320	7140	1126	0.33 ± 0.02	1.55	1.0-1.3
Composition C-4	257	1.59	8040	12780	1350	0.34 ± 0.10	1.80	1.2-1.4

[a]That is, "Cratering Effectiveness" as measured by small charge detonations in sand. Absolute cratering performance in terms of volume excavated per pound of explosive depends on the size of the shot; it is less for larger shots. Relative performance, on the other hand, is not as sensitive to charge size.

[b]Slurry blasting agent.

and TNT (these and others are listed in Appendix C, Tables C3 and C4). Aluminized slurry blasting agents require considerable boosting to insure complete detonation. The booster should not be placed closer than 6 in. from the bottom of the cratering charge, and it should have explosive surrounding it for at least a 4-in. thickness in all directions. There should be intimate contact between the booster and the main charge. Whenever possible the booster column should extend at least three-fourths the length of the cratering charge. In some cases it may be advisable to construct a booster column up the entire length of the charge.

Care must be taken to insure that the booster is also properly initiated. Pentolite and composition C-4 can be fired with 50-grain PETN detonating cord or a No. 6 blasting cap. For other booster explosives, 100-grain PETN detonating cord or a No. 8 blasting cap is preferred.

For several cratering charges, one effective firing chain is a surface blasting cap connected through a detonating cord trunkline to lengths of 50-grain PETN detonating cord which run downhole through the stemming to the boosters within the cratering charges.[8*] More powerful detonating cord should be used only if it is loosely enclosed in steel or strong plastic tubing because the cord itself may initiate deflagration or low-order detonation and create gas voids in the main charge. Encasing the down-hole detonating cord in tubing not only minimizes the likelihood of deflagration but helps to insure integrity of the stemming and reduces the likelihood of damaging the initiation system.

*Reference 8 also discusses methods of connecting lengths of detonating cord together.

In those cases (see Chapter 5) where it is desirable to fire the charges sequentially in order to minimize ground shock and airblast effects, or to achieve a directed excavation effect by firing rows in sequence, a delay firing system must be carefully designed and laid out to avoid premature cutoffs and consequent misfires. One method by which delays may be achieved is to use commercially available delay connectors and a single blasting cap. The delay connectors are placed as required throughout a detonating cord trunkline. Another method is to use delay blasting caps placed within the individual charges. Blasting caps offer delay times from 5 msec to several seconds.[9] Delay intervals are selected as a part of the explosive excavation design process as discussed in Chapter 5.

4.4 SPECIFICATION OF CRATERING EXPLOSIVES

a. Cratering Performance Guidelines

The selection of a cratering explosive will involve a number of compromises. A tradeoff must be made between the maximum excavated volume per pound of explosive and the cost. It is not possible to write detailed explosive specifications in this section because too many variables are involved. On-site drilling costs, transportation, storage, and labor costs depend upon specific applications. However, from the discussions in Appendix C, guidelines can be set down to provide a basis for specifying cratering explosives. These guidelines are as follows:

(1) Total heat of detonation and especially gas bubble energy should be as high as possible. Unfortunately, high energy explosives are expensive and a compromise must be reached between explosive cost and emplacement cost. A high ratio of heat of detonation to explosive cost is always advantageous. Typical values of such ratios, using nominal market prices as of this writing, are presented for several explosives in Table 9. The experimentally determined cratering effectiveness values (see Section 4.5) are also included in Table 9.

(2) As a rule of thumb, the best cratering performance can be expected when the explosive-to-rock-impedance ratio, I_e/I_m, is between 0.2 and 1.0, and the closer to 1.0 the better. For this calculation, I_e may be taken as the product of the explosive bulk specific gravity and the characteristic detonation velocity; similarly, I_m may be taken as the product of the media bulk specific gravity and its characteristic seismic velocity. In some media, for example soil and clay shale, the impedance ratio usually exceeds unity regardless of the explosive. Under such circumstances a low impedance explosive, such as ANFO, can be selected although impedance matching is not as important in common excavation as in rock excavation.

Detonation impedances for some explosives are given in Table 9, and acoustic impedances of some materials are given in Table 10.

(3) An upper limit to detonation pressure is desirable because an explosive with an excessively high detonation pressure will dissipate considerable energy in excessive crushing and plastic deformation near the charge. There is no clear indication as to what this upper limit should be but experience has shown that explosives with detonation pressures

exceeding 150 to 200 kbar tend to perform poorly as cratering explosives.*

b. Operational Considerations

A practical cratering explosive must be capable of being shipped to the site with comparative ease, stored without hazard for several months, and placed down-hole with efficiency. Also, fumes given off by the detonation must not be so toxic as to require evacuation of personnel. Five characteristics govern whether the above needs will be met: (1) the explosive density, (2) its water resistance, (3) its classification in the Code of Federal Regulations (CFR), (4) its viscosity or pourability, and (5) its composition.

Density and water resistance are important if there is a likelihood of water in the borehole due to seepage or runoff. The explosive must then be dense enough to displace water and sufficiently water-resistant to detonate when wet. A specific gravity of 1.1 or more is preferable under such circumstances. Nearly all high explosives, ammonium nitrate slurries, and gelatin dynamites are sufficiently water-resistant for application in flooded boreholes. ANFO and other dry ammonium nitrate mixes, such as ammonol, are not.

When water is not present, explosive density is important only from the standpoint of emplacement cost (assuming, of course, the impedance preferences discussed in Section 4.4a have been satisfied); i.e., a denser explosive of comparable performance will be cheaper to emplace because the charge cavity is smaller.

Shipping costs are directly related to the CFR classification (see Appendix C). Cratering explosives classified as oxidizing materials are reasonably inexpensive to ship. Class A explosives in large quantities are very expensive to transport and therefore should be avoided. Class C explosives are usually no more expensive to ship than oxidizers but there are restrictions as to the size of the shipment. Nitromethane, although it can be used as a powerful explosive, has no explosive

*Methods for measuring energy release and detonation velocity and for calculation of detonation pressures are presented in Appendix C.

Table 10. Acoustic impedances of some materials.[10,11]

Media	Nominal bulk specific gravity	Nominal seismic velocity (m/sec)	Acoustic impedance, I_m (m/sec)
Alluvium	1.54	1000	1540
Basalt	2.59	5400	13990
Clay shale	2.06	2000	4120
Granite	2.65	5100	13510
Limestone	2.66	5200	13830
Sandstone	2.40	2400	5760
Water	1.00	1460	1460

classification and is shipped as an industrial solvent.

It should be possible to pump or pour an ideal cratering explosive into the charge cavity without leakage into underground fissures and cracks. Upper and lower limits on the viscosity of the mixture are required unless the explosive is to be placed in sealed containers—an added expense. To accommodate these requirements, gelling agents are often added to slurries as they are being pumped down-hole. The mixture then sets up as a rubbery or putty-like solid. To minimize leakage, down-hole viscosity should be high. Some dry mixes such as prilled ANFO and pelletol (pelletized TNT) usually pose no pumping or seepage problem. Nitromethane, on the other hand, has a viscosity about equal to that of warm water and consequently must always be emplaced in sealed cavities or leak-proof containers.

Toxic fumes are minimized when the explosive is properly oxygen-balanced. An excess of oxygen in an explosive, such as might arise from the exposure of AN to water or high humidity, can produce brown nitrogen dioxide upon detonation and, under some circumstances, during storage. An oxygen deficiency such as occurs in TNT, pelletol, and tritonal may, depending on density, yield colorless but toxic carbon monoxide.* Generally, if oxygen imbalance is held within 4% by weight, toxic fume production will be negligible. Stability during storage is also improved if the above criterion is met. Most nitrate-based explosives, such as amatol, ANFO, and most slurries, are adequately exygen-balanced so that toxic fume production is minimized.

4.5 CRATERING EFFECTIVENESS AND ENERGY EQUIVALENCE

In this report "cratering effectiveness" or "cratering performance" is taken to mean the apparent crater volume excavated per unit weight of explosive compared to the volume excavated per unit weight of the reference explosive, TNT (see Table 9). The values in Table 9 were developed from tests in sand, but they may be applicable to any medium of similar density, strength, and impedance. As of this writing, comparable data for rock are unavailable. Care must be taken then when applying the values in Table 9 to rock. Calibration shots as discussed in Appendix D should be carried out whenever possible.

The term "energy equivalence" is taken to mean simply the ratio of heat of detonation of a given explosive to that of the reference explosive. Heats of detonation of the explosives discussed in this chapter are given in Table 6 through Table 9. The curves for airblast, ground motion, underwater shock, and missile range in Chapter 6 are based on tons of TNT. The energy equivalence of an explosive (A) in terms of tons of TNT can be found from the formula

$$\text{(Tons TNT)} = \text{(Tons explosive A)}\left(\frac{Q_A}{Q_{TNT}}\right), \tag{13}$$

where Q is the heat of detonation in calories per gram.

*For example, a 100-ton detonation of low density TNT will release 12 to 16 tons of carbon monoxide to the atmosphere.

4.6 EXPLOSIVE SELECTION BASED ON OVERALL COST CONSIDERATIONS

Candidate cratering explosives may be evaluated by considering explosive cost and density, relative cratering performance, charge geometry, and emplacement cost. Explosive costs can differ by a factor of as much as twenty while cratering performance in terms of excavated volume per weight rarely differs by a factor of more than two. Consequently, expensive explosives with marginally superior performance are usually undesirable while the cheaper explosives are to be preferred. The cheapest explosive, however, is not always the best. The cost of emplacing it must also be considered. A compromise must be made between explosive cost and emplacement cost such that the total cost of the explosive emplaced ready for detonation is minimized. A method of selection based on cost minimization is presented in Chapter 9.

4.7 SUMMARY

Most high explosives are too expensive and have detonation pressures too high for use as cratering charges. An explosive with a high gas bubble energy is most efficient for cratering.

Ammonium nitrate is the least expensive explosive ingredient available and, when mixed with fuel oil, makes an excellent and inexpensive cratering explosive. However, it has the drawbacks of low density and poor water resistance. The addition of aluminum to ammonium nitrate greatly increases energy release and also reduces the ratio of shock energy to bubble energy. Adding water and gelling agents to such a mixture gives a good slurry cratering explosive which is dense, water-resistant, and easily handled. Slurry explosives and blasting agents are available in a wide variety of formulations.

Cratering charges should be adequately boosted. Current recommended procedure is to require a continuous column booster extending for at least three-fourths the length of the main charge. If detonating cord is run downhole through the charge, it should be encased to prevent possible deflagration of the cratering explosives.

The primary characteristic desired in a good cratering explosive is a high gas bubble energy. Other desirable characteristics are low cost, a high heat of detonation, an impedance ratio with the medium between 0.2 and 1.0, and a detonation pressure under 150 to 200 kbar. The explosive should have sufficient density to displace water if necessary, it should have high water resistance if emplaced in very damp or wet boreholes, it should be classified as an oxidizing material (to minimize shipping costs), and it should have a high viscosity in the emplaced configuration.

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Chapter 5

Excavation Design

5.1 SCOPE

This chapter provides information and procedures currently used to design explosive excavations. Going beyond the qualitative introduction to cratering and the representative parameters discussed in Chapter 2, this chapter presents basic quantitative cratering data and outlines procedures for applying the data to the design of single-charge and row-charge excavations, and (with somewhat less experimental verification) to multiple row-charge excavations. Related matters including interconnecting row craters, underwater cratering, charge shapes, stemming, and delayed row-charge detonations are also discussed.

Appendixes D and E supplement the information contained in this chapter. The design and conduct of site calibration tests and the preparation of the basic charts and graphs used here are discussed in Appendix D. In looking toward further simplification, Appendix E introduces other useful design charts and explains how these may be prepared and used. The design procedures in Appendix E are extensions of those in this chapter and rely upon basic relationships found in this chapter and in Chapter 2.

5.2 SINGLE-CHARGE CRATERS

Because maximum explosive efficiency is desired for excavation applications, the design information used here is based on burial at optimum depth; i.e., the depth below ground surface at which the detonation will produce the largest apparent crater. Explosive engineering projects such as quarrying or underwater blasting may require that charges be placed at some different depth.

Experimentally obtained single-charge crater dimensions for three materials are given in Figs. 17, 18, and 19.[1-8] The optimum depth of burial and the radius, depth, and volume of the apparent crater are read directly from these graphs. This information, which is used to select the weight and corresponding depth of burial for single charges, is also used to design row-charge craters and will be referred to frequently in this chapter. The optimum scaled crater dimensions are shown on these charts to permit computation of crater dimensions for charge weights outside the range of the graphs.

To illustrate the use of these charts, assume that a single crater with a depth of 27 ft in rock is desired. Figure 17 shows that a charge of 27 tons of TNT buried at 48 ft will excavate the desired crater, and that it will have a radius of 54 ft and a volume of almost 4000 yd^3.

It is pertinent at this time to point out that the crater dimensions in Figs. 17, 18, and 19 are based on averages derived from experimental data which is characterized by appreciable amounts of scatter. Considerable departures from the values given by these graphs could be experienced; thus it is prudent to think of the crater dimensions as reliable with a 20% margin.

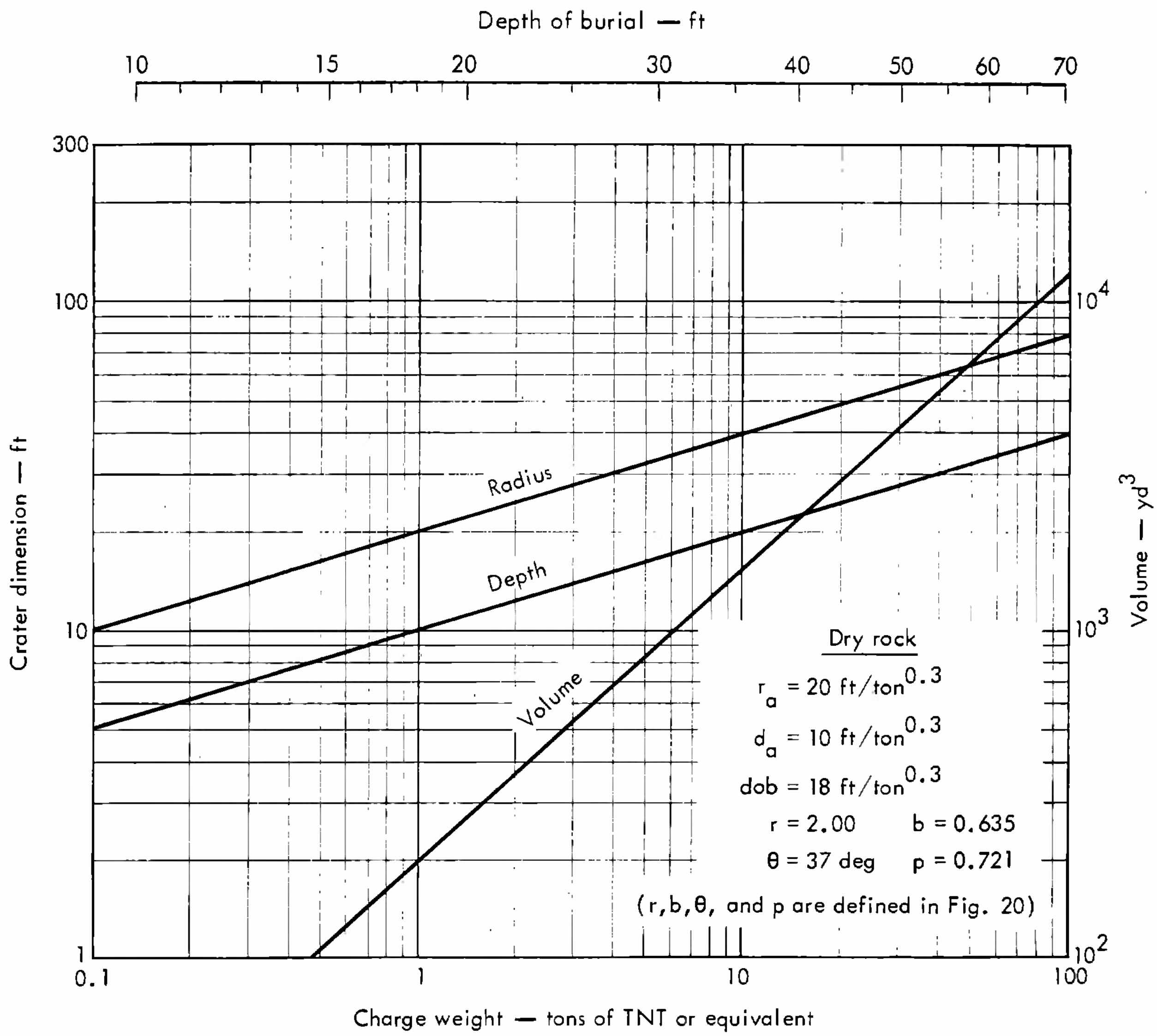


Fig. 17. Crater dimension data for dry rock.

When the material to be excavated varies widely from the materials represented by Figs. 17 through 19, it is suggested that test charges be detonated and the resulting crater dimensions be scaled up to the required charge weight level. Basic procedures for conducting these "site calibration" tests are presented in Appendix D.

5.3 ROW-CHARGE CRATERS

As discussed in Chapter 2, if a row of charges is properly emplaced and detonated simultaneously, a smooth channel or linear crater will be excavated. The design of row-charge craters is based on single-charge cratering data and the enhancement of row crater dimensions which occurs when charges are spaced closer together. The enhancement phenomenon permits flexibility in the choice of individual charge weights to excavate linear craters.

a. Flat Terrain

The design of a row-charge crater in flat, level terrain is less complicated

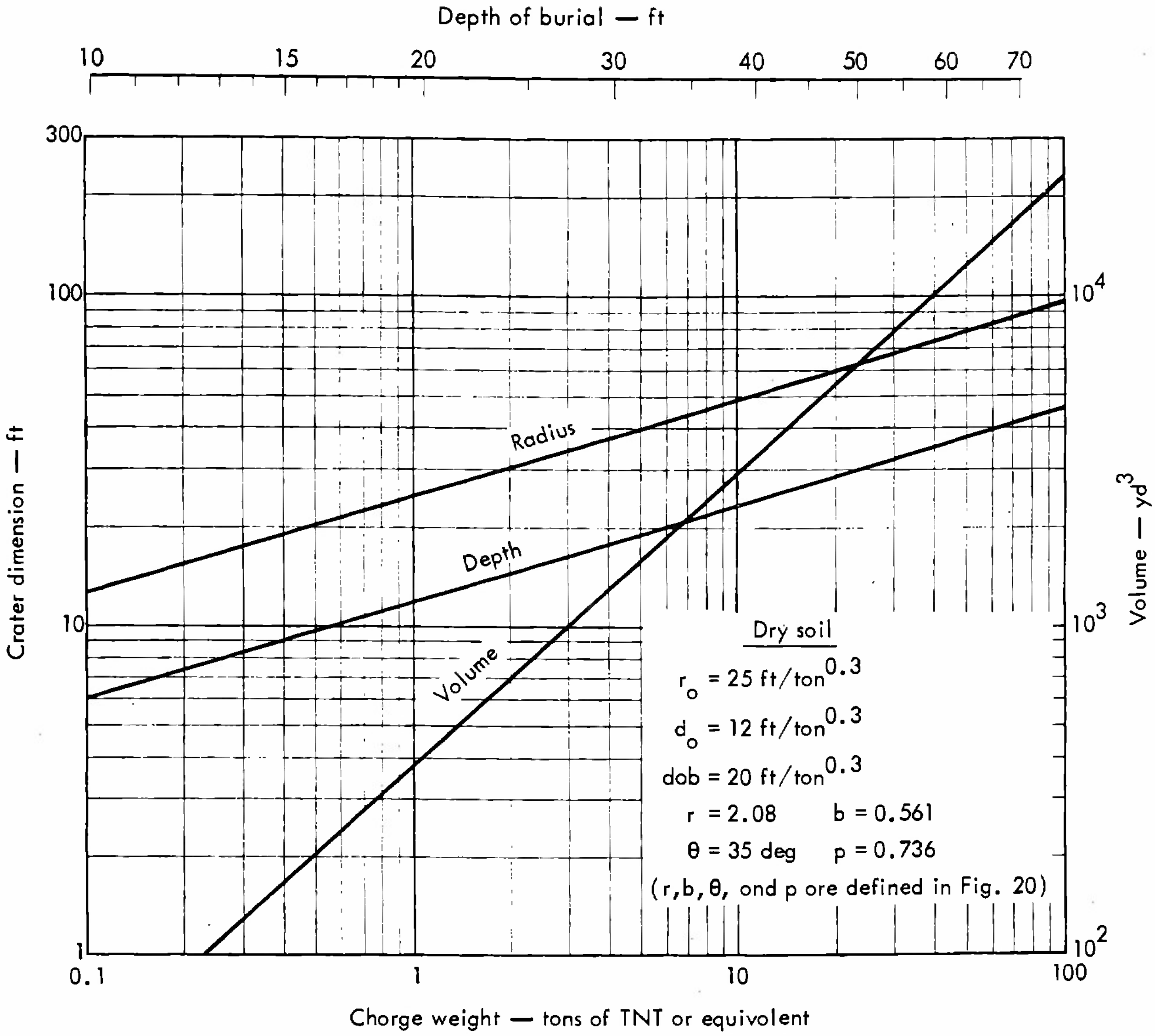


Fig. 18. Crater dimension data for dry soil.

than the design of a similar excavation through varying terrain, and will, therefore, be taken up first.

The first step in designing a row-charge crater in level terrain is to select the single-charge weight from the appropriate single-charge dimension chart (Figs. 17 through 19) which will produce the required half-width (i.e., radius) and depth of the cut. It is likely that only one of these two dimensions will be required to determine the weight; for example, a given weight may be adequate for the depth of a cut but not the width, and therefore width requirement would govern the weight to be used.

After the single-charge weight has been determined, the weight and spacing of the charges in the row can be altered, if desired, in response to construction considerations involved in emplacing the charges. This also is done by considering enhancement which, from the derivation in Chapter 2, can be expressed in terms of charge spacing:

$$e^2 \simeq \frac{1.4}{S/R_a} \qquad (14)$$

where

e = enhancement of single-charge crater dimensions (both width and depth are equally enhanced)

S = charge spacing

R_a = (optimum) single-charge crater radius

Because e = 1 (no enhancement) if $S/R_a = 1.4$, the single-charge weight determined from Figs. 17 through 19 would be the weight required for a row of charges spaced 1.4 R_a apart to achieve the required width and depth. The charges would be emplaced at optimum depth of burial as for single charges of this weight. If a closer spacing is used ($S/R_a < 1.4$ and $e > 1$), then it is no longer necessary

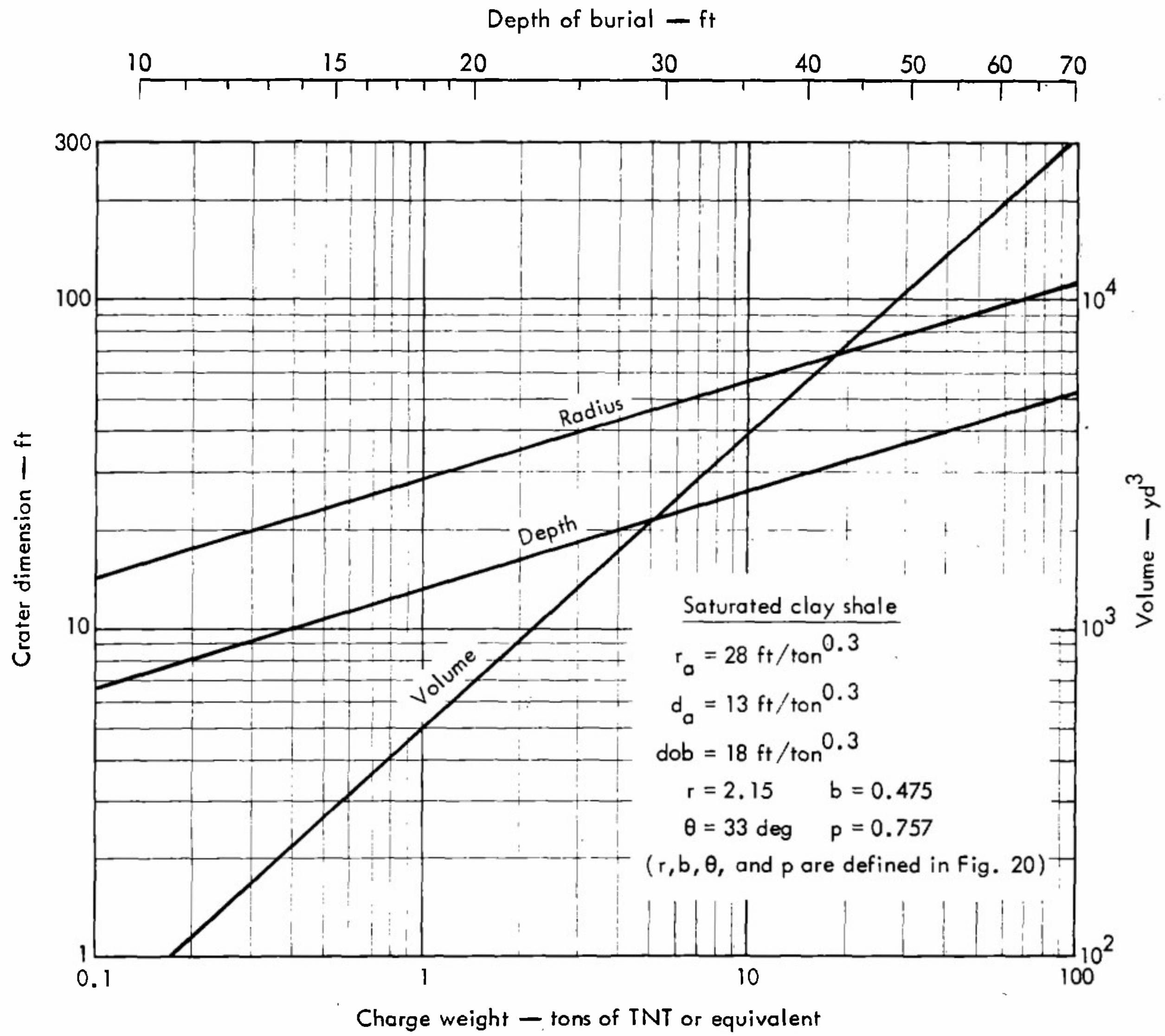


Fig. 19. Crater dimension data for saturated clay shale.

to use the same weight charges to obtain the required crater dimensions; however, the smaller charges would still be emplaced at the depth of burial used for the 1.4 R_a spacing (i.e., at the optimum depth shown for the single charge weight on the crater dimension chart). It may be noted that the enhancement phenomenon implies that if the charge weights determined for 1.4 R_a spacing are used at a closer spacing without increasing their depths of burial, the design no longer achieves the efficiency associated with optimum depth of burial. For a smooth excavation, charges in a row should not be spaced farther apart than 1.4 R_a; spacings greater than 1.4 R_a will result in the cusping of the crater sides and bottom.

Assume that the single-charge weight which will produce the required half-width (radius) and depth of the row crater is Y_s, but that it is desirable to use charges of lower weight, Y_r, in the actual excavation of the row crater. The required amount of enhancement can then be expressed by:

$$e = \left(\frac{Y_s}{Y_r}\right)^{0.3} . \qquad (15)$$

Combining Eqs. (14) and (15), an expression for the proper spacing of the smaller charges, Y_r, is obtained:

$$\frac{S}{R_a} = 1.4 \left(\frac{Y_r}{Y_s}\right)^{0.6} , \qquad (16)$$

where R_a is the single-charge crater radius of a charge of weight Y_r. The radius, R_a, is read from the appropriate single-crater dimension chart. The depth of burial for charge weight, Y_r, is obtained from the same chart, but it must be increased by the amount of enhancement. This increase in depth is equivalent to burying Y_r at the depth indicated for Y_s on the single-charge crater chart.

As an example of the foregoing, suppose it is desired to excavate a diversion channel 75 ft wide and 23 ft deep in dry soil. Figure 18 shows that the depth of the channel determines the charge weight, and that it will require a single-charge weight (i.e., unenhanced row-charge weight) of 11.0 tons. However, assume that 2-ton charges are desirable because of construction problems in emplacing charges larger than this size. Equation (15) gives the required amount of enhancement as:

$$e = \left(\frac{Y_s}{Y_r}\right)^{0.3} = \left(\frac{11.0}{2}\right)^{0.3} = 1.67.$$

The charge spacing for the 2-ton charges is found from Eq. (16):

$$\frac{S}{R_a} = 1.4 \left(\frac{Y_r}{Y_s}\right)^{0.6} = 1.4 \left(\frac{2}{11.0}\right)^{0.6} = 0.51,$$

where R_a is 31 ft for a 2-ton charge; therefore:

$$S = 0.51\,R_a = 0.51 \times 31 = 15.5 \text{ ft}.$$

The depth of burial of the 2-ton charges is the optimum depth obtained from Fig. 18 increased by the amount of enhancement; i.e.,

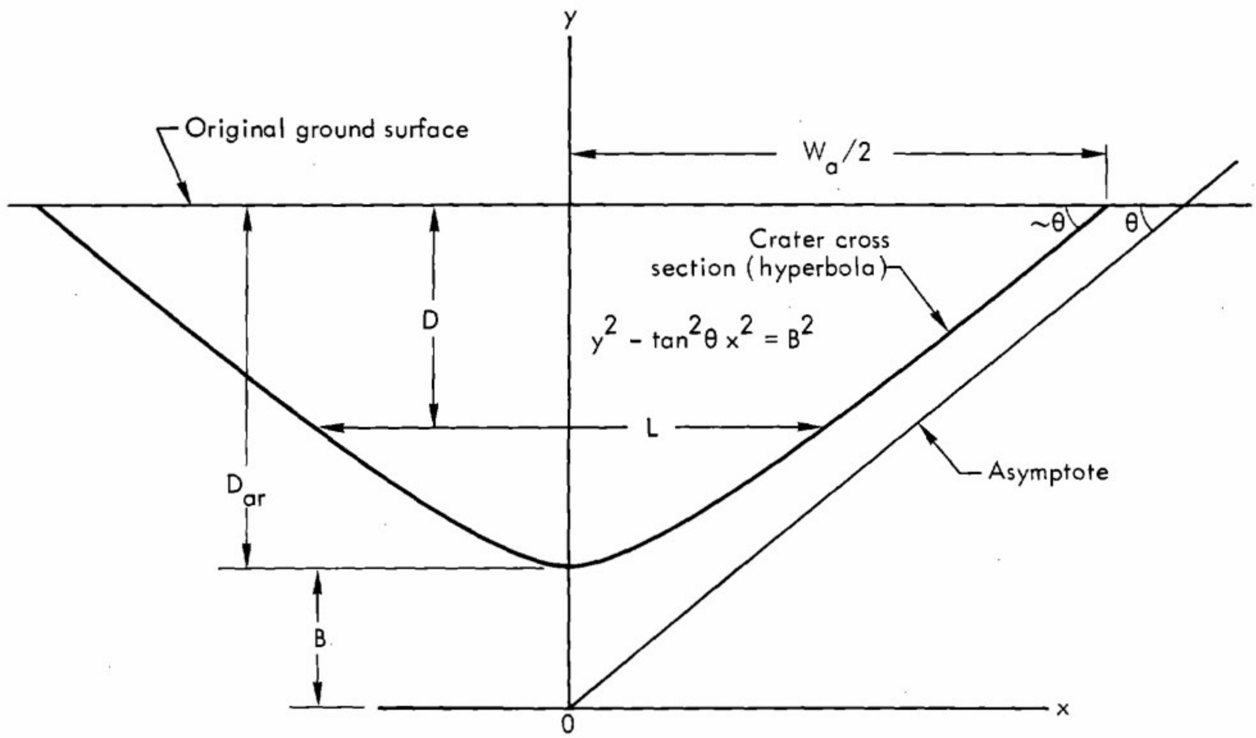
$$24.5 \times 1.67 = 41 \text{ ft},$$

which, it will be noted, is the same depth shown on Fig. 18 for a 11.0-ton charge, the original single-charge weight, Y_s, which was determined to be necessary for excavating the channel. Using the 2-ton charges at a depth of burial of 41 ft and spaced 1.5 ft apart will produce a crater with the following width and depth:

$$W_a = 2\,(eR_a) = 2\,(1.67 \times 31) = 103 \text{ ft}$$

$$D_{ar} = eD_a = 1.67\,(14) \simeq 23 \text{ ft.}$$

So far only the width or depth of the apparent crater have been considered as the design criteria; however, these criteria may not be sufficient for specifying the desired cut. If a project requires the excavation of a channel that can accommodate a specific navigation prism (i.e., one in which the width of the channel, L, is specified at some depth, D, below the original ground surface), then Eq. (17) in Fig. 20 can be used to compute the



$$Y = \left[p\frac{D}{d_a} + \sqrt{p^2\frac{D^2}{d_a^2} + \frac{L^2}{4r_a^2} - \frac{p}{b+1}\frac{D^2}{d_a^2}}\right]^{3.33} \qquad \text{Eq. (17)}$$

where $p = \dfrac{b+1}{2b+1}$

$b = \dfrac{\tan^2\theta\, r^2 - 1}{2}$ (and $B = bD_{ar}$)

$r = \dfrac{r_a}{d_a}$

(NOTE: Values for the parameters r, b, p, and θ are given in Figs. 17 through 19 for the three cratering media discussed in this chapter. For other media see Appendix D.)

Fig. 20. Typical hyperbolic row-crater cross section.

necessary single-charge weight. The concept of enhancement can be used as before to reduce this weight in a row of charges.

For example, assume that it is desired to excavate a canal through hard rock with a width, L, of 30 ft at a depth, D, of 10 ft below ground level. The single-charge weight is computed by Eq. (17) using the crater constants p and b given in Fig. 17:

$$Y_s = \left[0.72\left(\frac{10}{10}\right) + \sqrt{0.52\left(\frac{100}{100}\right) + \frac{900}{4(400)} - \frac{0.72}{1.64}\left(\frac{100}{100}\right)}\right]^{3.33}$$

$$= (1.52)^{3.33}$$

$$= 4.0 \text{ tons.}$$

This charge weight can be reduced by close spacing and enhancement using the procedures previously discussed.

b. Varying Terrain

The design of a row of charges to excavate a channel through varying terrain is an extension of procedures described in the preceding section. There are two design procedures which can be applied to a cut through varying terrain, and the choice of which is to be used in a particular case must be based on a consideration of operational factors. The two design techniques are very similar and differ only in the manner in which the concept of enhancement is applied. One method will result in a row of charges with varying weights, and the other will result in a row with all charges equal in weight. For convenience, the former is termed the "constant-enhancement" method (i.e., constant S/R_a), and the other as the "constant-charge-weight" design. In some instances, particularly where the terrain relief is not great, it may be more economical to use the constant-charge-weight method because of construction advantages of having a large number of charges the same size. If cutting through steep terrain where only a few relatively large charges are required, then the constant-enhancement approach may have a slight advantage.

(1) Constant-Enhancement Method

The constant-enhancement approach is discussed first. The design begins by determining the single-charge weight of the largest charge in the row, this being the charge directly beneath the point of highest elevation (or deepest cut). This charge weight is reduced to a convenient value by the use of close spacing and enhancement, if necessary, and then the remaining charges in the row are computed. Whatever charge spacing (in terms of S/R_a) is selected to reduce the largest charge to a convenient weight is maintained over the length of the detonation. Computing the weight and positioning of the remaining charges in the row requires a slightly different procedure depending on whether the excavation is a channel with a specified bottom elevation or one which will contain a navigation prism.

Consider first a cut with a specified bottom elevation. When the maximum single-charge weight has been determined according to the highest elevation (or, more exactly, the greatest depth of cut) and the appropriate crater dimension chart, it can be reduced to the desired level by Eq. (16), which establishes the charge spacing in the row in terms of S/R_a. The weights required in the remainder of the row are determined by means of a modification to the crater dimension chart. The modification consists of shifting the depth curve up the vertical axis by the amount of enhancement, this amount being computed by Eq. (15). Weights for any other depth of cut can then be read directly from the modified graph. Charge spacings are the appropriate fraction (S/R_a) of the unenhanced radius for a given charge, and depths of burial are read at the intersection of the depth of cut and the unenhanced depth curve.

In the typical case of a cut through varying terrain, there will be adjacent charges of unequal weight. The spacing between adjacent charges should be the average of the spacings computed for each of the charges.

Horizontal positioning of the charges is then determined by the following process. Assume that the weight of a charge in a row is Y_m and that the crater radius for this charge is R_{am}. The spacing, S_m, for this charge would then be:

$$S_m = \left(\frac{S}{R_a}\right) R_{am}. \quad (18)$$

Similarly, the spacing for an adjacent charge, Y_n, would be:

$$S_n = \left(\frac{S}{R_a}\right) R_{an}. \quad (19)$$

The actual spacing between Y_m and Y_n is the average of S_m and S_n; i.e.,

$$S_{m-n} = \frac{S_m + S_n}{2}. \quad (20)$$

This procedure is repeated for the remaining charges in the row. An illustrative example of designing a cut with a specified bottom elevation through varying terrain is given below.

Assume that it is desired to excavate a channel with a constant bottom elevation through dry rock along the profile shown in Fig. 21. The deepest cut is 25 ft (i.e., 37 - 12), and from Fig. 21 the maximum single-charge weight (Y_s) required is found to be 21 tons. However, assume that emplacement considerations dictate

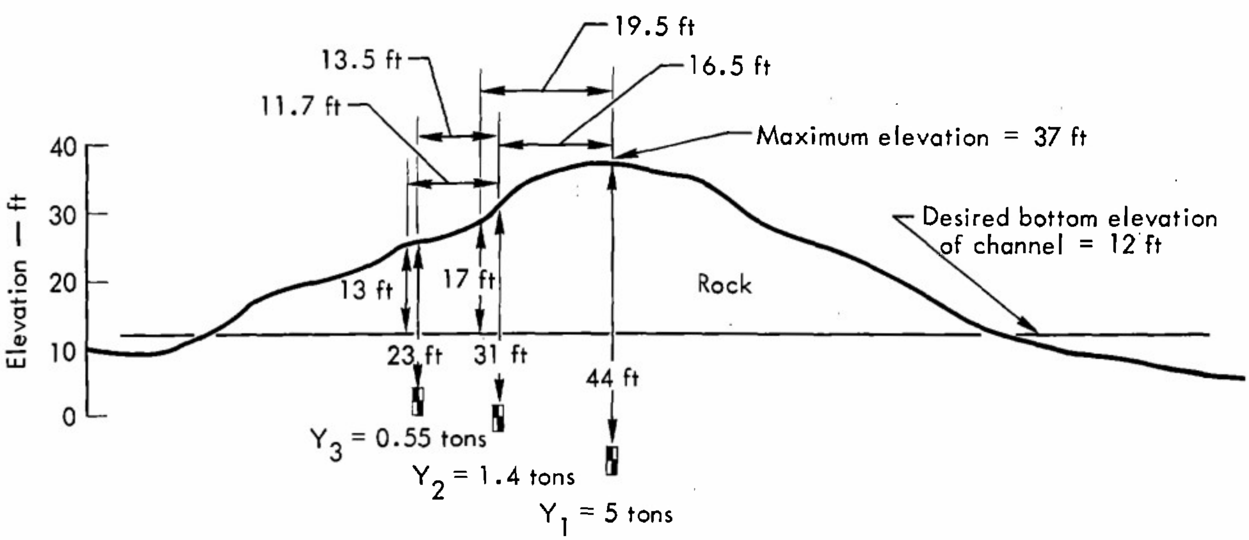


Fig. 21. Row-charge design to excavate channel with constant bottom elevation.

a maximum weight of 5 tons (Y_r). The necessary amount of enhancement is given by Eq. (15):

$$e = \left(\frac{Y_s}{Y_r}\right)^{0.3} = \left(\frac{21}{5}\right)^{0.3} = 1.54,$$

and Eq. (16) gives the required charge spacing in terms of S/R_a as:

$$\frac{S}{R_a} = 1.4\left(\frac{Y_r}{Y_s}\right)^{0.6} = 0.61.$$

The charge spacing of $S/R_a = 0.61$ will be maintained throughout.

It will now be convenient to modify Fig. 17 by shifting the depth curve vertically up on the graph by the amount of enhancement; i.e., a factor of 1.54. The modified form of the crater dimension chart can then be used to read required charge weights directly. An appropriately modified Fig. 17 for use in this example is shown in Fig. 22.

It has been established that the first and largest charge is 5 tons. From Fig. 22 it is found that R_a for a 5-ton

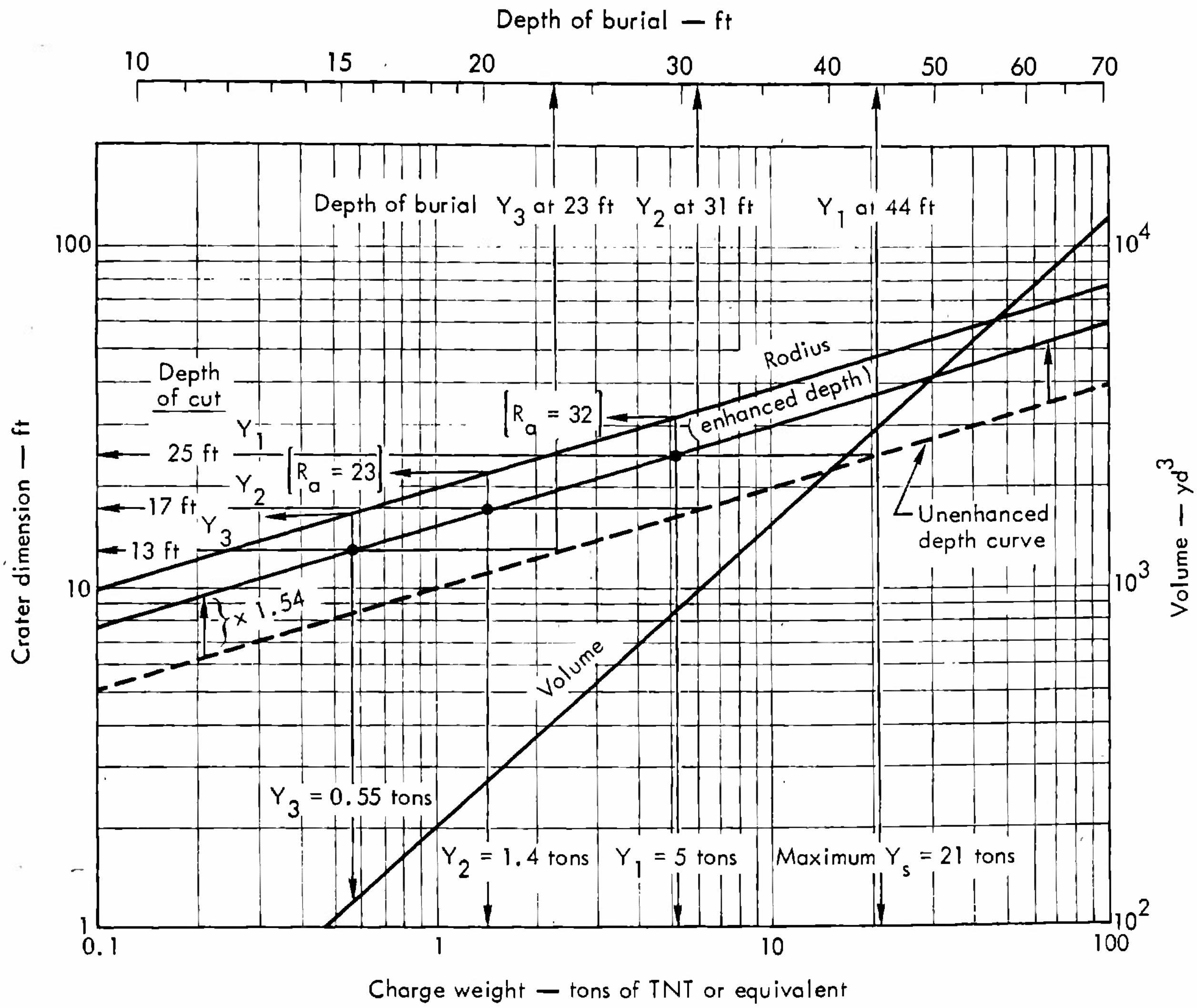


Fig. 22. Modified crater dimension chart to be used with example in Fig. 21.

charge is 32 ft. Designating this first charge as Y_1, then:

$$S_1 = 0.61 \times 32 = 19.5 \text{ ft.}$$

This charge of 5 tons will be buried at the same depths as the 21-ton single charge; i.e., 44 ft.

Proceeding with the design to the left of the highest point of the profile, the depth of cut a distance S_1 (i.e., 19.5 ft) to the left of Y_1 is measured. This depth of cut is 17 ft, and from the enhanced depth curve in Fig. 22, Y_2 is 1.4 tons, R_a is 23 ft, and the depth of burial is 31 ft. It follows that:

$$S_2 = 0.61 \times 23 = 13.5 \text{ ft.}$$

So that the actual spacing between Y_1 and Y_2 is adjusted to be:

$$S_{1-2} = \frac{S_1 + S_2}{2}$$

$$= \frac{19.5 + 13.5}{2} = 16.5 \text{ ft.}$$

The weight of the third charge, Y_3, is determined by measuring the depth of cut a distance S_2 (i.e., 13.5 ft) to the left of Y_2. The depth of cut is 13 ft and, according to Fig. 22 Y_3 is 0.55 tons, R_a is 16.5 ft, and the depth of burial for Y_3 is 23 ft. Thus:

$$S_3 = 0.61 \times 16.5 = 10 \text{ ft}$$

and

$$S_{2-3} = \frac{13.5 + 10}{2} = 11.7 \text{ ft.}$$

The design of the row charge is continued to the end of the cut, to both the left and right of Y_1, by continuing this procedure.

Consider now the design of an excavation which will circumscribe a navigation prism. This design procedure requires more computations than the foregoing, although essentially the same steps are followed. Rather than being able to obtain charge weights directly from a modified crater dimension chart, the basic single-charge weights have to be individually computed by means of Eq. (17). The enhancement (e) and spacing (S/R_a) used to reduce the largest weight to a convenient level is also used for the other charges in the detonation.

The procedure for designing a cut to accommodate a navigation prism is most easily illustrated by an example. Assume it is desired to excavate a channel through dry rock represented by the profile in Fig. 23. Let the design specifications for the channel be a width of 25 ft at elevation zero. Starting at the highest elevation of 33 ft, the required single-charge weight is determined by Eq. (17):

$$Y_1 = \left[0.72\left(\frac{33}{10}\right) + \sqrt{0.52\left(\frac{1089}{100}\right) + \frac{625}{4(400)} - \frac{0.72}{1.64} \cdot \left(\frac{1089}{100}\right)}\right]^{3.33}$$

$$= 65 \text{ tons.}$$

If it is desired to reduce the 65-ton charge to 30 tons, then Eq. (15) gives the required enhancement:

$$e = \left(\frac{Y_s}{Y_r}\right)^{0.3} = \left(\frac{65}{30}\right)^{0.3} = 1.25,$$

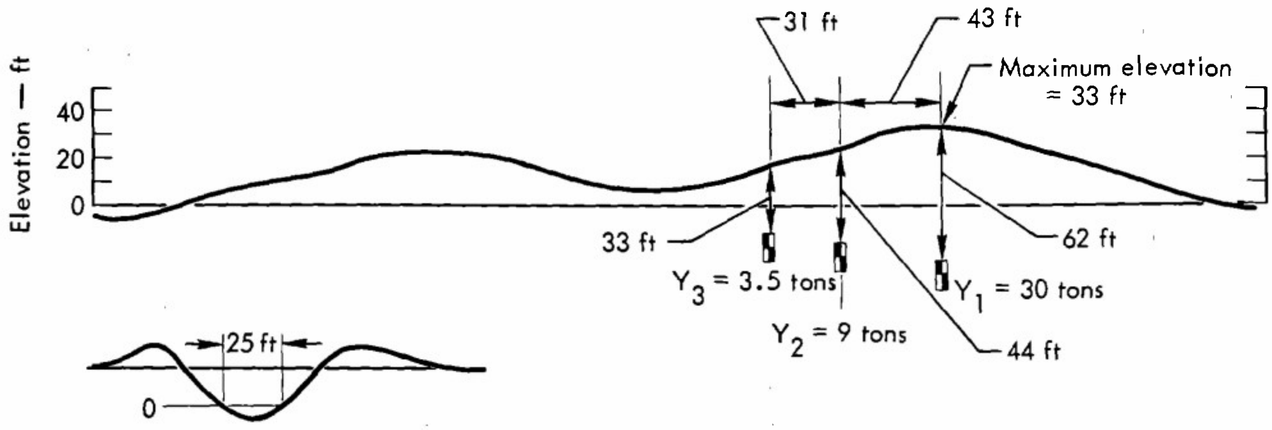


Fig. 23. Row-charge crater with navigation prism through varying terrain.

and Eq. (16) gives the required charge spacing:

$$S/R_a = 1.4\left(\frac{Y_r}{Y_s}\right)^{0.6} = 1.4\left(\frac{30}{65}\right)^{0.6} = 0.9.$$

The 30-ton charge will be buried at the depth for a 65-ton charge, which, from Fig. 17 is 62 ft. The appropriate charge spacing for Y_1 is S_1, and is computed as:

$$S_1 = 0.9 \times 56 = 50 \text{ ft},$$

where 56 ft is the crater radius for a 30-ton charge.

Proceeding to Y_2, the next charge to the left of Y_1, the elevation S_1 ft to the left of Y_1 is determined to be 22 ft. The single charge weight for Y_2 is computed by Eq. (17):

$$Y_2 = \left[0.72\left(\frac{22}{10}\right) + \sqrt{0.52\left(\frac{484}{100}\right) + \frac{625}{4(400)} - \frac{0.72}{1.64}\left(\frac{484}{100}\right)}\right]^{3.33}$$

$$= 20 \text{ tons}.$$

This charge weight is reduced in the same proportion that Y_1 was reduced so that now:

$$Y_2 = 20\left(\frac{30}{65}\right) \simeq 9 \text{ tons}.$$

The 9-ton charge is buried at the depth for a 20-ton charge, which is 44 ft. The appropriate spacing for Y_2 is S_2 and is computed as

$$S_2 = 0.9 \times 39 = 35 \text{ ft},$$

where 39 ft is the crater radius for a 9-ton charge. The actual spacing between Y_1 and Y_2 is:

$$S_{1-2} = \frac{S_1 + S_2}{2}$$

$$= \frac{50 + 35}{2} \simeq 43 \text{ ft}.$$

Continuing to the third charge, Y_3, the elevation, a distance of S_2 (35 ft) to the left of Y_2 is determined to be 15 ft, so that:

$$Y_3 = \left[0.72\left(\frac{15}{10}\right) + \sqrt{0.52\left(\frac{225}{100}\right) + \frac{625}{4(400)} - \frac{0.72}{1.64}\left(\frac{225}{100}\right)}\right]^{3.33}$$

$$= 7.6 \text{ tons.}$$

The reduced weight will be:

$$Y_3 = 7.6\left(\frac{30}{65}\right) \simeq 3.5 \text{ tons.}$$

The depth of burial will be 33 ft and the spacing for Y_3 is:

$$S_3 = 0.9 \times 29 = 26 \text{ ft.}$$

The spacing between Y_2 and Y_3 is then:

$$S_{2-3} = \frac{S_2 + S_3}{2} = \frac{35 + 26}{2} \simeq 31 \text{ ft.}$$

This procedure is continued until the excavation design is complete.

Often it may be advisable to excavate the canal or channel by interconnecting row-charge craters. A connecting row-charge crater may use a different charge spacing, and often it will be advantageous to increase the charge spacing as the depth of cut decreases because fewer charges will be required. The connection of row craters is covered in Section 5.5.

(2) Constant-Charge-Weight Method

Now the alternate procedure of constant-charge-weight row-charge design will be discussed. In this method the relative spacing between charges, S/R_a, is varied rather than the weight of the charges.

Taking the case of a cut with a specified bottom elevation first, the initial step is to select the charge weight which is to be used throughout the detonation. The spacings between the remaining charges are determined by means of the appropriate crater dimension chart and Eq. (16). The method can be illustrated by using the profile and channel depth shown in Fig. 24 as an example.

Assume that 3-ton charges will be used. The deepest cut is 25 ft and Fig. 17 shows that a single-charge weight of 21 tons is required; therefore the charge spacing is given by:

$$S/R_a = 1.4\left(\frac{Y_r}{Y_s}\right)^{0.6} = 1.4\left(\frac{3}{21}\right)^{0.6} = 0.43,$$

and, because R_a for a 3-ton charge is 28 ft, then

$$S_1 = 0.43 \times 28 = 12 \text{ ft.}$$

The 3-ton charge is buried at the depth for a 21-ton charge, 44 ft. The next step is to look at the depth of cut, S_1 ft from the first charge, and in Fig. 24 it is 23 ft. According to Fig. 17, this depth of cut requires a single-charge weight of 17 tons and a depth of burial of 42 ft. The spacing, S_2, for charge Y_2 is computed as follows:

$$S/R_a = 1.4\left(\frac{3}{17}\right)^{0.6} = 0.50,$$

and therefore

$$S_2 = 0.50 \times 28 = 14 \text{ ft.}$$

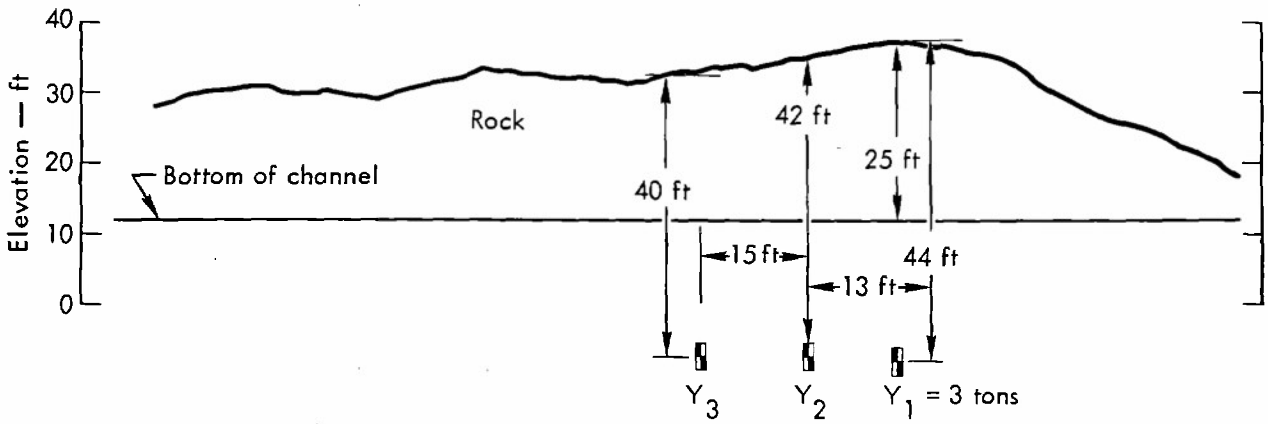


Fig. 24. Example of cut with constant bottom elevation and constant charge weights.

As with the constant-enhancement method, the actual spacing between Y_1 and Y_2 is the average of S_1 and S_2:

$$S_{1-2} = \frac{S_1 + S_2}{2} = \frac{12 + 14}{2} = 13 \text{ ft.}$$

To compute the spacing between Y_2 and Y_3, look at the depth of cut S_2 ft from Y_2. This depth in Fig. 24 is 21 ft, and so a single-charge weight of 14 tons and a DOB of 40 ft are required. S_3 is computed:

$$S/R_a = 1.4\left(\frac{3}{14}\right)^{0.6} = 0.56;$$

therefore

$$S_3 = 0.56 \times 28 = 16 \text{ ft,}$$

and

$$S_{2-3} = \frac{S_2 + S_3}{2} = \frac{14 + 16}{2} = 15 \text{ ft.}$$

The process is continued in this manner until the end of the cut. In some instances it may occur that the spacing (S/R_a) computed for a charge is greater than 1.4. Because this is the upper limit of row charge spacing, it will be necessary to select a lower yield for the remaining charges such that S/R_a is everywhere less than 1.4. Otherwise the procedure as described above has no exceptions.

The procedure for designing a constant charge weight row to excavate a crater for a specific navigation prism is similar to the above. Individual single-charge weights are computed by Eq. 17 rather than by obtaining them from a crater dimension chart, but the spacings are computed in the same manner as the above example.

Appendix E discusses design procedures in more detail and introduces ways in which the time required to perform a large number of repetitive computations may be reduced.

The use of enhancement to reduce individual row-charge weights is subject to an important constraint. As smaller charges are spaced closer together, the number of charges detonated at one time must be increased in order to avoid a short, elliptical-shaped crater. As a general rule, each row-charge detonation should result in a crater whose length is at least twice its width. The following relationship gives the number of charges in a row which should be detonated to maintain this minimum ratio:

$$N > \frac{4}{S/R_a}, \tag{21}$$

where N is the number of charges in a row.

5.4 PARALLEL ROWS OF CHARGES

The use of multiple rows will be warranted whenever a broad, shallow excavation is required and where a single row of explosives, of sufficient size to achieve the desired width, would result in unnecessary overexcavation of depth. At the current level of development of explosive excavation technology, the only experimentally verified alternative to a single row of charges is a set of two parallel rows, the two rows being detonated with a small time delay between them.[9]

The design procedure for two parallel rows is described below, and reference is made to Fig. 25 which shows a schematic cross section of the crater produced by the configuration. As shown in Fig. 25, the separation between rows should be 1.5 times the half-width of a crater produced by a single row of charges, and the width of the channel (W_c) at the preshot ground surface will be 3.5 times $W_a/2$. The bottom of the crater between the rows will be relatively flat with perhaps a slight mound of fallback.

The single charge weight, Y_s (i.e., unenhanced row-charge weight), which will produce the required width is determined as follows:

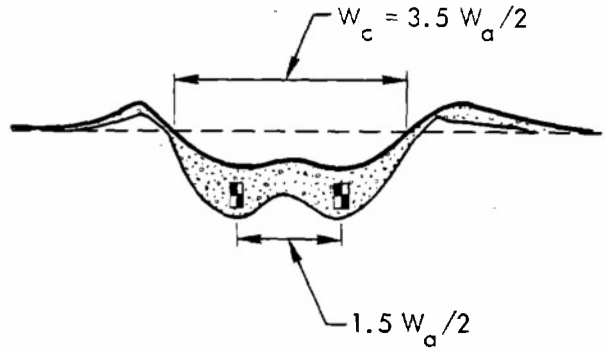


Fig. 25. Schematic cross section of crater produced by two parallel rows of charges.

$$W_a/2 = R_{as} = \frac{W_c}{3.5}, \tag{22}$$

where R_{as} is the crater radius corresponding to the single-charge weight of Y_s. Knowing R_{as}, Y_s can be read directly from the crater dimension charts. To use a smaller charge, Y_r, rather than Y_s, Eq. (16) gives the required charge spacing as:

$$\frac{S}{R_{ar}} = 1.4\left(\frac{Y_r}{Y_s}\right)^{0.6},$$

where R_{ar} is the crater radius corresponding to Y_r. The rows are separated by 1.5 R_{as} and the charges are buried at the optimum depth for Y_s.

The time delay between the rows is given by:

$$T_D \simeq 150\ Y^{1/3}\ \text{(msec)}, \tag{23}$$

where Y is the weight (in tons) of an individual charge in a row.

For example, assume it is necessary to cut a channel 100 ft wide and 10 ft deep through soil. From Fig. 18, it is found that a single row of 10-ton charges spaced 70 ft apart would meet the width objective, but the maximum depth of the channel would be almost 25 ft, a considerable overexcavation.

A design for the required excavation with a double row of charges would be performed as follows:

From Eq. (19):

$$W_a/2 = R_{as} = \frac{W_c}{3.5} = \frac{100}{3.5} \simeq 28.5 \text{ ft},$$

and therefore Y_s = 1.5 tons from Fig. 18. A double row of 1.5-ton charges, each with a charge spacing of 40 ft (= 1.4 × 28.5) and separated by 43 ft (= 1.5 × 28.5) would excavate the required channel with a maximum depth of approximately 13 ft.

The time delay between the rows is given by Eq. (23):

$$T_D \simeq 150\ Y^{1/3}\ (\text{msec}) = 172 \text{ msec}.$$

The nearest commercially available delay cap is 175 msec, and this would be acceptable.

In the example above, the single row would require 10 tons for 70 ft of channel or 0.14 tons/ft; the double row requires only 3 tons/40 ft, or 0.075 tons/ft of channel, a reduction of almost a factor of 2.

Convenient charts and suggested step-by-step procedures for the design of double-row-charge excavations are developed and presented in Appendix E.

5.5 INTERCONNECTING CRATERS

After a row of charges has been designed to excavate a channel, other considerations such as safety may limit the total weight that can be detonated at one time. It may be necessary to detonate the charges as a series of separate row charges rather than all at once. It is, therefore, desirable to be able to smoothly connect one row crater to another. The designer should be aware of this requirement ahead of time so that he can establish the limits of each detonation and provide for the connection of the row craters. All charges in a long row may sometimes be emplaced before firing any portion of the row, in which case provision for connections must be made prior to emplacement.

At the point of connection between two row craters, the distance between the end charges of the two rows should be the optimum crater radius of the larger of the two charges. This distance is obtained directly from the crater dimension charts. In the event that only one row of charges is emplaced at a time, then the end charge of a follow-on connecting row should be placed beneath the end lip crest of the existing row crater. No adjustment to the charge weight is required to compensate for the lip material.

The technique of connecting one row to another is not well-developed. The connection of row craters may result in a low mound of material being deposited in the first crater when the second one is formed. Remedial excavation with conventional methods will be necessary if the mound obstructs the channel.

5.6 DELAYED ROW-CHARGE DETONATIONS

Some applications of explosive excavation may appear to be not feasible because of the proximity of buildings or other structures and the consequent risk of ground-shock-induced damage. Should a first analysis indicate that a project is not feasible for this reason, then consideration should be given to excavating by means of delayed charge firings. A limited amount of experimental data have been acquired on relatively large charges (1-ton) emplaced in rows and fired in some ordered sequence. These data indicate that marked reductions of seismic motion are possible.[9]

For sequential detonations, a time delay of the order of milliseconds is introduced between charge detonations in the row. The detonation sequence begins at one end and progresses toward the other. Delay intervals can be achieved by using commercially available delay caps, or delay connectors (see Section 4.3). The delay interval between charge detonations is computed according to the following equation:

$$T_D = 25\ Y^{1/3}\ (\mathrm{msec}), \tag{24}$$

where Y is the individual charge weight in tons, or the average charge weight in a row consisting of a mixture of charge sizes.

Delayed firings can be expected to result in a reduction of crater depth compared to a corresponding simultaneously detonated row. Widths are apparently not affected. This depth reduction can be compensated for in the design by increasing the charge weights 15% above those required for simultaneous firing. The ground shock from a delayed row can be predicted on the basis of the largest charge in the row, rather than the total weight of the explosive used in the row.

5.7 UNDERWATER CRATERING

Experience in underwater cratering is limited and considerable experimentation remains before quantitative design criteria can be fully established. For the present, the design procedures outlined for dry land cratering are assumed applicable; however, the water overburden must be accounted for. Because water is much less dense than earth materials and possesses no shear strength, it requires less energy to displace than an equal volume of rock or soil. When designing cratering detonations under water, the water overburden should be treated as an added layer of bottom material that is one-half as thick as the depth of water. This means that all crater depths and burial depths should be referenced to a hypothetical surface one-half the depth of water below the water surface.

The presence of water may drastically alter the process of crater formation because ejecta may be washed back into the crater. The extent of this washback will depend on the water depth and the material being cratered. Ejecta throwout ranges will also be less when water is present. Underwater craters will generally tend to be shallower and wider than a crater on dry land. It is strongly recommended that extensive experimentation be carried out prior to designing any large-scale underwater cratering project.

5.8 CHARGE GEOMETRIES AND STEMMING

Thus far, the charge has been implicitly assumed to act as a point source of energy. Many chemical explosive cratering experiments have been conducted using spherical, center-detonated charges. There is evidence, however, that although a spherical charge is the most desirable shape, it is not a requirement for successful results. The degradation of cratering efficiency caused by minor departures from the spherical shape are not significant. The most recent experiments have used cylindrical-shaped charges. Cylindrical charges have been tested in cratering experiments with length-to-diameter ratios of almost ten to one and the evidence suggests that ratios of four or five to one can be used without having to increase the weight in compensation. When the height-to-diameter ratio of the charge is four or more, the effective center of the charge is currently assumed to be at a point one third of the charge length from the bottom, and depths of burial should be referred to this point.

All cratering charges should be stemmed to prevent loss of energy from premature venting through the emplacement hole. The emplacement hole should be filled with stemming material all the way to the surface. Stemming materials can be sand, gravel, or soil. If the explosive is a water-resistant slurry, the stemming material should be saturated with water prior to detonation. Water alone can be used for stemming if no other material is available.

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Chapter 6

Safety and Environmental Considerations

6.1 SCOPE

This chapter discusses the potential hazards from ground motion, airblast, underwater shock, water waves, and missiles which can result from the detonation of large chemical explosive charges in construction operations. A discussion of the effects of explosive excavation operations on the local environment and ecology is included to assist in the preparation of environmental statements as required by the National Environmental Policy Act of 1969. A brief summary of the safety considerations for each potentially hazardous effect is provided, and procedures are given for estimating the effect's magnitude and damage potential. The emphasis is on public safety considerations and environmental preservation. The safety aspects of operational matters, such as explosives handling and loading and area security, are discussed in Chapter 7 and Corps of Engineers' Manual EM 385-1-1.

6.2 GROUND MOTION

a. General Safety Considerations

Ground motion, also called ground shock or seismic motion, will accompany all surface and subsurface detonations and may cause structures on and beyond the construction site to move or vibrate. These motions, depending on their magnitude, may cause architectural damage (cracking of finished surfaces, plaster, stucco, and breaking of windows) or even structural damage. Additionally, the perception of these ground motions may cause uninformed persons to fear for their safety and to submit unwarranted claims for property damage.

The amount of explosive energy coupled to the medium and transmitted as seismic energy depends on the conditions of emplacement, depth of burial, and type of surrounding medium. The coupling effect or seismic efficiency becomes greater as voids surrounding the explosives are reduced, burial is deeper, and the medium is more competent (rock as opposed to soil). Typically less than 1% of the total explosive energy is converted into seismic energy.[1]

The most influential factor governing the magnitude of the seismic signal received at any location is the geology at that location. Current data indicate that at large distances from the detonation point, the surface motion at a site located on soil can sometimes be five times as high as the surface motion at a site located on rock with other parameters being equal.

b. Ground Motion Prediction

Particle velocity and acceleration are the ground motion parameters of interest and are also the parameters which most seismic instruments are designed to measure. Particle acceleration has been shown to have fairly consistent values at which given levels of damage occur for residential structures. No similar correlation with damage to engineered structures such as high-rise buildings has been developed to date. Analysis of potential damage to engineered structures requires the expertise of a structural engineer. The frequency of the ground motion is also important in determining the response of residential and high-rise or engineered structures, but the capability does not exist to predict reliably the frequency of peak ground motions.

The prediction of ground motion is based on empirical equations of the following form[2]:

$$GM = CY^{m}R^{-n}, \tag{25}$$

where

GM = amplitude of peak particle motion (velocity or acceleration)

Y = weight of detonated explosives

R = range

C, m, and n = constants found experimentally

Equations with appropriate constants for predicting peak amplitudes of ground motion for a variety of detonation media and site conditions are listed in Table 11. The equations in Table 11 are plotted in Figs. 26 and 27 for explosive detonations

Table 11. Ground motion prediction equations.[2]

Detonation medium	Geologic environment at point of interest	Prediction equation[a] (R in ft; Y in tons of TNT)
High strength rock	Rock	$A = 4.5 \times 10^4\ Y^{0.70}\ R^{-2.00}$
		$V = 20.0 \times 10^4\ Y^{0.73}\ R^{-1.87}$
	Soil	$A = 9.7 \times 10^4\ Y^{0.70}\ R^{-2.00}$
		$V = 67.0 \times 10^4\ Y^{0.73}\ R^{-1.87}$
Weak rock	Rock	$A = 1.8 \times 10^4\ Y^{0.70}\ R^{-2.00}$
		$V = 8.3 \times 10^4\ Y^{0.73}\ R^{-1.87}$
	Soil	$A = 4.2 \times 10^4\ Y^{0.70}\ R^{-2.00}$
		$V = 28.0 \times 10^4\ Y^{0.73}\ R^{-1.87}$
Soil	Rock	$A = 0.8 \times 10^4\ Y^{0.70}\ R^{-2.00}$
		$V = 3.3 \times 10^4\ Y^{0.73}\ R^{-1.87}$
	Soil	$A = 1.6 \times 10^4\ Y^{0.70}\ R^{-2.00}$
		$V = 11.0 \times 10^4\ Y^{0.73}\ R^{-1.87}$

[a]A = Peak particle acceleration in g
V = Peak particle velocity in in./sec

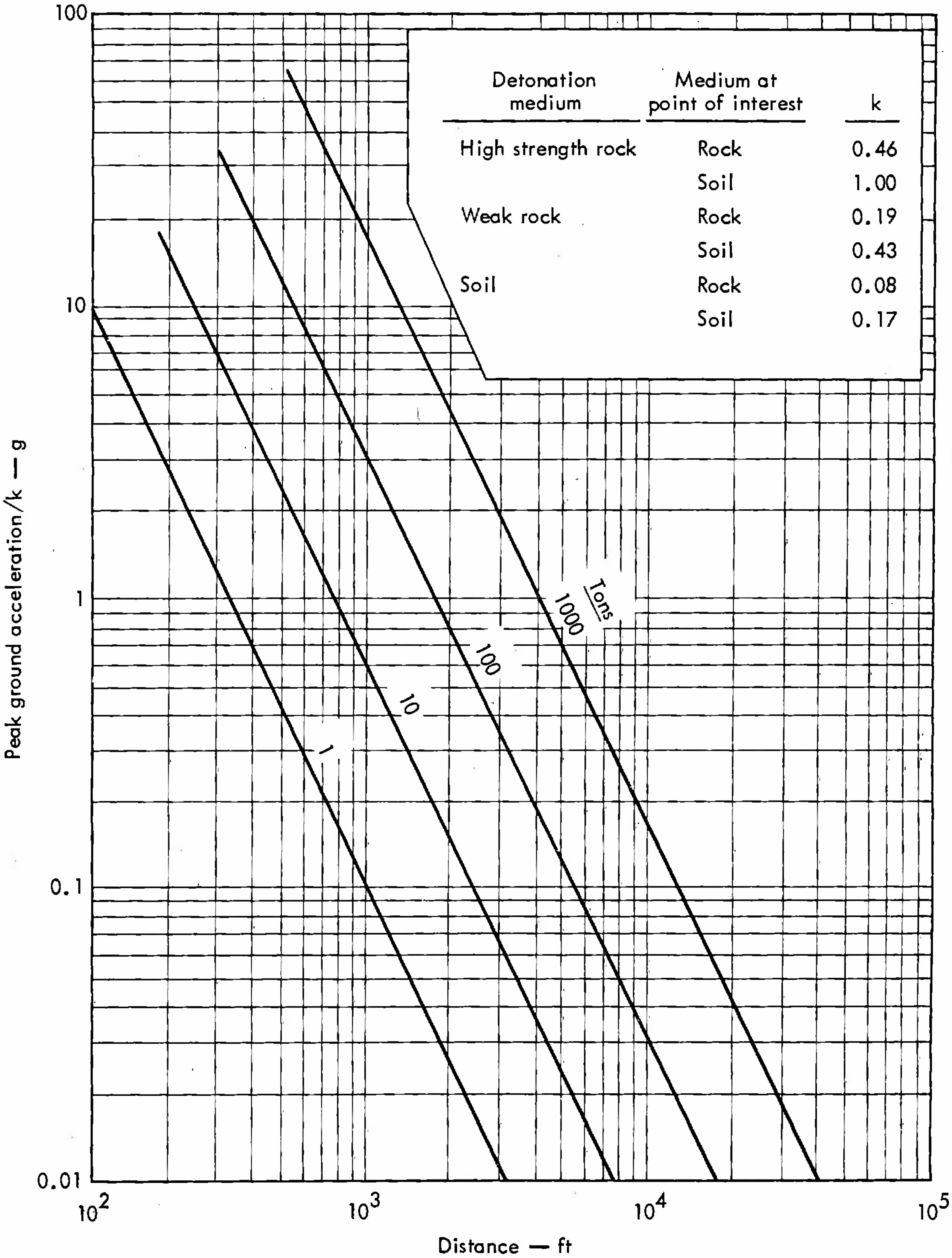

Fig. 26. Ground acceleration predictions.

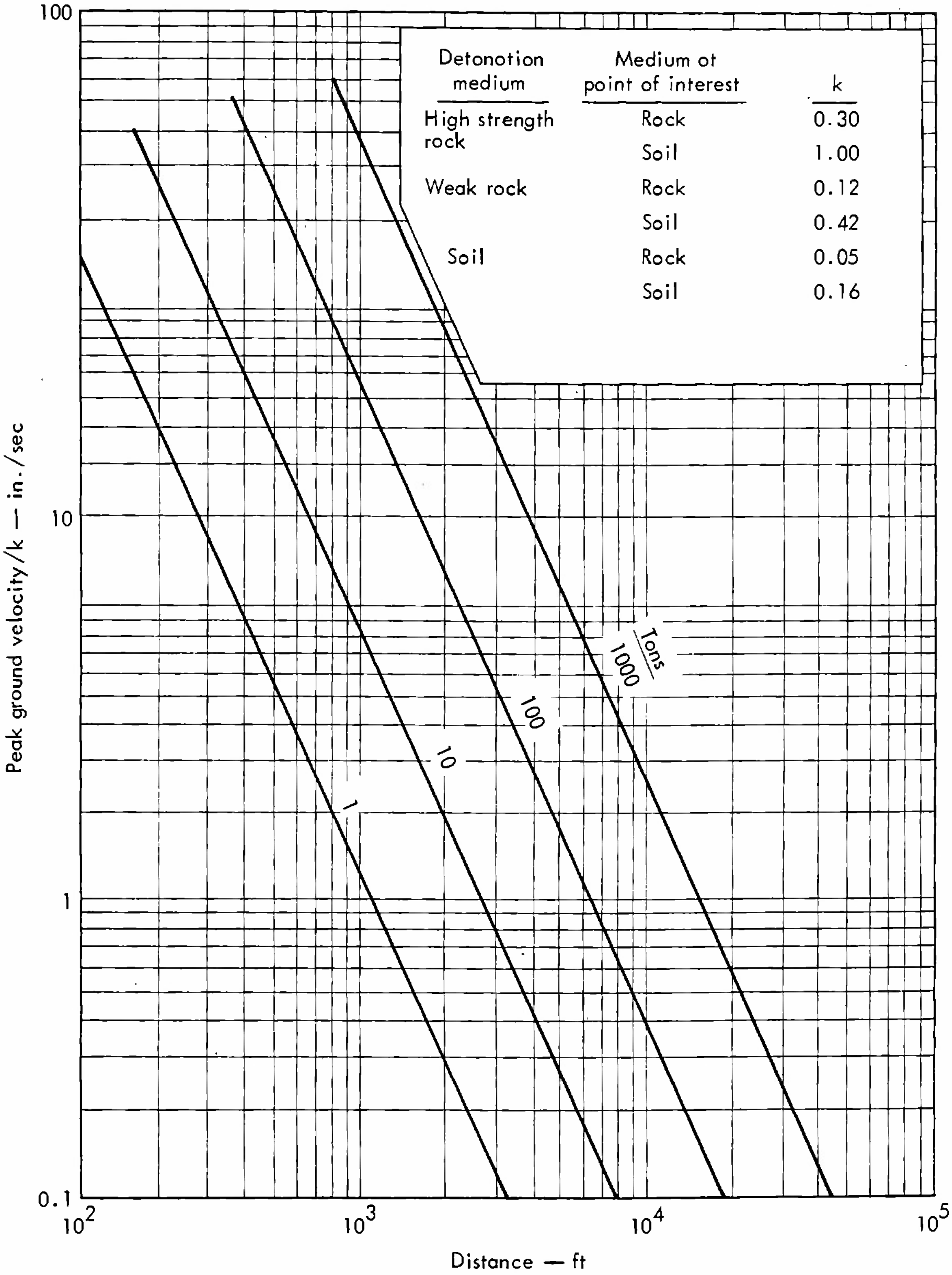

Fig. 27. Ground velocity predictions.

of 1 to 1000 tons of TNT* for ranges of concern. For multiple charges, the total weight of explosives to be instantaneously detonated is used as the value of Y in the prediction equations.[3]

To find the acceleration or velocity at a range of interest, first locate the range of interest on the abscissa, then project a line upward to the appropriate explosive weight line, and finally read the corresponding ordinate value. This ordinate value, when multiplied by the "k" factor for the situation of interest (detonation site medium and medium at point of interest), will give the predicted acceleration or velocity.

c. Calibration Tests

If critical residential, high-rise, or other engineered structures are located within areas where structural damage is possible, calibration tests may be desirable. In these calibration tests, small explosive charges are detonated to determine by actually measuring acceleration and velocity the seismic transmission properties of the area and building structural response characteristics. Previous experience has shown that a value of 0.67 to 0.73 for the weight scaling exponent, m, in the prediction equations is fairly reliable for all sites. However, the proportionality constant, C, and the range exponent, n, can vary greatly with charge size and from site to site depending on transmission properties. Therefore, using site data to generate curves similar to Figs. 26 and 27 would give a value for n, which is equal to the slope of the curve and is developed for the local geology. With m and n known, C can be determined and the equation thus obtained will give a more reliable prediction of ground motions to be expected from detonations at the site in question.

d. Damage Criteria

The considerable amount of information which is available regarding ground shock and its damaging effects on structures shows conclusively that an absolute threshold of damage cannot be defined. The severity of ground shock-induced damage to any structure will depend as much on the type and condition of the structure as on the level of ground motion to which it is subjected. Damage levels for various structures and equipment are presented below. They are based on average structures in an average state of repair. Application of the limits stated to structures other than those indicated should be made with extreme caution and by persons experienced in analyzing structural stability problems.

(1) Residential Structures

Ground acceleration is the best parameter for use in predicting damage levels to residential structures, although in the past considerable data relating levels of damage to ground velocity have been accumulated. Damage to residential structures associated with various accelerations expressed in gravity units is given in Table 12.

(2) Miscellaneous Structures and Equipment

The available data pertaining to damage from ground motion sustained by structures and equipment associated with

*When the explosive being used is not TNT, its equivalent weight of TNT should be used for Y (see Section 4.5).

Table 12. Residential structure damage criteria.[4]

Acceleration (g)	Expected damage level
Up to 0.02	No damage
0.02-0.10	Possible architectural damage usually of a minor nature
0.10-0.30	Probable architectural damage
Over 0.30	Probable structural damage

construction operations are presented in Table 13.

(3) <u>High-Rise Buildings and Engineered Structures</u>

The prediction of ground motion-induced damage for high-rise buildings and other engineered structures involves more uncertainties than are encountered in predictions for residential buildings. These structures are susceptible to damage from the low frequency portion of the

Table 13. Effect of ground velocity on structures and equipment.[a]

Effect on structure or equipment	Degree of damage: Threshold (in./sec)	Moderate (in./sec)	Severe (in./sec)
Rigid frame buildings:			
Structural damage	60	—	—
Small plywood buildings:			
Structural damage	60	—	—
Skid-mounted engines not tied to ground:			
Failure of suspension system	—	20 to 40	—
Cracks in cast metal parts	—	20 to 40	—
Skid-mounted engines firmly tied to ground:			
Failure of suspension system	40	—	—
Cracks in cast metal parts	40	—	—
Steel storage tanks of light construction:			
Rupture	—	—	30
Steel storage tanks specifically designed and built to withstand ground motion:			
Distortion	200	—	—
Wheeled trailers on styrofoam pads:			
Suspension and frame damage	120	—	—
Wheeled trailers with heavy loads:			
Structural damage to suspension and frame	—	—	40
Building equipment:			
Loose objects thrown about (office machines, hand tools, etc.)	12	—	—

[a]Adapted from Ref. 5.

seismic waves, particularly if they approach the natural frequency of the structure. Since frequency of ground motion tends to decrease with range, high-rise buildings and other large engineered structures could be susceptible to damage at greater range than are residential structures. The frequency of ground motion tends to be lower in soil than on rock, and this fact may increase the damage range for high-rise buildings and engineered structures situated on soil. High-rise and engineered structures within the predicted range for 0.001 g ground motion should be analyzed for structural response characteristics.[6]

Several techniques for analyzing the response of engineered structures are available. Selection of the applicable techniques to be used in analyzing a structure is the prerogative of the structural engineer. The analysis techniques, known as the response spectrum method, time-history response method, and spectral matrix method, are described in detail in Refs. 7 and 8.

(4) Area Features

Man-made structures, such as tunnels, dikes, steep slopes along highway and railway cuts, etc., and natural features, such as steep embankments and rock formations, may also be susceptible to damage or movement as a result of ground motions. The variety of features that may exist precludes a statement of any damage criteria. Each feature of concern should be evaluated. Where evaluation indicates damage is probable, necessary damage prevention measures and safety precautions should be placed in effect at the time of detonation. These measures may include temporary bracing, evacuation of personnel and animals from dangerous areas, and postshot evaluation of structures.

e. Human Perception and Injury

The mere perception of ground motion by some people residing in the area around a detonation point may prompt them to file damage claims. Others may fear for their personal safety. For comparison with other damage criteria, the approximate threshold of perception, which varies with individuals and with the frequency of the motion, is an acceleration of 0.002 g or a velocity of 0.08 in/sec (Ref. 2).

Injury to persons as a result of seismic motions from high explosive detonations could be caused by displaced objects (e.g., falling plaster, items displaced from shelves, loose chimney sections, decorative stone from building facings). In areas where the ground acceleration is predicted to be more than 0.02 g, the physical condition of all structures should be examined to determine the requirements for evacuation to prevent injury to persons. At predicted accelerations of 0.1 g or more, persons should be evacuated from all structures unless analysis by a structural engineer indicates no personal injury can occur in the structures. If evacuation is considered necessary during a detonation, personnel should move away from a structure a distance equal to twice the height of the structure.

f. Determination of Areas of Concern

The following procedure may be used to identify the areas in which damage may occur to structures and area features:

(1) Using topographic and geologic maps of the area in the vicinity of the proposed detonation and Fig. 26 (or the ground motion prediction equations), determine the ranges at which various acceleration values will be experienced. Ranges at which the predicted accelerations are 0.3 g, 0.1 g, etc., down to 0.001 g should be determined.

(2) Plot these values on a map of the site and surrounding area; label the various acceleration contours.

(3) Determine the types of structures and area features within the various acceleration contours.

(4) Using the damage criteria contained in this chapter, evaluate the expected effects of the proposed detonation.

(5) When it is determined that the excavation project is otherwise feasible but residences, engineered structures, or area features are found to be located within areas of possible or probable damage, an on-site survey should be conducted to determine the types of structures and features that exist, their physical condition and any local geologic or other condition which might reduce or enhance the ground motion damage potential. The nature of this survey is discussed in the next section.

6.3 STRUCTURES SURVEY

a. Predetonation Survey

Predetonation surveys are conducted to: (1) estimate the total structure exposure, (2) identify special hazards and structures requiring specific analysis, (3) document the predetonation conditions of structures and area features, (4) determine the required safety measures, and (5) make damage cost estimates.* This survey should be conducted by personnel experienced in evaluating the ground motion response and physical condition of structures and features. As each structure or feature is inspected, its predetonation condition (cracked plaster, cracks in exterior finish, broken windows, foundation settling, etc.) should be documented and supported with photographic evidence and measurements of crack widths and lengths, amount of settlement, etc. The need for preventive measures, such as bracing, chimney removal, or padding, required to prevent or minimize additional damage should be determined.

Personnel conducting the survey should determine whether structures or articles of historial, religous, or antique value are present in the area. Since these items are not replaceable and often cannot be evaluated in monetary terms, special measures of protection or removal to a safe storage area may be required.

b. Postdetonation Survey

A postdetonation survey should be conducted as soon as possible to document the condition of all area structures, items, and features identified as susceptible to damage. As with the predetonation survey, the documentation should be comprehensive and prepared in a manner that will be usable for judging damage claims

*Damage costs are discussed in Chapter 9.

and supporting legal actions. Postdetonation survey data will also assist the planner in making future predictions for the area or similar areas.

6.4 AIRBLAST

a. Safety Considerations

For underground detonations at cratering depth, damage due to ground motion and missiles will usually be dominant, and safety concern from airblast is generally limited to ranges where broken windows could cause personal injury. The range at which airblast effects may cause damage or injury is a function of the charge size, number of charges, burial depth, and atmospheric conditions at detonation time.

b. Airblast Generated by Cratering Detonations

After the initial shock front from an underground explosion strikes the ground-air interface, a wave disturbance in the air is generated by the sudden mound growth of the displaced earth as shown in Fig. 3(c). The wave then propagates through the atmosphere at a velocity determined by meteorological conditions. This portion of the air overpressure wave is termed the "ground-shock-induced airblast."

As the gas bubble generated by the underground explosion expands toward the ground-air interface, it transfers additional velocity to the overlying soil and rock. The specific pressure of this gas bubble when it "vents" will determine the magnitude of the second part of the air overpressure wave, which is called the "gas-vent airblast pulse." This occurs at the time of venting as depicted in Fig. 3(d). If the pressure of the gas produced by the explosion is equal to or less than atmospheric when it finally breaks through the rising mound of earth, there will be no observable gas-vent airblast pulse. This condition is dependent on the depth of burial of the charge, the explosive used, and the specific properties of the detonation medium.

Thus two distinct pressure wave peaks may exist and be measured from an underground explosion. It is the wave with the highest peak overpressure (highest pressure above ambient) that is of interest in the development of airblast prediction procedures.

c. Meteorological Conditions Affecting Airblast Wave Propagation[2]

Airblast waves generated by explosions attenuate very rapidly to low-amplitude pressure waves which approximate sound waves. If a sound ray is defined as the direction of propagation of the sound wave, then the path which a sound ray takes is directly dependent on the changes in sound velocity with elevation (velocity gradient) in the atmosphere. The velocity of sound is in turn dependent on the temperature gradient and direction and speed of winds in the atmosphere. The velocity of sound increases downwind, decreases upwind, and increases with increasing temperature. If the velocity of sound is uniform throughout the air above the ground surface, the sound rays will move out uniformly in all directions in a pattern analogous to the spokes of a wheel. If the sound velocity decreases from the surface upward, all sound rays will be turned upward away from the ground surface, and the airblast overpressure along the surface will decrease very rapidly. This would be a desirable situation from an operational and safety standpoint. On the other hand, if the sound velocity increases with altitude, the rays will be turned toward the ground and possibly intersect or overlap, resulting in a subsequent increase in airblast overpressure at specific locations as compared to that which would result from propagation through a uniform velocity field. This would generally be an undesirable situation because it could cause sound rays to focus at points on the ground, and the resulting amplification of the airblast overpressure could cause property damage, injury to personnel, or an annoyance to populated areas.

d. Prediction of Overpressure Amplitudes

Airblast overpressure amplitudes are usually predicted by scaling the overpressure expected from a 1000-ton explosive charge of TNT detonated in a standard atmosphere with a zero sound velocity gradient.* Several theoretical calculations have been made of the airblast overpressure resulting from a 1000-ton "free" air burst (i.e., no boundary reflections) as a function of range from the point of detonation. The standard overpressure-range curve that is used is designated the "IBM Problem M" curve and is shown in Fig. 28.[9]

Curves for other charge weights (Y) and pressure conditions (P) may be constructed by shifting the Problem M curve up or down according to the relation:

$$\Delta P = \Delta P_m \left(\frac{P}{P_m}\right), \tag{26}$$

and moving the curve right or left according to the relation:

$$R = R_m \left(\frac{YP_m}{Y_m P}\right)^{1/3}, \tag{27}$$

where

P = atmospheric pressure

ΔP = peak overpressure amplitude

R = range

Y = weight of explosive

Subscript m denotes data taken from the IBM Problem M curve (P_m = 1000 mbar)

The curves for explosive weights of 1, 10, and 100 tons detonated in free air at standard pressure conditions are constructed as follows. Applying Eq. (26) for the condition stated ($P = P_m$), no vertical shift is required. Applying Eq. (27), the new R values are: $R_1 = \frac{1}{10} R_m$; $R_{10} = \frac{1}{4.63} R_m$, and $R_{100} = \frac{1}{2.15} R_m$. The new curves are drawn and labeled in Fig. 28.

e. Transmission Factor for Underground Detonations

The parameter used to relate the levels of overpressure for air burst and subsurface detonations is called the airblast transmission factor (TF). The transmission factor is defined as the ratio of peak overpressure amplitude, ΔP, for a subsurface detonation to that expected at the

*When the explosive being used is not TNT, its equivalent weight of TNT should be used for Y. See Section 4.5.

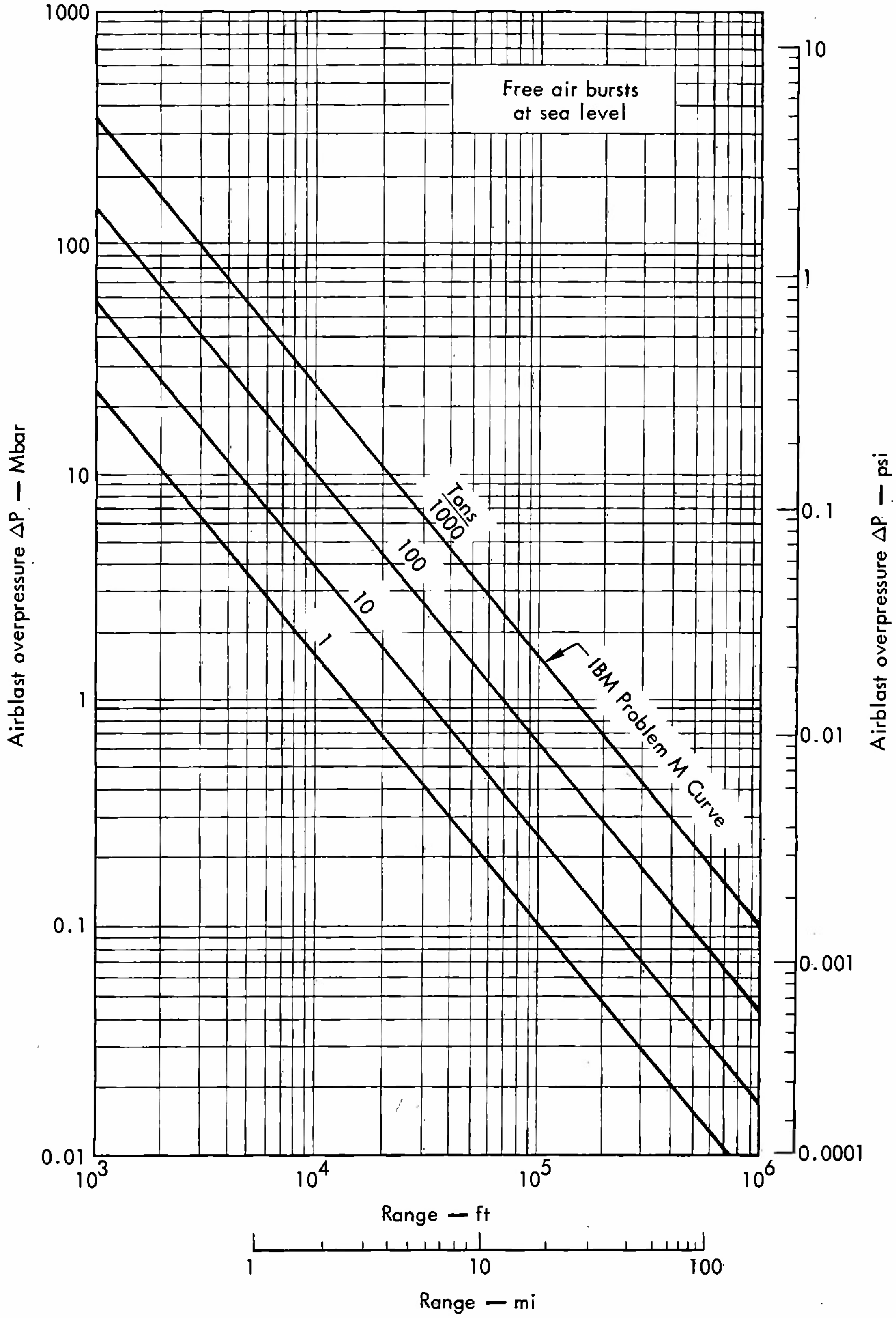

Fig. 28. Airblast overpressures from explosive detonations (adapted from Ref. 10).

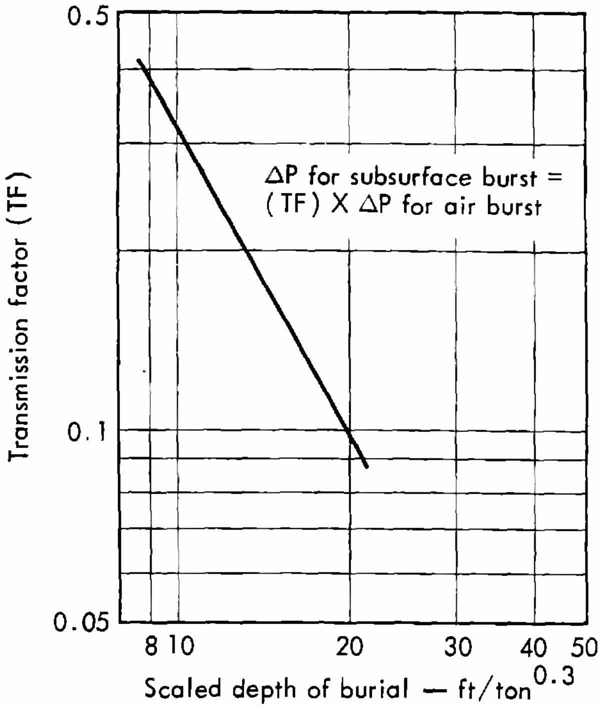

Fig. 29. Airblast transmission factor for underground detonations (adapted from Reed[12]).

For charges in a row, correction factors must be applied to the overpressures propagated perpendicular to the row and off the end of the row (axial direction). Figure 30 is used to determine the appro-

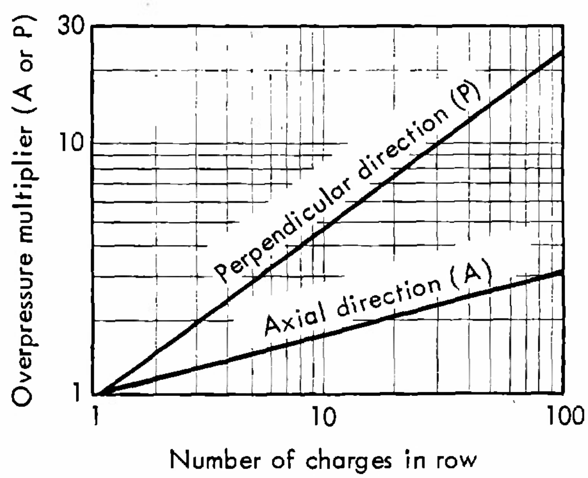

Fig. 30. Airblast multiplier for row of charges (Ref. 9).

same range from the same weight of explosive detonated in free air, or[11]

$$TF = \frac{\text{subsurface burst overpressure}}{\text{free air burst overpressure}}$$

$$= \frac{\Delta P_{\text{subsurface}}}{\Delta P_{\text{airburst}}} . \qquad (28)$$

Figure 29 is a plot of the transmission factor as a function of scaled depth of burial in the region of general interest for explosive excavation applications.[12]

f. Charge Configuration Multipliers

When the configuration of the explosives to be detonated at one time is varied from a single-charge configuration to multiple charges in some geometrical pattern, correction factors must be applied to the overpressure values predicted for a single charge.

priate multiplier to be applied to the overpressure calculated for one of the charges in a row of equal charges, or for one charge of the average charge weight if the charges are of different sizes.[9] The multiplier for a point located at 45 deg to the axis of a row may be determined by taking the axial direction multiplier, A, and adding to it one-half of the perpendicular multiplier, P: thus, $(A + \frac{P}{2})$. See Ref. 13.

For a square array of four equal charges or a square with five equal charges (one at each corner and one in the center), a multiplication factor of 1.7 times the single-charge overpressure has been experimentally determined for charges with a spacing of approximately 25 ft/ton$^{0.3}$ or greater, and 2.8 for charges with closer spacing.[13,14]

g. Adjustment of Predicted Overpressure for Specific Meteorological Conditions[10]

As previously discussed, if the sound velocity at some height above the surface is higher than the velocity at the surface and intermediate levels, a portion of the early blast wave will be bent back or "refracted" into a ground level sound arc, or ring, some distance from the detonation. This refraction phenomenon (also referred to as ducting or focusing) will produce higher airblast overpressures at the ground level than would be predicted by using the curves in Fig. 28. Similarly a decrease in air temperature with altitude will produce lower airblast overpressures at the ground level than would be predicted.

A summary of amplification and reduction factors for overpressures as compared to the standard 1000-ton free air burst curve in Fig. 28 is given in Table 14.

h. Damage Criteria

Injuries to personnel could occur as a result of the direct exposure of personnel to the airblast overpressures but more likely as a result of personnel being struck by debris set in motion by the airblast. The eardrum of a man is the critical organ to be considered in limiting his exposure to air overpressure. The threshold for eardrum damage is 5 psi, with 15 to 20 psi being the value which will produce eardrum rupture in 50% of the population exposed.[16] When personnel are within structures, the incident air overpressure may be amplified by a factor of up to three by reflection from the various surfaces of the structure. This possible amplification factor plus the possibility of injuries from flying glass and other debris should be considered in setting the safe distances from an explosion for personnel in buildings.

Table 15 presents general criteria for estimating the possible extent of damage resulting from a predicted overpressure, ΔP. In using these criteria for damage assessment purposes, cognizance must be taken of the fact that strength specifications for window panes are not uniform. In addition, the ability of a sheet of glass to withstand a given overpressure is dependent on its size. Although the damage

Table 14. Airblast overpressure amplitude correction factors for various atmospheric conditions.[2]

Location relative to detonation or atmospheric condition	Altitude interval of interest (ft)	Correction factor as applied to standard predicted overpressure	Range interval (mi)
Downwind and/or atmospheric temperature inversion	Surface to few thousand[a]	2 to 3	Up to 30
Upwind and/or decrease in air temperature with altitude	All	1/10	≥100[b]

[a]Reference 15 presents a detailed procedure for predicting airblast focusing in this altitude range.

[b]$\Delta P \propto R^{-2}$ inside 100 mi.

Table 15. Airblast damage criteria.[11,17]

Overpressure, ΔP (mbar)	(psi)	Degree of estimated damage
2	0.029	Possible window damage, particularly to large store windows
3	0.044	Some damage to large plate glass windows. can be expected
4.5	0.065	Some damage to average size windows can be expected
13	0.19	Extensive damage to windows; probable damage to average wooden doors
40	0.58	Most small casement windows smashed
Over 40	Over 0.58	Structural damage possible

criteria in Table 15 are empirical in nature and very qualitative, they are useful guides in making airblast safety analyses for chemical explosive excavation projects. The damage criteria are expressed both in terms of millibars (mbar) and psi (1 psi = 69 mbar).

i. Procedure for Predicting Airblast Overpressure Amplitudes

The airblast overpressure to be used for explosive excavation project safety analyses may be predicted by the following procedure:

Step 1

Scale the 1000-ton free air burst curve in Fig. 28 to a curve for the total weight of the detonation and local atmospheric pressure by the method discussed in Section 6.4d using Eqs. (26) and (27):

$$\Delta P = \Delta P_m \left(\frac{P}{P_m}\right)$$

and

$$R = R_m \left(\frac{YP_m}{Y_m P}\right)^{1/3} .$$

If a multiple charge detonation, use the average charge weight of the row or array.

Step 2

Determine the scaled depth of burial of the detonation in ft/ton$^{0.3}$. If a multiple charge detonation, use the average scaled depth of burial of the individual charges in the row or array (calculated on the basis of the individual charge weights).

Step 3

Determine the transmission factor (TF) at this scaled depth of burial using Fig. 29 and apply this factor, at specific ranges of interest, to the curve generated in Step 1 using the expression from Eq. (28):

$$\Delta P_{\text{subsurface burst}} = TF \times \Delta P_{\text{air burst}} .$$

If a multiple charge detonation, apply the appropriate overpressure multiplication factor determined from Fig. 30 for the charge configuration and location of points of interest with respect to the charge.

Step 4

Apply the appropriate factors from Table 14 to this curve to estimate maximum, minimum, and average overpressure that may occur at ranges of interest.

Step 5

Locate overpressures on the curve at which damage criteria apply. Locate houses and structures of interest on the range scale.

As an example, assume that five 10-ton charges, each buried at a scaled depth of burial of 20 ft/ton$^{0.3}$, are to be detonated simultaneously to form a row crater at sea level. Since the overpressure vs distance for a single 10-ton detonation in free air is already plotted in Fig. 28, values of overpressure at any range can be read directly from this curve. Overpressure values at 1000 and 80,000 ft would be 57 and 0.3 mbar, respectively. This completes Step 1. Since the scaled depth of burial is given, Step 2 is also complete. From Fig. 29, the transmission factor for a depth of burial of 20 ft/ton$^{0.3}$ is 0.1. Thus, the overpressure values at 1000 and 80,000 ft for the 10-ton charge at a scaled depth of burial of 20 ft/ton$^{0.3}$ are determined to be 5.7 and 0.03 mbar, respectively. The overpressure multiplication factors can be obtained from Fig. 30. The factors for five charges are 3.0 in the perpendicular direction and 1.5 in the axial direction. Thus, the overpressure in the perpendicular direction will be 17.1 mbar at 1000 ft and 0.09 mbar at 80,000 ft; in the axial direction, 8.55 mbar at 1000 ft and 0.045 mbar at 80,000 ft. This operation completes Step 3. These values, the appropriate correction factors for atmospheric conditions, and the structure locations are plotted, and Steps 4 and 5 are now complete. Figure 31 is the plot of overpressure vs range for the example. Hypothetical structures of concern are located along the abscissa and damage criteria along the ordinate.

Figure 31 shows that the predicted overpressure at the nearby industrial buildings under zero sound velocity gradient conditions is 1.4 mbar (for buildings located perpendicular to the row) or 0.7 mbar (for buildings located in the axial direction). In either case, no damage is expected. If inversion conditions prevail, much higher overpressures are predicted at the same range. The overpressure perpendicular to the row exceeds 4 mbar (probable damage to average size windows), and in the axial direction it is 2.1 mbar (possible damage to large windows). Therefore, in order to avoid window damage to the industrial buildings, it would be desirable to avoid detonation under inversion conditions.

6.5 UNDERWATER SHOCK

a. Safety Considerations

The shock wave produced by a detonation in water can be lethal to swimmers, divers, or marine life, and can damage underwater structures. The underwater shock from a detonation in a geologic medium overlain by water will be considerably reduced in magnitude compared to

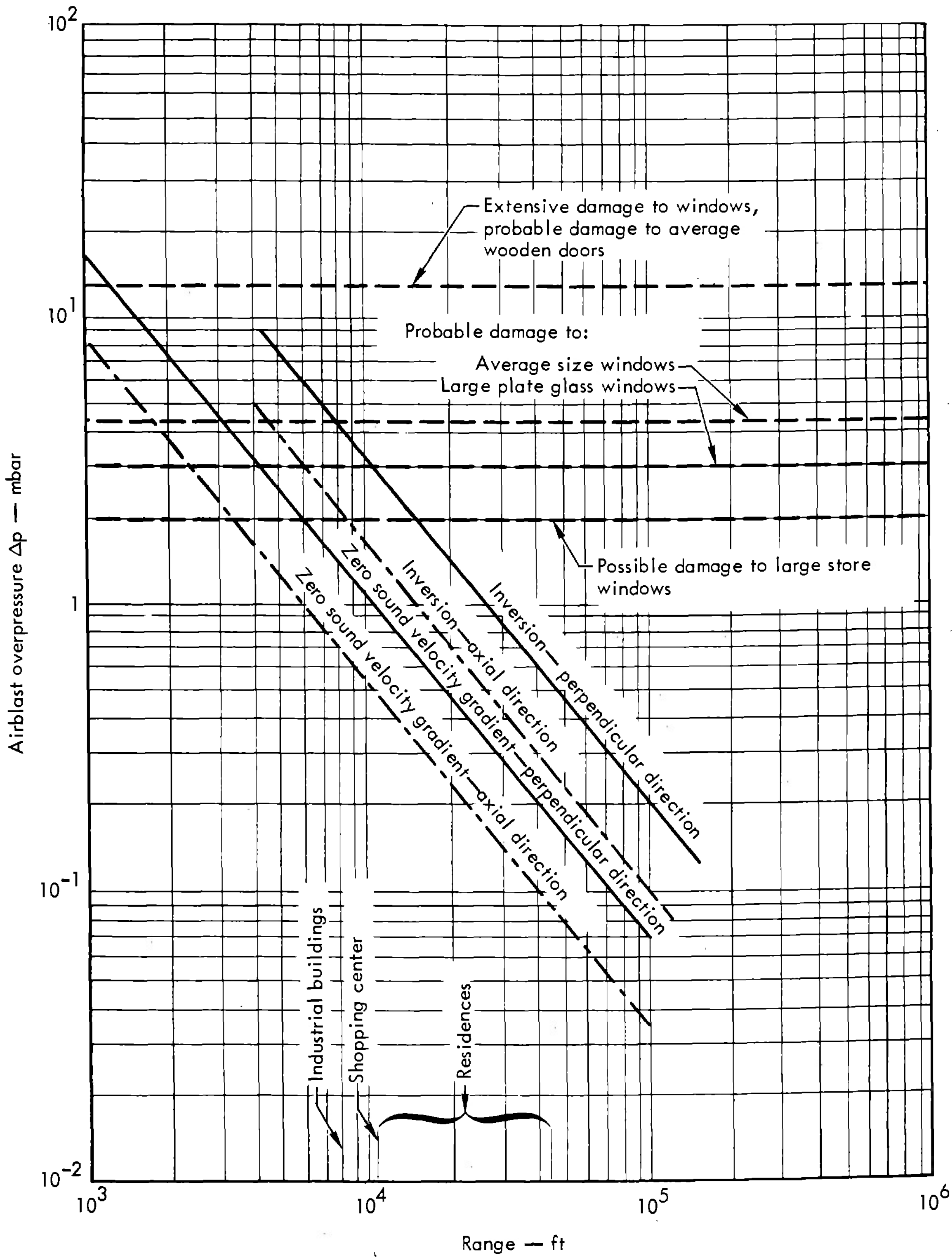

Fig. 31. Example problem: Airblast overpressure from five 10-ton charges in row buried at 20 ft/ton$^{0.3}$.

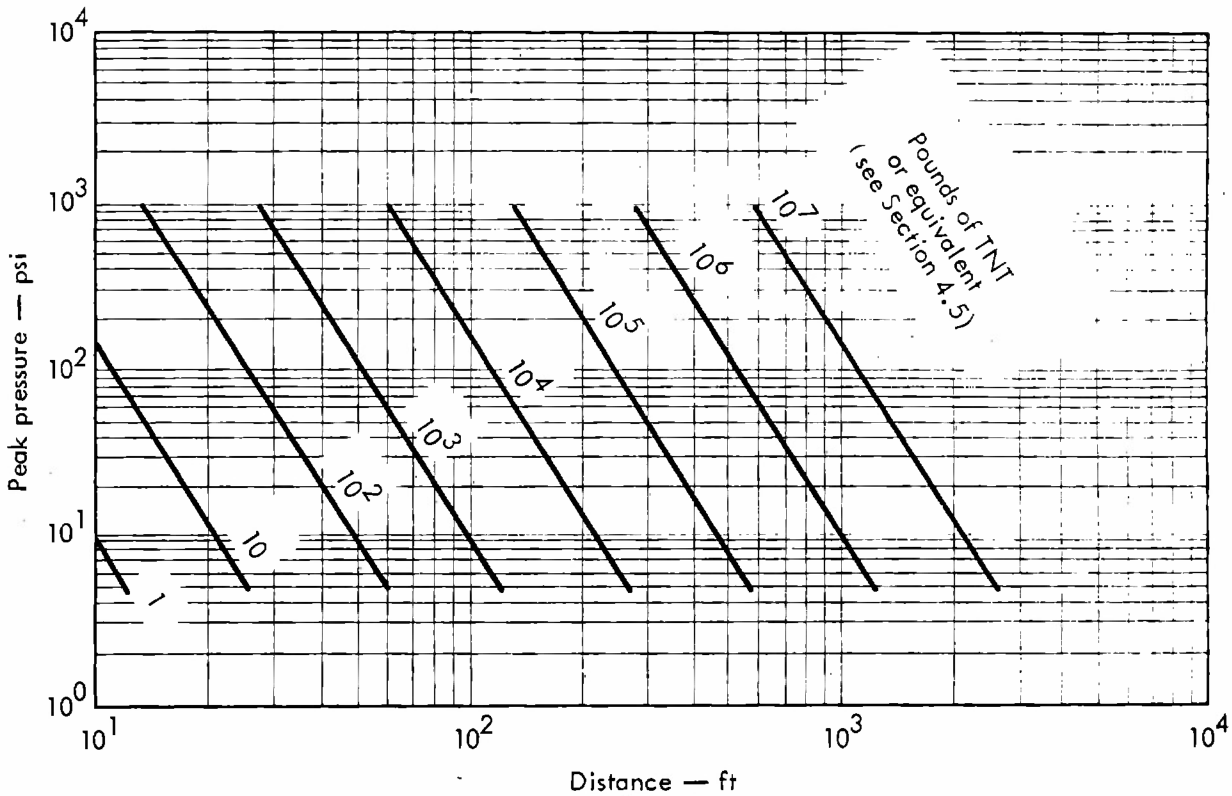

Fig. 32. Underwater shock from explosions in medium overlain by water for Z = 1.9D and $D/Y^{1/3}$ = 0.292.

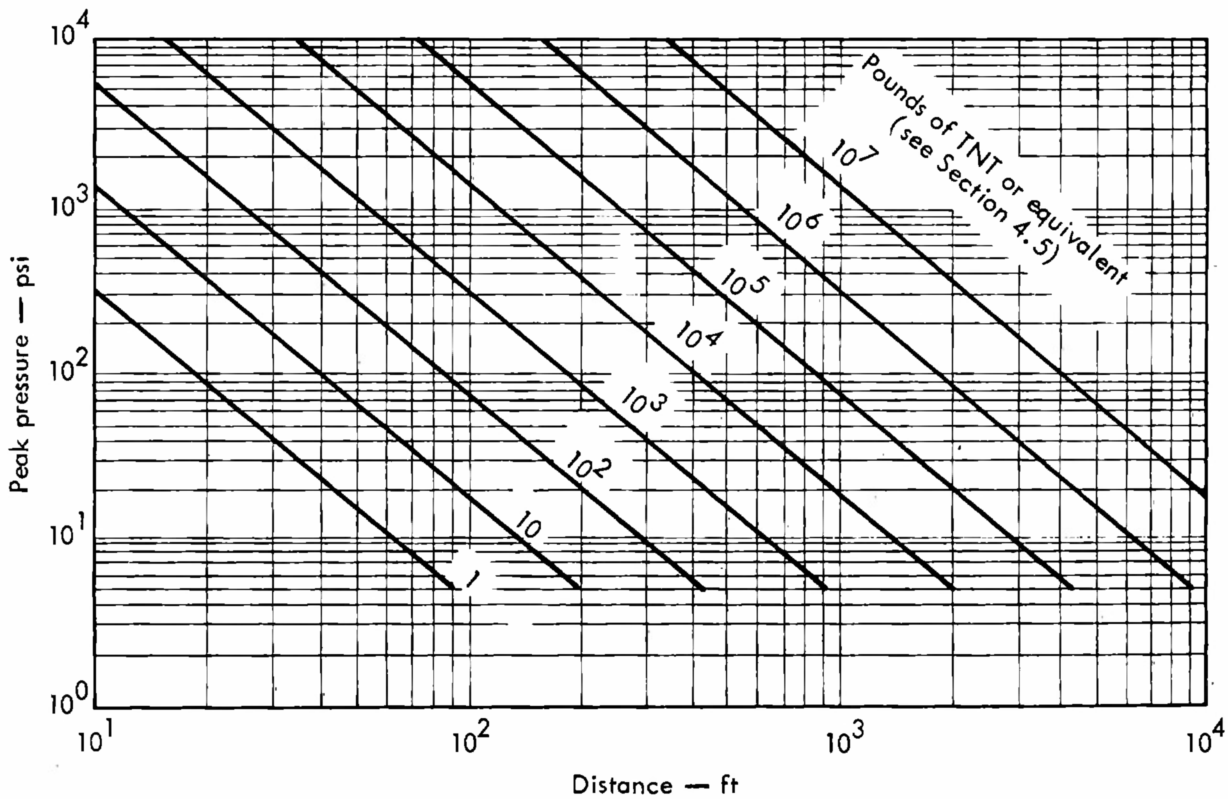

Fig. 33. Underwater shock from explosions in medium overlain by water for Z = 1.225D and $D/Y^{1/3}$ = 0.585.

the ground shock from a corresponding land cratering detonation. The factors which govern the injury or damage that will be sustained are: (1) proximity to the source of the blast, (2) size and character of the explosive, (3) the degree of submersion of the receiver (swimmer, diver or structure), (4) the influence of boundary reflections, (5) the duration of the pressure pulse, and (6) the location of the charge with respect to the medium-water and water-air interfaces.[18]

b. Charge Buried in Medium Overlain by Water

Investigations of detonations in a medium overlain by shallow water show two trends: first, the shallower the water the faster the rate at which peak pressure attenuates; and second, the magnitude of the peak pressure decreases at a particular range as the charge is moved from mid-depth to bottom to below bottom.[19] Figures 32 and 33 may be used to estimate the peak pressures at various ranges for charges below the bottom in shallow water ($D/Y^{1/3} < 0.6$) which is the general region of interest for explosive excavation projects. In the figures, water depth, d, is in feet; charge, Y, is in pounds and charge depth below the water surface, Z, is in feet. The validity of the curves in Figs. 32 and 33 for other than the Z/d and $D/Y^{1/3}$ values given have not been established.

c. Safety Criteria

Air-filled organs, such as in the chest and abdominal regions of a man or swim bladder of fish, are vulnerable to injury from underwater shock waves. Available data indicate that damage to humans and marine life with air-filled organs is a function of impulse as well as peak pressure.

In the underwater situation, no maximum safe values of peak overpressure or impulse for humans have been determined to date. General practice is to remove all persons from the water in the general vicinity of the blast prior to the detonation. Where removal of all personnel is not possible, there is some evidence that a 5-psi overpressure may be tolerated by a swimmer with head in the water.[20] This value is based on the fact that the human eardrum has a threshold in air of 5 psi for rupturing whether the pulse duration is of 3 msec or 400 msec.[21,22] In calculating the range to the 5-psi overpressure, conservative assumptions should be made concerning charge detonation conditions, amplifications of peak pressure due to the type of explosives employed, and effect of hydrostatic pressure on submerged persons.

No specific criteria in terms of underwater shock pressure are available for fish and marine life. Fish cage experiments during underwater cratering detonations show that some fish can be killed out to the maximum range of ejecta missiles, but the kill range is generally somewhat less than this. Marine life containing air-filled organs, such as fish, have been injured at peak pressures exceeding 70 psi with death commencing at peak pressures of 130 psi.[23] Oysters and other types of marine life having no swim bladders are practically invulnerable to shock wave damage.[23]

Damage criteria for underwater structures cannot be generalized. Each type of structure will require examination by

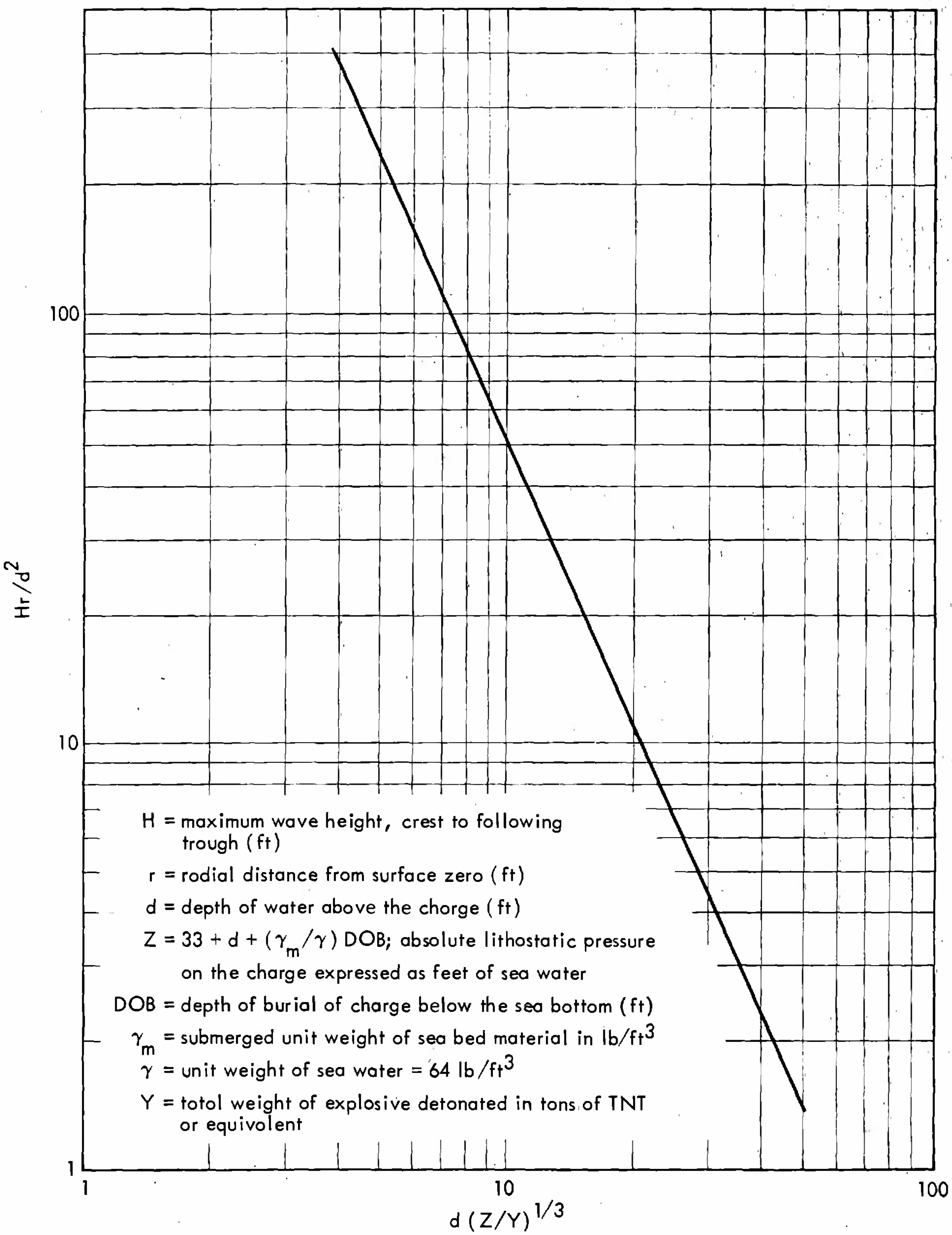

Fig. 34. Maximum water wave height generated from charges detonated in medium overlain by water.

a qualified engineer to determine the structure's ability to withstand the peak shock pressures, impulse, and energy associated with the situation of interest.

6.6 WATER WAVES

a. Safety Considerations

Explosion-generated water waves of sufficient size can cause damage to waterfront installations, such as seawalls breakwaters, or piers. Small boats in the area may be upset. Also, wave run-up on the shore can cause damage to structures and equipment.

b. Wave Height Predictions

The maximum height of waves generated by a single-charge or multiple-charge array buried beneath the sea floor can be predicted by using the curve in Fig. 34 (Ref. 24).

c. Damage Criteria

The construction of structures adjacent to bodies of water and the methods used to berth small craft vary considerably depending on the expected exposure to the sea or the wind, and water-traffic-generated water waves in inland waters. Thus, no general damage criteria can be stated for structures such as piers, wharves, breakwaters, and other facilities adjacent to the water. In each situation the wave height at the points of interest will be required and the effect of the movement of the water mass or change in water level on the structure must be evaluated.

In evaluating the effects of the water wave one must consider its potential to inundate, to displace, and to transport both floating objects and those fixed to or resting on the adjacent shoreline.

6.7 MISSILE HAZARDS

a. Safety Considerations

Missiles (ejecta and other fragments) resulting from explosive detonations on the surface of the ground can be hazardous to personnel at ranges beyond the hazard range for airblast overpressures. Missiles from the detonation of buried charges pose a similar hazard. Property such as motor vehicles, construction equipment, buildings and other structures can be damaged by missile impacts. Thus, the missile hazard is important in selecting explosive storage areas and in establishing safe separation distances for personnel and mobile equipment from storage areas, as well as in planning for and analyzing the effects of an explosive excavation detonation.

In evaluating the missile hazard to personnel and mobile equipment, the maximum range of missiles is the principal concern. Where structural safety or missile impact effects are of importance the maximum missile range, missile size, and distribution per unit area must be considered. Missile size and distribution per unit area may be used to determine the cost effectiveness of protecting fixed objects within maximum missile range.

b. Prediction of Missile Range

For surface explosions of 0.5 to 500 tons the maximum missile ranges depend on the shape of the explosive charge. If the configuration of the stored explosive is approximated by a hemisphere or a sphere, ranges may be calculated using the following relationships[25]:

Hemispheres: $R_{max} = 630\ Y^{0.4}$ (29)

Spheres: $R_{max} = 1470\ Y^{0.4}$, (30)

where

R_{max} = maximum range in feet

Y = charge weight in tons of TNT equivalent

For buried explosives in the range of 0.5 to 500 tons, the depth of burial influences the range of the missiles. Figure 35,* developed from field data, may be used for the prediction of maximum missile range.[25]

When used for personnel safety considerations, ranges calculated from the above relationships should be multiplied by a safety factor of 2.

c. Missile Size and Distribution

The following procedure has been developed for estimating missile size and distribution per unit area.[26] The procedure was developed from data for detonations of half-buried spherical charges in a rock medium and is believed to be conservative for cratering detonations, because half-buried charges in rock produce more missiles than either surface charges or fully buried charges in either rock or soil.

Step 1

Select maximum and minimum ranges at which missile characteristics are desired, and consider this as a ring.

Step 2

Calculate the area of the selected ring.

Step 3

Using the distance from the detonation point to the midpoint of the ring, determine from Fig. 36 the specific area value (the area in square feet in which one missile will be found).

Step 4

Calculate the number of missiles in the ring by dividing the ring area by the specific area value.

Step 5

Convert the mid-ring radial distance to a multiple of the crater radius. The following relationship based on detonations of half-buried charges is used to determine the apparent crater radius for the selected charge: $R_a = 10.0\ Y^{0.3}$ (R_a is apparent crater radius in feet, and Y is charge weight in tons).

Step 6

Estimate missile size distribution from Fig. 37, using the missile size distribution curve which approximately corresponds to the mid-ring radial distance in crater radii determined in Step 5 above. It is necessary to interpolate between size distribution curves when the mid-ring distance does not approximately equal any given curve in Fig. 37.

As an example, consider a spherical charge of TNT weighing 50 tons. Missile

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*Figure 35 was developed from field data in which both nuclear and chemical (TNT and nitromethane) explosives were used.

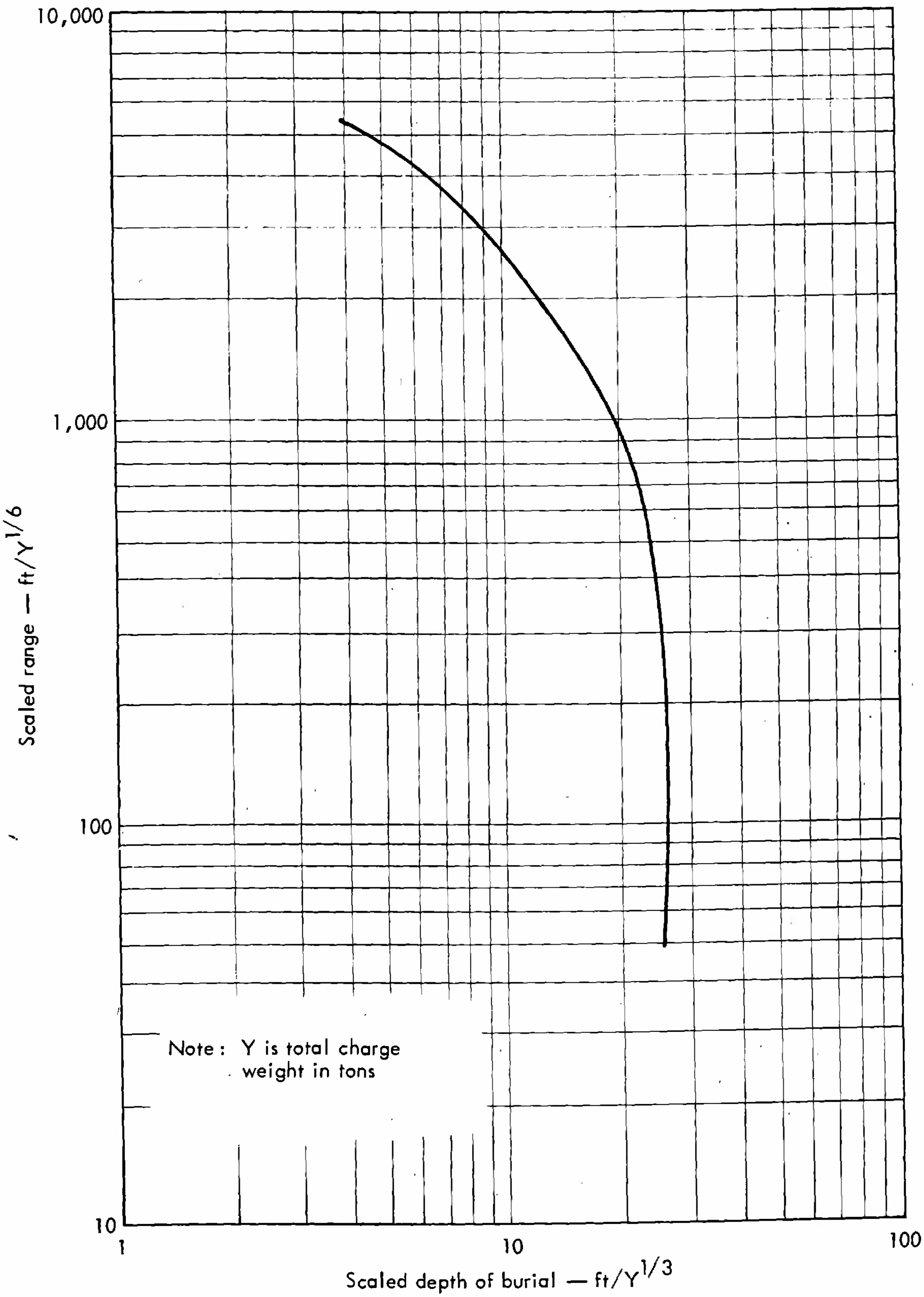

Fig. 35. Prediction of maximum missile range from detonation of buried charges (from Ref. 25).

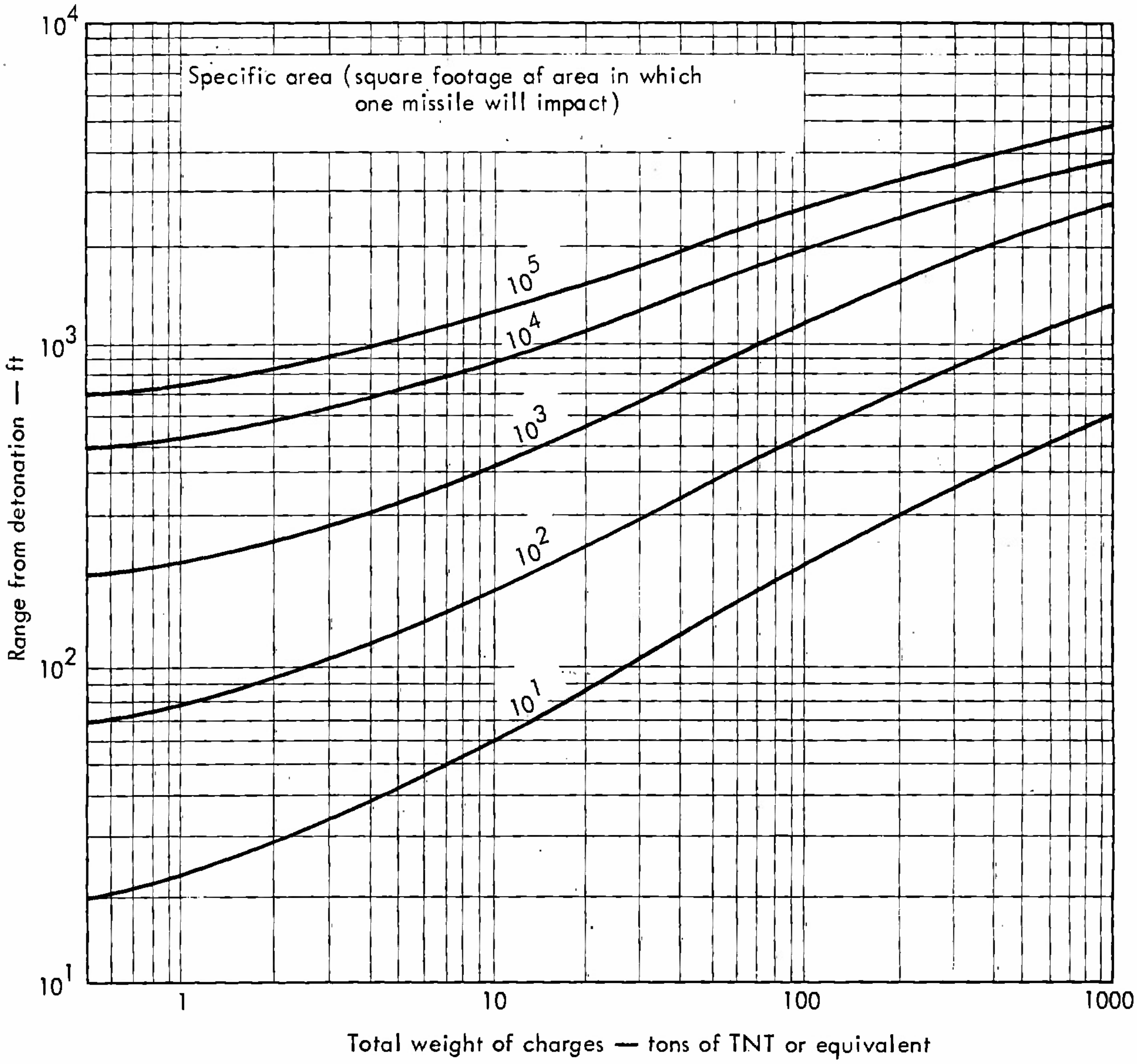

Fig. 36. Specific area at various ranges for missiles from cratering detonations (adapted from Ref. 26).

characteristics are desired within a ring of radii 500 to 600 ft from the center of detonation. The area of this ring is approximately 345,600 ft^2, and its specific area value of 550 ft (from Fig. 36) is approximately 300 ft^2 per missile. (This means that one missile impacts in every square about 17 ft on a side and located within the region of interest.) Thus, the 500- to 600-ft ring would contain approximately 1,150 missiles. From the relationship given in Step 5 above, the apparent crater radius for a 50-ton spherical charge detonated half-buried at the surface of a hard rock medium would be about 32 ft. The center of the 500- to 600-ft ring is about 17 crater radii (550/32 = 17). Interpolating between the 12- and 26-crater-radii size distribution curves shown in Fig. 37, it is found that approximately 50% of the missiles will be less than 4 lb and 80% will be less than

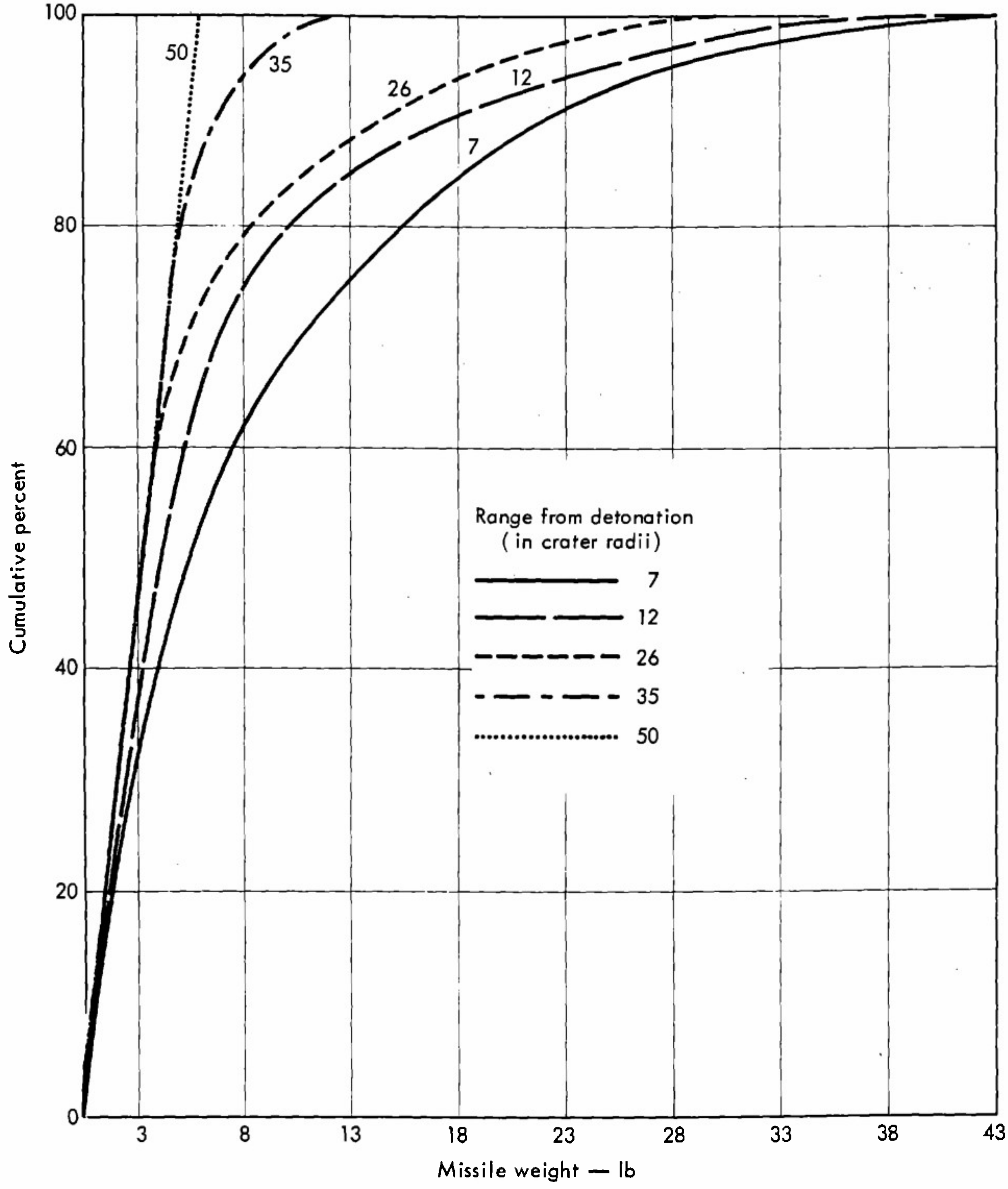

Fig. 37. Missile size distribution at various ranges from cratering detonations ((adapted from Ref. 26).

9 lb, and no missile will exceed approximately 30 lb. A structure or object occupying 600 ft^2 of ground surface in this zone may incur two missile hits of the weights stated above.

6.8 ENVIRONMENTAL CONSIDERATIONS

a. General

This section discusses the principal environmental consequences due to the ground motion, air and/or underwater shock overpressures, ejecta and dust, and dirt cloud resulting from cratering detonations. These effects prevail for only a very brief time at detonation. Except for the potential safety and damage hazards previously discussed, long-term aftereffects in most cases are expected to be inconsequential. The ecology of the area could be affected (e.g., the influence of underwater detonations on fish); however, it is not possible to draw specific conclusions about such effects from the data currently available. Specific investigations are required to determine the environmental impact of using explosive excavation on any particular project; however, the impact from certain types of projects is generally known and is briefly described in the following paragraphs. Guidelines are provided for the conduct of specific investigations.

b. Land Cratering Projects

For construction projects at land sites removed from bodies of water and streams or rivers, the effects of the explosive detonations on the environment will be minimal and, except for the problems resulting from the distribution of dust and dirt, will be confined to the area immediately adjacent to the detonation point. Ejected material will cover the area immediately adjacent to the detonation point and will destroy trees and plants for several hundred feet depending on the size of the charge detonated and emplacement conditions. The area of plant life destroyed by the ejecta is not significant because the area most likely would be cleared in normal construction operations. Seismic motions will not be of sufficient magnitude at distances outside the construction area to interfere with plant life. Air overpressure damage to plants will also be confined to the immediate area about the detonation point. Terrestrial animals and birds will normally leave the immediate area prior to detonation due to the noise of other construction activities and may be expected to remain away until such activity ceases.

The dust and dirt resulting from explosive detonations, particularly when carried out under selected weather conditions, may be less disturbing to the environment than that associated with conventional construction techniques. Detonations can be scheduled at times when the winds will carry the dust and dirt to areas of minimum concern. The shorter construction time and minimum amount of heavy construction equipment inherent in using explosive excavation to move or fracture the media will reduce the requirement for access roads, the time during which dust and dirt are created, and the time construction noise persists, thereby reducing deleterious effects on the landscape.

c. Underwater Cratering Projects

The effects of underwater detonations are generally inconsequential on the environment outside the water. Effects on fish and other water life are more difficult to classify and require more study.

Investigations were conducted in conjunction with the Project Tugboat underwater cratering tests in Hawaii in 1970.[24] These investigations included observing and recording the marine life at selected locations prior to, during, and following the detonations, determining by the anchored fish cage technique the distances from the detonations to which fish kills and injuries were incurred, and collecting the dead and stunned fish following each detonation. Fish counts at various stations during the period of observation produced highly variable data making it impossible to draw definite conclusions. For nominally 40-ton detonations, the distances to which all fish in fish cages were killed ranged from 100 to 210 ft for three separate detonations. The maximum distance to which any fish were killed or injured probably did not exceed 300 ft for any single 40-ton detonation. Certain specimen of fish appeared to be more vulnerable to shock than others, based on the postshot collection program. Many of the stunned fish recovered and survived.

d. Guidelines for Environmental Investigations of Specific Projects

(1) Determination of Predetonation Conditions

Prior to the detonation of explosives, the environmental and ecological conditions that exist must be investigated. This investigation should have the objectives of identifying all of the damage that might be inflicted on the area and its terrestrial and aquatic (flora and fauna) inhabitants, indicating the protective measures that may be required to prevent or minimize damage, and serving as a documented baseline of predetonation conditions. Activities such as field surveys to count the actual numbers and types of flora and fauna should be undertaken if considered necessary after consultation with agencies managing such resources. Where possible, the State and Federal Agencies managing the resources of an area should be invited to perform the predetonation investigations and to conduct research programs and tests in conjunction with the project. These agencies have an official interest in specific problem areas, are staffed with professional people capable of performing tasks connected with their area of interest, and are already in possession of the requisite basic data.

(2) Determination of Preventive Measures

Areas of concern identified by predetonation investigations should be examined in detail to determine what measures can be taken to eliminate or to reduce adverse effects. To some extent the measures discussed in Section 6.2d(4) apply to natural area features. The possibility of using a small underwater shot prior to the main detonation may be desirable in certain instances to reduce fish kill. Preventive techniques should be explored in consultation with the State and Federal Agencies and other organizations concerned with the particular problem and the feasibility of implementation of the techniques established.

(3) Determination of Detonation Effects

To determine the postdetonation conditions, a thorough reconnaissance of the area noting and recording any damage to foliage and wildlife is desirable. Photographs of the area and of any specific damage are very valuable extensions of written descriptions.

In aquatic construction areas, the effects of the explosion on fish and other marine life may be difficult to determine because of tides, currents, wave actions, and water depths. Under these conditions it may be desirable to place caged specimens in the water at various distances from the detonation point. Detonation effects can be determined by counting and retrieving any dead marine life, reexamining fixed marine life about the area, and retrieving and recording the condition of restrained specimens.

Postdetonation surveys should be conducted not only with the objective of documenting effects but also of determining and evaluating measures for the reduction of undesirable effects on the environment in future projects.

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Chapter 7

Field Operations

7.1 SCOPE

The operational considerations and recommended staffing for the emplacement construction and detonation of an explosive excavation project are discussed in this chapter. The preshot field construction activities, the procurement and emplacement of explosives, and the sequence of operations leading to detonation are covered with the view toward clarifying the types and scope of field activities which comprise explosive excavation projects. Appendix F supplements this chapter by providing guide specifications for use in contracting for emplacement construction, explosives procurement, and the emplacement, arming, and firing services for an explosive excavation project.

7.2 OPERATIONAL CONSIDERATIONS

In explosive excavation the success of the project depends both on the design and on the implementation of the design. There are as yet few agencies or organizations with experience in explosive excavation design. On the other hand, there are many firms capable of performing the construction work to implement a design, and therein lies a possibility for cost-saving innovations brought on by competition. The contracting procedures for the type of construction support involved are well known and routinely practiced.

Generally the explosive excavation design will specify the weight, the location, and the depth of burial for each cratering charge to be emplaced and detonated by the contractor, as well as the type of explosive and total quantity to be detonated at one time. Predictions for detonation hazards and damage effects, such as maximum missile range, airblast overpressure, and ground motion, will have been made during the initial design phase, and provision must be made to assure that detonation takes place only when the criteria used in assessing these damage and hazard effects are met. To accomplish this a high degree of control is required, and execution authority should be maintained by the Contracting Officer or his designated representative.

Thus, while the construction support required in explosive excavation is not of itself so unique or sophisticated as to require specialists for design or new contractural procedures, explosive excavation does add a certain degree of complexity to normal construction operations familiar to most construction agencies. The important operational aspects of an explosive excavation project are discussed below.

a. Detonation Scheduling

Where explosive excavation is used on a portion of a civil works project the detonation should be phased into the overall construction schedule. It would generally be desirable to perform the detonation before the conventional construction associated with that portion of the project is carried out. This would facilitate control of the area when explosives are being used. The project construction equipment is then readily available for postshot construction to alter the excavation if necessary and to prepare necessary drainage to protect the excavation. The material produced by the explosive excavation detonation would thus be available for use as fill or riprap.

As discussed in Chapter 6, airblast overpressure might be a major consideration for scheduling the detonation on some projects. The hazards from airblast can sometimes be reduced by selecting a detonation time when the prevailing upper level winds blow away from centers of population. In selecting detonation times that will meet the safe firing criteria for airblast overpressure and area surveillance, weather information as to anticipated cloud cover, air temperature, and wind speed and direction is generally required. Predictions available from the local U.S. Weather Bureau or local airports are normally adequate for this purpose. The detonation schedule is then confirmed by visual observations and temperature and wind measurements made on site.

Another consideration, if detonations are in or near the water, is fish spawning runs. The detonation should be scheduled to minimize fish kill.

b. Detonation Effects Measurements and Documentation

Photographic coverage, airblast overpressure, and seismic motion measurements at locations selected during the safety analysis may be warranted to verify design predictions and to document the detonation and its effects. These data may also be useful in evaluating the validity of any possible damage claims. Generally these measurements can be made with self-contained recording units that require no field support other than access and voice communications with the control point.

c. Project Control Point

Project management during detonation should be conducted from a field office located to provide a clear view of the detonation area, but outside of the maximum missile range. This control point serves as the firing point and command center for implementing the project safety program and coordinating effects measurement programs.

d. Area Control

The hazardous area should be evacuated and controlled during periods of explosives emplacement and detonation to insure that project personnel and the general public are not subjected to unnecessary danger. Personnel access during explosive emplacement operations should be limited to those engaged directly in the work. The size of the controlled access area is dependent upon the amount of explosives to be emplaced, the potential ejecta range, relative operational hazards, and practical work problems. The area thus established should be barricaded and posted with signs designating the presence of explosives and access restrictions.

The tentative detonation schedule should be furnished to the Federal Aviation Agency (FAA) for issuance of a warning to pilots. Vertical and horizontal boundaries of the hazardous area, as well as coordinates of the project location, should be included. Generally the FAA will request that the schedule be confirmed just prior (12 to 24 hr) to each detonation and that they be notified immediately after the area is cleared.

Prior to detonation, the entire hazardous area should be rechecked to insure that all entrance roads are barricaded and posted with signs designating that blasting is in progress. The evacuation should be confirmed by a ground sweep just prior to arming the charges. Once this check has been made, a strict system of accountability and guard posts should be instituted to restrict access into the area to authorized personnel only, these generally being the arming party. Guard posts must be in continuous communication with the control point. The following additional safety precautions are applicable:

(1) Visibility at detonation should be sufficient to maintain visual surveillance of the entire shot site and air space over it to 1000 ft above the maximum height of hazardous effects to aircraft.

(2) The entire area should be kept under surveillance from the start of countdown through the detonation.

(3) A direct communications channel should be provided between the guards and the person at the control point with execution authority.

(4) Detonation shall not take place if any person, domestic animal, vessel, or aircraft is observed within the potentially hazardous area. Exceptions are project participants who will be in communication with the control point and in adequately designed protective shelters.

e. Logistical Support

The logistical support required by explosive excavation is dependent on the nature of the project application and its location. In general it includes that support necessary to place a work force, and materials and equipment on project to perform emplacement construction, explosives storage, loading, and detonation services, and is identical to most construction projects except that a reliable communications system encompassing the explosives excavation portion of the project is essential.

f. Explosives Storage and Handling

The provisions of the Corps of Engineers Safety Manual[1] pertinent to the use of explosives are applicable in explosive excavation construction. However, the cratering charges most likely to be employed in this type of construction are the free flowing granular, water gel, or slurry blasting agents which may not be classified as explosives but rather as oxidizing materials (nitrocarbonitrates). Further, bulk delivery and mixing equipment is not specifically covered by the safety manual but will often be used, considering the quantities normally required in explosive excavation. In these instances the recommendations of the other publications listed under "Applicable Publications" in Appendix F should be followed to supplement the requirements of the safety manual.

A fenced placarded storage compound, cleared of flammable material, and with separate locked magazines for storage of initiators and boosters is recommended. The compound should be large enough to accommodate the bulk explosives (blasting agents), to maintain the recommended safe separation distances, and to allow space for the bulk mixing and delivery equipment. A sheltered work space should be provided for fabrication and assembly of explosive components within the compound. The compound should be treated as a limited access area.

7.3 STAFFING FOR FIELD OPERATIONS

The concept of explosive excavation contract construction as stated previously is similar to normal contracting procedures except that the Contracting Officer or his representative retains detonation execution authority. It is the exercise of this authority that suggests some innovation to the normal staffing on projects be followed to provide a well-informed management and staff extending from the formulation of the project concept through design and execution. The individual who will have execution authority should be a member of the project design group. Further, an advisor from the design group should be a member of the field construction management staff. Construction methods and techniques will thus be made available for consideration prior to completion of design, and design intent and safe firing criteria used in assessing hazardous detonation effects will thus be clear to a representative of the field staff, assuring that the project objectives will be fully met.

A security officer, responsible for the development and implementation of a project security/safety plan should be a member of the project field staff. This function may be fulfilled by contractor personnel, but the individual shall be directly responsive to the one with execution authority at detonation. He will also supervise the contractor guard personnel employed to evacuate and control the detonation area. These are the only recommended changes to the contracting agency's normal construction staffing.

7.4 PRESHOT CONSTRUCTION

a. Emplacement Construction Techniques

Emplacement construction will normally be by some method of drilling or a combination of drilling and hole enlarging. Full-charge-diameter drilling is the most straightforward method of emplacement construction and is the one used in normal drilling and blasting operations. The several methods usually used are auger, calyx, and rotary drilling; selection of the one best-suited to a particular project will normally be based on availability of equipment and the strength of the material to be drilled. Hole springing and underreaming are two methods of hole enlarging presently being investigated and both appear to offer an economic advantage in emplacement construction under some conditions. Hole springing is a method of enlarging a small diameter drill hole with explosive charges to obtain a cavity of the proper size to receive the design charge. This method appears to offer an economic advantage in certain types of media. For charges up to 10 and possibly 20 tons, underreaming should offer an economic advantage. This method uses a tool that will go down a relatively small hole and expand to enlarge the lower portion of the hole to the required dimensions. The advantages of this method of construction are that the total length of hole does not have to be drilled to the diameter of the explosive charge and the amount of stemming is less than that for a full-charge-diameter hole. Because of the additional tooling cost for this method of drilling, it may not be more economical than full-charge-diameter drilling except on a project where there are many large diameter charges to be emplaced. For bidding purposes, the contractor should be furnished the desired charge configuration and not the method of construction. This will leave the emplacement method up to the contractor and will normally produce the lowest bids.

b. Stemming

After the explosives and booster have been emplaced, the remaining hole should be stemmed. The primary function of the stemming is to contain the gas pressure for as long a time as possible; i.e., to prevent premature venting. As mentioned in Chapter 5 there are a number of materials that are satisfactory for stemming. Free flowing materials are usually convenient and adequate for the purpose. This material should be dampened and tamped sufficiently to insure consolidation as it is being placed. The material may be saturated to facilitate consolidation if the explosive is water-resistant.

7.5 EXPLOSIVE EMPLACEMENT, ARMING, AND FIRING

The recommended procedure is for the explosive contractor to emplace, prime, and fire all explosive charges and furnish all necessary items of equipment to accomplish this work. The design will specify all particulars concerning the explosives to be used as the cratering charge: how much, what kind, and where; and bids may be solicited on that basis. The initiation system should be designed and installed by the contractor furnishing the explosives to insure that all components are compatible and a reliable system results. This places on the explosives contractor the total responsibility and consequently total liability for the proper detonation of all charges. The contract should specify that all explosives work be accomplished under the direct supervision of an explosives engineer of proven experience and ability in blasting operations.

A column booster equal in length to three-fourths the charge length and primed its full length by detonating cord has proven adequate for initiating bulk blasting agents. In any case the booster and any individual increment thereof must be capable of complete initiation of all the material used in the main explosive charge. Unless timed delays are specified, all charges for any given detonation should be fired simultaneously within a reasonable degree of accuracy. Detonating cord downline cut in equal lengths and tied to a loop trunk line is adequate. Delay shootings require special care. Close supervision by experienced persons of all design and field operations related to the delay system is recommended.

Firing of the charges is accomplished from the control point by hard wire via a controllable source of electrical energy capable of activating the electrical explosive initiators. Where the distance from the control point to the detonation area is too great to reliably fire the initiators, a special remote-controlled blasting unit with key-operated safety switches should be used. Firing commands should always originate from the control point.

7.6 DETONATION PROCEDURES

Prior to actual detonation the main firing and control cables will be run from the control point to the detonation area. A dry run on all systems including the initiation system should be made with actual or simulated loads activated at connected stations.

Upon receipt of notice from the person with execution authority, the charges will be armed by connecting detonator leads to the blasting machine and by connecting auxiliary equipment.

At the discretion of the contracting officer, and when it has been determined that the detonation area is clear, the contractor's explosives engineer will give the appropriate warning signal and turn on the firing unit allowing countdown to commence. After detonation the firing unit will be locked out and the explosives engineer will inspect the area for evidence of misfire, unburned explosives, etc. Upon receipt of his report of "all clear," personnel access restrictions may be removed.

In the event of a misfire, the explosives engineer is to recommend the exact procedure to be followed. Depending on the apparent reason for the misfire, the following procedure will normally be followed: the detonator cables will be removed and a continuity check performed to determine integrity of detonators. If the continuity check is satisfactory, short out detonators and recheck functioning of the blasting unit with simulated loads. Replace faulty units as required. Repeat the entire arming and firing procedure.

REFERENCE

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Chapter 8

Postshot Evaluation and Construction

8.1 SCOPE

A cratering explosion in soil or rock produces significant changes in the media surrounding the visible crater. To assess the potential long-term behavior of the excavation, one must be able to predict or measure the dimensions and physical characteristics of the various crater zones created by the detonation. The nature of these zones determines such behavior factors as slope stability, settlement, and seepage, all of which must be considered before placing a foundation in or around the excavation. Additionally, the excavation may not conform to project requirements, in which case postshot earthwork will be necessary to shape the excavation to the dimensions specified by the project design. This follow-on construction will vary depending on the results achieved by the detonation and the nature of the project.

This chapter presents information pertaining to methods of collecting data on the crater and its material properties, and analyzing the data in relation to the engineering properties and the behavior of the various crater zones. The construction activities required to convert the excavation into a useful facility are also discussed.

8.2 CRATER PROPERTIES EVALUATION

One of the first steps after the detonation is to measure the dimensions of the excavation. These measurements allow a comparison with preshot predictions and with design requirements, and provide information for subsequent construction.

Next, data must be collected and analyzed so that the nature of the excavation can be determined and the long-term behavior can be predicted. Generally this investigation will involve determining particle size distribution, measuring the angle of deposition, and assessing strength, compressibility, and permeability as they affect the proposed use of the excavation. The data which are collected and the analyses which are performed will depend on the project. Necessary data collection methods and analyses for most cratering applications are discussed in this section.

a. Surface Investigations and Evaluation

(1) Crater Measurements

The survey method employed will depend upon the accuracy required and the size, nature, and geographic location of the excavation.

For very large excavations aerial photography should be considered. The costs may be less than those for extensive ground surveys. This method requires a limited ground survey to establish control points. Stereoscopic photographs are used to prepare topographic maps from which the crater dimensions are derived. The volume of the apparent crater and the volume of the crater lip out to the boundary of the continuous ejecta can be computed by standard borrow pit calculations utilizing preshot surveys as a control base. Computer programs are available for making such calculations.

Ground surveys are accomplished by reestablishing preshot survey control and preparing cross sections. This technique will require field personnel to work on and in freshly excavated craters. The work is generally slow because of rugged slopes and difficult footing. Ground surveys are generally most suitable for small craters and for rapid preliminary estimates. In the case of an underwater excavation, the apparent crater profile can be determined quickly with a fathometer.

The primary purpose of these crater measurements is to determine whether the dimensions of the excavation meet the design specifications, in order that any required postshot construction work can be estimated. The measurements also allow the bulking factor to be estimated. The bulking factor is the ratio of the volume of bulked material to the preshot in situ volume. From an analysis of data derived from craters in various media, the following rule of thumb gives a reasonable estimate of the bulking factor:

$$\text{Bulking factor} = \frac{V_a + V_{al}}{2\,V_a}, \qquad (31)$$

where

V_{al} = volume of apparent lip (mass of material)

V_a = volume of apparent crater (void)

This formula was developed by assuming that the volume of the true crater is twice the volume of the apparent crater. The formula neglects the uplift in the lip but since the uplift is bulked also, the error is minimized, resulting in values generally differing by less than 10% from measured values.[1] The advantage of this formula is that extensive and costly field investigations can be avoided and the bulking factor can be determined using volumes found by ground surveys or aerial photography.

The postshot porosity for the rubble can be determined from the bulking factor for the rubble and the initial porosity. The relationship is established by determining the volume of voids before and after the detonation. The void volume after the detonation is evaluated by adding the new voids introduced by the bulking process to the original voids. This process implies that the initial porosity is not diminished by compression due to the blast. Figure 38 shows these relationships. Entering the figure with the bulking factor (such as 1.4), and reading up to the known initial porosity (in this case assumed to be 20% as indicated with the dotted line "A"), the postshot porosity can be read directly. This porosity may have limited value as an indication of seepage, but is a valuable property for estimating settlement. The bulking factor for the rupture zone, discussed in Ref. 2, can also be found by using Fig. 38 as explained later in Section 8.2b(1).

(2) Particle Size Distribution

For crater rubble, particle size distribution appears to be the best indirect measure of permeability. Seepage and slope stability are influenced by particle size. Also, settlement of rock fill has been found to be a function of particle size. In quarry operations, particle size is the major specification. Determining

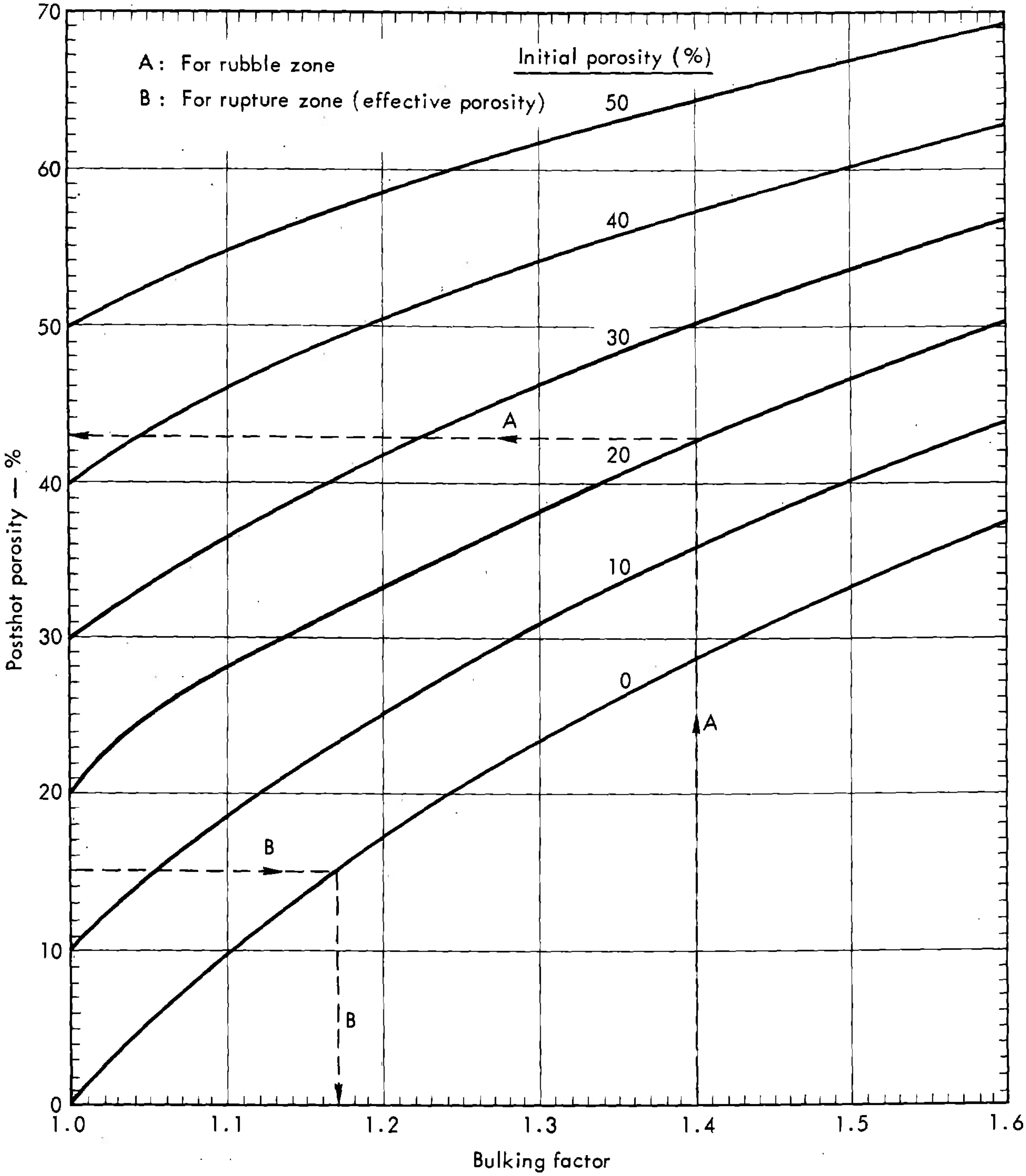

Fig. 38. Relationships for initial porosity, postshot porosity, and bulking factor.

particle gradation on craters has generally been accomplished by mechanical sieving and weighing. High costs and the time required to obtain data by using this procedure have stimulated development of two alternate techniques: grid photography and point-count sampling.

The grid photography technique is a method of sampling the distribution of particles to determine their frequency of occurrence by size and weight. This method involves photographing the surface of ejecta through a square with side dimensions varying from 6 to 10 ft, and containing a grid of 1-ft squares. These photographs are taken with the camera mounted at the apex of a lightweight pyramidal frame. From these photographs, the percent of the area occupied by particles of a given size group can be estimated for each grid square. The percent of the area occupied is then assumed to be directly related to the volume and weight of the ejecta. The results of this technique have compared favorably with a mechanical sieving analysis; however, the method has been subjected to only limited testing.[3]

The point-count technique is new and possibly more economical and practical than either the grid photography method or the mechanical sieving process. One or two men utilizing the point-count technique can provide a reasonable gradation curve. The method involves the establishment of the sample area within the bounds of the continuous ejecta. A tape and a compass are utilized to set up a series of random sample lines. A grid spacing interval is selected such that the interval is larger than the largest block to be encountered. After selecting the grid spacing interval and laying out the tape, the investigator will consistently select particles lying under one side of the tape, at the predetermined grid spacing interval, and measure the length of the intermediate axis of the particle. The blocks or particles of material under the tape, at the spacing interval, make up the sample and the length of the intermediate axis provides the particle size. The particle sizes are marked on a tally sheet that has been subdivided into a series of size classes. These data are then used to compute the percentage of particles making up each size class. By counting at least 500 samples, a 90% confidence level can be attained for the gradation curve. In controlled tests, this relatively simple procedure has compared favorably with the more expensive and time-consuming procedures of sieving and weighing.[4] In addition, this method is preferred over the grid photography method because the length of the intermediate axis is actually measured.

(3) Angle of Deposition

Empirical data indicate that the slopes of explosive excavations in rock materials are inherently stable. The fallback and ejecta come to rest after violent impact. The fallback comes to rest on a slope, the inclination of which is termed the angle of deposition. This angle is the inclination of the line best fitting the more planar, upper portion of the fallback, disregarding both the steeper slope immediately below the crest (sometimes caused by the exposure of the true crater lip) and the more curved lower portion of the crater. In general, this line is tangent to the apparent crater surface at the level of the preshot ground surface. This angle, at which particle movement stops, is generally several degrees flatter than the angle of repose, at which particle movement would resume. The angle of repose is the maximum stable angle for a given cohesionless material. Weathering can reduce this angle for some materials. Table 16 contains typical angles of repose for various materials.

In general, the angle of repose of a cohesionless mass of angular materials varies between 37 and 45 deg. For craters in rock having angular fallback particles, angles of deposition generally range from 31 to 37 deg.

b. Subsurface Investigations and Evaluation

(1) Drilling and Coring

The most common method of collecting subsurface data is drilling. Postshot drilling may use NX (3-in. diameter) size core borings or larger. Data acquired from these borings can be used to infer the nature and extent of the fallback and rupture zones. In some instances, it may be desirable to drill inclined holes to better observe rock structural conditions.

Information from the cores can be supplemented by the use of borehole photography. If the film records are clear a good determination of subsurface rock conditions can be obtained. In addition to structural information, if fracture widths are measured, the borehole camera can provide quantitative information on effective porosity.

Effective porosity, although conceptually simple (see Section 2.2), is difficult to measure in practice. In reporting effective porosity, care should be taken to specify the method of measurement so that the meanings of numerical values are clear. Effective porosity is believed to be the best indirect measure of permeability in the rupture zone.

The bulking factor for the rupture zone can be determined by measuring the effective porosity both before and after the shot. Figure 38 can be used for this purpose. For example, by entering the figure with a postshot effective porosity such as 15%, and reading over to the known initial effective porosity (in this case assumed to be zero, as indicated by the dotted line "B"), the rupture zone bulking factor of 1.17 is read. Bulking factors thus obtained can be used to determine the postshot density for such computations as slope stability analyses.

Water pressure tests can be performed in connection with the drilling to develop an estimate of permeability. Below the water table, pumping tests can be used to develop quantitative permeability data. In such tests, a well flowmeter can be employed to determine the horizontal permeability of the media at different depth intervals.

(2) Geophysical Surveys

A number of geophysical exploration techniques may have application to postshot surveys. These survey methods,

Table 16. Typical angles of repose for various materials.[5]

Material	Angle of repose (deg)
Shingle stone	39
Ore, broken	45
Shale, broken	30-35
Shale, fragments	34-38
Marl, fragments	33-36
Metamorphic rock, fragments	34-38
Stone, crushed	37
Limestone, fragments	38-42
Sandstone, soft	33-37
Sandstone, fragments	45
Igneous rock	37-42
Rubble	37-45

which are based on the laws of seismic wave propagation, can provide valuable information about subsurface velocities and structure. There are three basic types of seismic surveys: refraction, vibratory, and continuous reflection. The principle and application of each type are briefly described below.

The seismic refraction survey relies upon the existence of velocity changes with depth and the accurate measurement of refracted compressional-wave travel times, from a source to a receiver, through the rock or soil. All measurements are obtained at the ground surface. The energy source is generally a small charge of explosive, the seismic waves are detected with sensitive geophones, and the data are recorded on single- or multi-channel electronic recorders. The recorded travel-time data can be reduced to obtain propagation velocities and the location of velocity interfaces. The methods of interpretation of the basic data are outside the scope of this report, but a number of excellent references exist on this subject (see Ref. 6). Carefully applied, a seismic refraction survey may determine depth of fallback and extent of uplift.

The vibratory seismic technique consists of measuring, at the surface, the wavelength of Raleigh (surface) waves generated by a mechanical or hydraulic vibrator. The measurements are performed at a number of frequencies; the depth of penetration of the wave below the surface is frequency-dependent. Although the wavelength of surface waves is actually measured, their propagation velocity is very close to that of a shear wave in the same material, and the final result is interpreted as a profile showing the variation of shear-wave velocity with depth. The vibratory seismic method is more sensitive to increases in fracturing and bulking than the refraction method. When compared to preshot vibratory surveys, the method can often be used to outline the extent of blast-induced fracturing.

The continuous reflection profiling method can only be applied to surveys of craters under water. The technique relies upon recording the subsurface reflections of repetitive acoustic pulses transmitted by an underwater transducer, commonly referred to as a "sparker" or "pulser." The presence and strength of the reflections depend on there being sufficient contrast between the acoustic impedances (which can be taken as velocity times specific gravity) of the different subsurface materials. The information is displayed as a continuous profile of reflection amplitude plotted against depth, and has been used to assist in interpreting the structure of underwater craters (rupture zone and the true crater boundary).

8.3 ANALYSES OF ENGINEERING PROPERTIES AND BEHAVIOR

Three factors determining the engineering behavior of earth material are of interest in most applications of explosive excavation: slope stability, settlement, and seepage. The long-term behavior of explosive excavations is controlled mainly by environment and the nature of the material. Explosive excavations in clay shale or soil are more adversely affected by weathering, water movement, and frost-heave action than excavations in rock. Explosive excavations in rock have caused no engineering problems. The engineering properties of strength, compressibility, and permeability are analyzed in the following paragraphs as they relate to slope stability, settlement, and seepage.

a. Strength and Slope Stability

Slope stability analyses commonly include field observations, comparisons with other slopes, and the determination of a factor of safety. Although influenced by settlement and seepage, the stability of rock slopes is primarily determined by the shear strength of the material when the unconfined compressive strength is greater than about 3000 psi.[7] The same is true for soils, except that the limiting value of unconfined compressive strength is much lower.

The factor of safety is often expressed as the ratio of shear strength to shear stress. Average values of shear strength and stress along an assumed failure surface normally make up this ratio; however, with the advent of the computer, values are sometimes evaluated for discrete increments all along the assumed failure surface. If the shear strength is greater than the shear stress, the slope is stable and the factor of safety is greater than one. The degree of stability is related to the amount by which the factor of safety exceeds one.

This section will relate shear strength and shear stress to the stability of the fallback and rupture zones and will provide an evaluation of the state-of-the-art in the assessment of slope stability.

(1) Strength and Stress

In order for a slope failure to occur in a crater, large masses of material must be displaced. The displacement must involve sliding between the displaced and intact rock mass and must be of sufficient magnitude to impair the engineering use of the excavation. Mere surficial movement does not ordinarily constitute slope failure. In general, the resisting forces along a failure surface are partially frictional in nature and partially due to cohesion. The frictional portion is predominant, except for clay slopes. For a rubble mass, there is no cohesion; therefore, the only contribution to strength is friction.

The mass strength of in situ rock is limited by the strength of discontinuities (i.e., joints, bedding planes, faults, weak seams, and the like); hence, a determination of the spacing, the location, and the orientation of the major discontinuities is an essential step in the study of any particular case. Determining a value to use for the shear strength of the rubble or the rock discontinuities is difficult and requires judgment and experience.

The mass strength of clay shales is more difficult to analyze. If overconsolidated clays and clay shales are subjected to large displacements, there is a significant decrease in shear strength. Upon release of confining pressure (usually the overburden), these materials tend to expand and produce displacements large enough to reduce the shear strength.

The stresses either in a rubble mass or in in situ rock must be estimated. Several methods of continuum mechanics and mechanics of discontinua have been used to make such estimates. These stresses are functions of the geometry of the crater, the properties of the media, and the method of loading. Still, to determine the factor of safety, the values of shear strength must be known, and these values are more elusive than stress values.

(2) Stability in Fallback Zone

A strength analysis of the fallback zone is primarily an assessment of the stability of cohesionless rubble slopes. Since strength is a function of the angle of internal friction and the stresses are a function of the geometry, a factor of safety for a dry cohesionless slope can be expressed as:

$$\text{F.S.} = \frac{\tan \phi}{\tan \beta}, \tag{32}$$

where

ϕ = angle of internal friction

β = angle of deposition or slope inclination

If the angle of repose is substituted for ϕ, and the angle of deposition, β, is measured, the factor of safety can be calculated. The factor of safety for dumped large aggregate, a situation roughly analogous to the formation of crater slopes, normally varies between 1.1 and 1.5.[5]

(3) Stability in Rupture Zone

For other than very weak rocks, the strength of the rock mass in the rupture zone is governed by the spacing and the orientation of the natural and blast-induced fractures; therefore careful site documentation is necessary. The materials near the crater have been highly fractured by the explosion. In the rupture zone natural discontinuities are disrupted and large subsurface displacements are common.[8] Disruption appears to increase toward the surface so that the throughgoing fractures near the top of the slope are likely to be offset. Although dependent on initial orientation of the planes of weakness, deep-seated displacements resulting from an explosion are generally radial; hence, a major throughgoing discontinuity might not be completely offset and an offset certainly cannot be predicted.

In general, blast-induced fractures in the rupture zone will be clean, and the potential strength along the fracture can be compared to the frictional resistance of rock against rock. Exceptions are possible where existing clay-filled fractures are widened by the blast. Analysis of data for dry materials indicates that a throughgoing clean discontinuity dipping toward the crater would have to be inclined steeper than about 30 deg to develop an unstable block.[9] A throughgoing plane of weakness at greater than 30 deg is unlikely because of the disruption of material toward the surface. This unlikely situation plus the buttressing effect of the fallback at a relatively flat crater slope suggests that the rupture zone is stable.[10] Modification of the initial loading, such as cutting the toe of the crater slope, or the building of structures, or the development of additional pore water pressure would require separate analysis.

(4) State-of-the-Art

There are basically two approaches which may be taken to assess the long-term stability of crater slopes: the analytical and the empirical.

In the analytical approach, a given slope geometry can be subjected to a limit equilibrium analysis, a continuum mechanics analysis, or a discontinua analysis. Three methods of limit equilibrium analyses are applicable to crater geometry: the method of slices, the method of slices for a composite failure surface, and the method of wedges. The capability to apply a continuum mechanics analysis via the finite element method is being improved. Applications of discontinua analyses of large rock blocks sliding on planes of weakness are also under study. The accuracy of the analytical approach is dependent upon a knowledge of slope material properties and of the geometry of potential failure surfaces.

In the empirical approach, a prediction of the performance of a crater slope is based largely upon empirical data relative to the observed performance and characteristics of comparable man-made or natural slopes. To use the empirical approach, sufficient site information must be collected so that the material in which an excavation is planned may be characterized for comparison with observations and data which have been previously gathered and catalogued in other locations. The primary advantage of this approach is that it can circumvent a lack of detailed site data and crater slope properties. The principal disadvantages are that safety factors cannot be determined with any precision and that the flexibility of application to slope stability analyses of craters in varying materials is limited.

In conclusion, the assessment of stability is an involved process which, owing to the large number of factors which influence stability and the possible modes of failure, requires application of sound engineering judgment. Although craters in rock appear to be stable, the reduction in strength of clay shales caused by displacement indicates that explosive excavations in these materials may not be stable.

b. Compressibility and Settlement

Knowledge of the settlement characteristics of the fallback is required to determine the suitability of fallback materials as a foundation for such engineering structures as road and railroad beds. This knowledge is also needed to determine the potential reduction in the total height of an embankment and the subsequent effect on the integrity of any impermeable surface material.

There has been no extensive documentation of either the measured settlement or compressibility characteristics of crater fallback materials. Limited observations have been made, and it is believed that the general pattern of behavior for cohesionless fallback is similar to that experienced for crushed rock and dumped rock-fill materials.

The major portion of the settlement will occur immediately following deposition of the fallback in the crater. Settlement of the fallback with only the passage of time will be slow. Increased saturation or loading (particularly dynamic) can initiate a marked and sudden increase in settlement. The degree and extent of settlement will vary with the weather and geological location of the excavation. In areas of high precipitation, surface water can saturate the excavated slopes and accelerate crushing of rock points and induce larger settlement. The crushed rock would then continue to settle at a decreasing rate. Settlement will tend to steepen the fallback slopes, but also will tend to increase the density of the materials. The former effect is detrimental to stability but the latter is beneficial with a resulting increase in shear strength.

Analytical methods of assessing the magnitude of potential settlement involve determining the compressibility characteristics of crater fallback material, determining the stresses involved, and applying the stresses to the compressibility characteristics. Techniques for laboratory measurements of compressibility of crushed rock have recently been developed and research to extend this work is continuing.[11] Most of the information presently available on the settlement characteristics of large masses of rock aggregate is based on empirical data. Compression tests have shown that compressibility (1) increases with decreasing rock hardness, (2) is greater for angular, rough-surfaced particles than for rounded, smooth-surfaced particles, and (3) increases with decreasing uniformity of particle sizes.[12]

For the construction of an expedient (blasted-into-place) dam, the magnitude of total settlement may be estimated by the equation[12]:

$$S = 6.8 \times 10^{-5} H^{1.84}, \tag{33}$$

where

S = the settlement in feet

H = height of the dam embankment in feet.

If a need develops to evaluate the compressibility of the rupture zone for foundations of such projects as bridges or spillways, the standard methods of testing in situ rock masses can be used. These techniques, including the plate jack test, pressure chamber tests, and borehole deformation tests, are discussed in Ref. 13. The compressibility of the rupture zone will be greater than that of the in situ rock mass because of the blast-induced fracturing.

c. Permeability and Seepage

Seepage and permeability characteristics of the excavation are a function of the local geology as well as of the disruptive effects of the explosion. The increase in permeability of the medium from the detonation can be expected to reduce the buildup of seepage and pore-water pressures in most materials.

Permeability determines the ease of drainage and thus is an important factor in assessing long-term stability of excavations, settlement, and drainage requirements. Direct measurements of the permeability of fallback and ejecta materials have not been made in conjunction with postshot investigations of test craters to date. Approximate values of the porosity and the permeability of various materials are shown in Table 17. The only recorded values of effective porosity are for basalt, but research is continuing to expand this information for possible future correlation with permeability.

The relationship between porosity and permeability remains a subject of active research. It has been shown that an increase in grain size increases permeability, yet has no effect on porosity in

Table 17. Approximate values for porosity and permeability of various materials.

Type of material	Porosity (%)	Effective porosity[a] (%)	Permeability,[b] k(cm/sec) at 20°C
Intact rock (laboratory samples)	2-29	—	10^{-10} - 10^{-3}
Clay	37-84	—	10^{-10} - 10^{-7}
Silt	50-54	—	10^{-7} - 10^{-6}
Fine sand	44-49	—	10^{-6} - 10^{-3}
In situ-fractured rock	<1-43	—	10^{-4} - 10^{-2}
Saturated clay shale	30-40	—	—
Dry rhyolite	2-26	—	—
Dry basalt	<1-43	<1 - 10	—
Clean sand	26-53	—	10^{-3} - 1
Crushed rock	37-44	—	1 - 10^{2}
Gravel	24-44	—	1 - 10^{2}
Rock-fill dams	27-32	—	—
Fallback	30-62[c]	—	10^{1} - 10^{4}
Saturated clay shale (BF = 1.2)	42-50[c]	—	10^{1} - 10^{3}[d]
Dry rhyolite (BF = 1.4)	30-47[c]	—	10^{1} - 10^{3}
Dry basalt (BF = 1.5)	33-62[c]	—	10^{3} - 10^{4}
Rupture zone	>*In situ* value	>*In situ* value	*In situ* value - fallback value
Dry basalt	—	<1 - 25	—

[a]For jointed rock only. Measured by borehole camera technique.[14] The wide range for *in situ* rock is the result of the variations of the media. The wide range in the rupture zone is due to the fact that the effective porosity in the rupture zone varies spatially from intact rock to rubble zone as noted on Fig. 11.

[b]Values of permeability lower than 10^{-7} cm/sec are generally considered to be characteristic of impermeable material (e.g., homogeneous clays below the zone of weathering; unfractured crystalline rock); materials with values higher than 10^{3} cm/sec are considered to be free-draining (clean gravels or coarse sands, etc.). Many, perhaps most, materials have permeabilities falling somewhere between these values, and thus range between impermeable and free-flowing in their behavior.

[c]Initial porosity plus increased porosity based on bulking factor (Fig. 38).

[d]Initial permeability; this will probably decrease with time as the material slakes and disintegrates.

theory and, in fact, decreases porosity.[15] Particle shape is an important factor in both porosity and permeability. The compaction of soils decreases permeability. Hydraulic head conditions will cause flows to be either laminar or turbulent, also influencing permeability. An increase in surcharge will reduce cracks and fissures and thus reduce both porosity and permeability. This effect is reflected by reduced porosity with depth. A relation between porosity and permeability is yet to be established and may never be.

Permeabilities have been estimated from grain size curves, porosity, and certain empirically established relationships. In general, the permeability of the fallback is orders of magnitude larger than that of the preshot medium. Prior to the detonation in a relatively impervious rock, water flow, if any, is along fractures. Both the porosity and the permeability are increased significantly by the increased fracturing caused by the detonation. It is a logical assumption that the permeability of the rupture zone will approach that of the fallback near the true crater boundary, and that rupture zone permeability will decrease with distance from the crater in a manner similar to the decrease in fracturing and effective porosity (see Fig. 11).

Seepage data are required for all types of slope-stability assessments and for the analysis of the flow-through characteristics of dam embankments. The state of knowledge of crater seepage pressures is very limited.[16] This subject is now being investigated.

On the basis of the permeabilities of the various crater zones and investigations of craters in clay shale, it is predicted that the groundwater surface for craters in rock will be depressed below its initial position. Such a depressed water table would be favorable to the potential stability of slopes. Flow from the water table into the crater, however, could cause a drawdown condition to develop adjacent to the rupture zone. The water in or below the apparent crater will eventually reach an equilibrium level, with inflow from precipitation or by seepage from the surrounding area being equaled by outflow and evaporation. If the crater is composed of rocks that are susceptible to rapid disintegration due to weathering, the gradual breakdown of the rocks may, in time, fill the voids in the fallback and rupture zones. If such filling material is not washed out, permeability will be decreased and the level of the groundwater surface may rise.

Research is now directed at establishing relationships between particle size distribution and permeability for crater rubble and between effective porosity and permeability in the rupture zone.

8.4 POSTSHOT CONSTRUCTION

The crater configuration will determine whether grading, leveling, or further earth moving is necessary to meet the design specifications. The analyses of seepage, settlement, and slope stability will determine whether compaction or slope stabilization measures are necessary.

The amount of construction required to complete the excavation depends on the type of project. Projects such as irrigation ditches may require no postshot construction at all. On the other hand, projects such as highway cuts may require work on the entire crater to prepare the roadbed and to stabilize the side slopes. Those portions of a crater which require improvement to convert it into a useful facility for various project applications are discussed below.

a. Canals

The navigational prism of a canal must be maintained for the useful life of the facility. Some dredging may be required initially to further shape the bottom and to remove rubble mounds. Changes in the geometry of the canal affect the navigational prism.

b. Watercourses

Little, if any, construction is necessary to improve the hydraulic characteristics of the channel for water movement. It is suggested that the flow be allowed to seek its own bedslope and to develop its own channel geometry. Slumping of the side slopes and changes in the channel width and depth may occur in certain materials as the channel attains equilibrium with the flow and the sediment load. Should there be a requirement for the installation of concrete or bituminous linings, extensive shaping, forming, and construction work would be necessary.

c. Harbors

By proper excavation design, the postshot construction necessary to complete the navigational prism of the channel and the turning basin will be limited to dredging to remove the crater lips if they are a hazard to navigation. The use of explosive charges should be considered as an alternative to dredging.

Most of the postshot construction will be directed toward preparing the area for harbor facilities. Sections of the lip and slopes of the explosively excavated harbor basin will have to be leveled, graded, and compacted to provide a solid and stable foundation for harbor structures. In addition, the harbor will usually require storm protection. Rubble produced by the excavation could be a good source of armor stone and aggregate for construction of such protective structures as breakwaters, seawalls, and revetments.

d. Channel Improvements

Little or no additional postshot effort is required on projects of this nature. Some ejecta removal along the adjacent shoreline may be required.

e. Highway and Railway Cuts

Preparation of the roadbed foundation and stabilization of slopes and drainage are the major area of concern. Careful engineering must be exercised to assure minimum maintenance and repair after completion.

The fallback and ejecta could be used as subgrade material provided engineering specifications are met. If the fallback is unsuitable for subgrade material, the crater bottom has to be cleared and leveled to the required elevation. This foundation for the roadbed may need compaction to prevent excessive settlement, and measures will be necessary to provide adequate drainage.

The slopes of the cut may be stabilized to prevent surface ravelling and mass slippage along a plane of weakness. Wire mesh or grout can be used to prevent individual rocks from rolling downslope. The slopes could be flattened at locations where the postshot investigation indicates that mass slippage is probable.

f. Quarries and Rock Fracturing

The most important design criteria in developing a quarry with explosives are the depth of emplacement and the siting on sloping terrain in order to facilitate recovery of the rock. Because of variations in rock media, particle gradation will vary and secondary blasting of some of the resulting blocks may be necessary. It may also be necessary to clear boulders and ejecta that fall in planned hauling areas. In order to completely develop the quarry, a road net for access to the site must be constructed.

g. Expedient (Blasted-into-Place) Dams

Ejecta deposited across a streambed could be the basic material for a dam. The ejecta deposit may be reworked into a configuration better suited for a more permanent facility. Some type of impermeable embankment surface could be applied during postshot construction activities to prevent seepage through the embankment. The practicality of a dam of this nature is dependent on the in situ rock properties and the postshot construction requirements.

h. Overburden Removal

The extent of postshot work in this case is dependent on the intended use. If the blast has exposed valuable mineral deposits, access roads must be cleared and normal mining operations set up. If quarrying is the objective, then follow-on quarry development must be considered. In either case ejecta and fallback must be cleared or moved to facilitate operations. Use of the fallback and ejecta as fill material should be considered.

8.5 SUMMARY

The methods and procedures involved in postshot evaluation and the amount of additional construction required after an explosive excavation are dependent on the nature of the material and the proposed use of the excavation. Not all of the investigative procedures and techniques outlined in this chapter can or need be used on each project. A determination of the required data must be made, methods of data collection must be devised, and knowledgeable individuals must be available to interpret the meaning of the data as it applies to performance. The principles of soil and rock mechanics, engineering experience, and judgment are the essential elements in predicting the behavior of an excavation and the amount of postshot remedial work that may be necessary. Postshot construction methods are similar to those applied to projects accomplished entirely by conventional means.

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Chapter 9

Project Costs

9.1 SCOPE

This chapter discusses the elements of cost of an explosive excavation project. The principal elements are listed in Table 18. Only cursory treatment is given to cost elements encountered by engineering and construction organizations in everyday practice. Cost elements that are less common or are unique to explosive excavation are covered in more detail. These include emplacement construction, explosives, field operations, and possible damage costs from seismic motions and airblast. The influence of the most significant cost elements is illustrated by the example analysis in Chapter 10.

9.2 PRECONSTRUCTION PLANNING, ENGINEERING, AND DESIGN

Preconstruction planning, engineering, and design costs for an explosive excavation project should be about the same as those estimated for an excavation project accomplished by conventional means. Site calibration detonations, however, are unique to explosive excavation projects. The site data (described in Chapter 3) needed for explosive excavation design are generally of the same type as those required for planning and designing other methods of excavation. However, if the project planning agency has little or no experience in the explosive excavation field, it may be desirable during the planning, engineering, and design processes

Table 18. Elements of cost.

Principal element	Discussed in
1. Project planning costs	
a. Preconstruction planning, engineering and design	(Section 9.2)
(1). Site investigations and explosive excavation calibration	
(2). Design and preparation of plans and specification	
b. Land acquisition and easements	(Section 9.3)
2. Project execution costs	
a. Mobilization, camps, and access roads	(Section 9.4)
b. Explosives and emplacement construction	(Section 9.5)
c. Explosive excavation field operations	(Section 9.6)
d. Safety measures, claims, reparations and damage repair	(Section 9.7)
e. Project design modifications and field changes	(Section 9.8)
f. Contingencies, supervision, and administration	(Section 9.9)

to consult with persons more experienced in explosive excavation and explosions effects. If the project is near built up areas, data regarding the response of structures to seismic motions and airblast overpressure are especially important. Qualified persons may be found by contacting the U.S. Army Engineer Nuclear Cratering Group (NCG),* manufacturers of explosives, and organizations that have provided contractual explosive and seismic analysis services to NCG and the Atomic Energy Commission.

Explosive excavation projects in geologic media which are significantly different from those media for which cratering data have been developed may require site calibration cratering tests to develop cratering characteristics for design. Generally, however, sufficient data will be available. Measurements of seismic and airblast effects and structure response from small-scale cratering tests would help to establish the upper limit of explosive quantities that can be safely detonated simultaneously at the site.

The size of the charges required for calibration explosions would be determined by the anticipated charge size for the required excavation and by the spacing of natural fractures in the medium. Strong, massive materials would produce large block sizes and would require larger explosions to develop crater dimensions than would weaker or more fractured materials. The charge weights for calibration tests may range from 1,000 lb to as much as 10 tons. The cost of such a series, based upon NCG experience, would range from $10,000 to $50,000.

*To be redesignated the U.S. Army Engineer Explosive Excavation Research Office on 1 July 1971.

9.3 LAND ACQUISITION AND EASEMENTS

Land acquisition and easement costs for an explosive excavation project normally would be the same as those estimated for the project were other means of excavation used along the same alinement. In certain cases, however, the effects of the explosions, especially ejecta distribution and ground shock, should be considered in determining whether additional land should be acquired. This could be determined from a preliminary design of the explosive excavation.

9.4 MOBILIZATION, CAMPS, AND ACCESS ROADS

Emplacement construction equipment requirements should be considered in determining mobilization and access road construction costs. The effects of the explosions should be considered in locating construction camps. Otherwise, mobilization, construction camp, and access road construction costs would normally be the same as if other means of excavation were used.

9.5 EXPLOSIVE AND EMPLACEMENT CONSTRUCTION

Explosive and emplacement construction costs are interrelated. The less dense explosives or blasting agents are usually less expensive, but they require larger and consequently more costly emplacement cavities. Explosive selection, therefore, not only involves choosing an explosive that is an efficient cratering explosive for the geologic medium and is compatible with site conditions, but also one for which the combined cost of emplacement construction and the explosive itself is minimized. Subsequent paragraphs present information to assist in selecting the explosive and in estimating emplacement construction costs.

a. <u>Explosive Costs</u>

Whenever possible, explosives cost data should be obtained from manufacturers. Preferably the cost data should be "down-hole" costs to include shipping, emplacing, arming, and firing services. Table 19 gives explosives data extracted from Chapter 4. Emplacing, arming, and firing costs are estimated to be $80 to $120 per ton based on previous experimental explosive excavation work.

An explosive should be selected which is suitable for the geologic medium involved and which will minimize the combined cost of emplacement construction and explosive. With the data obtained from manufacturers, the explosive selection process may be narrowed by the calculation of certain parameters and their arrangement as illustrated in Table 20. In Table 20 maximum cratering volume effectiveness and mid-range unit explosive cost for each explosive shown in Table 19 have been assumed as applicable data. Explosives costs include $10/ton for 400-mi transportation from factory to site and $80/ton for explosive services. The explosives are compared on the basis

Table 19. Explosives data.

Explosive composition	Specific gravity	Density (ρ) (lb/ft^3)	Cratering volume[a] effectiveness in relation to TNT, E	Cost, c ($/ton) Delivered to site[b]	"Down-hole"[b,c]
ANFO[d]	0.93	58	1.0-1.1	40-200	120-320
2% Al-AN slurry[e] (blasting agent)[f]	1.30	81	1.0-1.2	60-260	140-380
AN slurry	1.40	87	1.0-1.2	200-400	280-520
8% Al-AN slurry (blasting agent)	1.33	83	1.2-1.4	160-360	240-480
20% Al-AN slurry (blasting agent)	1.20	75	1.5-1.7	260-540	340-660
35% Al-AN slurry (blasting agent)	1.50	94	1.6-1.8	300-700	380-820

[a]Based on small-scale tests in sand.

[b]Add $25 per 1000 ton-mi for overland transportation from explosives manufacturing plant to site.

[c]Allows $80 to $120 per ton for emplacing, arming, and firing.

[d]Not suitable for use in wet environment.

[e]All slurries are suitable for use in a wet environment.

[f]Blasting agents have no high-explosive component.

Table 20. Illustrative example — explosives selection.

Explosive composition	Specific gravity	Density (ρ) (lb/ft^3)	Cratering volume[a] effectiveness in relation to TNT, E	Assumed[b] "down-hole" cost/ton, c ($/ton)	Required cavity volume ($2000/\rho E$) (ft^3/ton TNT or equivalent)	"Down-hole"[c] cost[b] (c/E) ($/ton TNT or equivalent)
ANFO[d]	0.93	58	1.1	210	31.35	191
2% Al-AN slurry[e] (blasting agent)[f]	1.30	81	1.2	250	20.57	208
AN slurry	1.40	87	1.2	390	19.16	325
8% Al-AN slurry (blasting agent)	1.33	83	1.4	350	17.12	250
20% Al-AN slurry (blasting agent)	1.20	75	1.7	490	15.69	288
35% Al-AN slurry	1.50	94	1.8	590	11.82	328

[a]Based on small-scale tests in sand

[b]Includes $10/ton for 400-mi overland transportation of explosives at $25 per 1000 ton-mi, and $80/ton for emplacing, arming, and firing.

[c]Includes cost of transporting, emplacing, arming, and firing of explosives.

[d]Not suitable for use in wet environment.

[e]All slurries are suitable for use in a wet environment.

[f]Blasting agents have no high-explosive component.

of required cavity volume and cost per ton TNT equivalent cratering power. Ideally, both the required cavity volume and the cost per ton TNT should be minimized. The explosives are listed from top to bottom of the table in order of decreasing cavity volume per ton TNT equivalent. That order is the same order as decreasing emplacement drilling costs at the same site for geometrically similar shapes of charges of equal TNT cratering power. Thus, any explosive having a higher unit cost per ton TNT equivalent than one below it may be eliminated from further consideration for the project because it would result in both higher drilling costs (larger cavity required) and higher explosives costs than the one below it. Table 20 shows that, for the basic data assumed, use of AN slurry would result in higher combined drilling and explosives cost than use of an 8% Al-AN agent.

After such a preliminary elimination process, combined emplacement construction and explosive costs are now determined for the remaining explosives starting with the one listed nearest the top of the table. Emplacement drilling costs are discussed in Section 9.5b, below. Comparisons can be made by determining the combined cost for one emplaced charge. One continues down the table as long as the combined cost decreases, but, as soon as the combined cost increases, use of the preceding explosive considered would result in the least cost for explosives and emplacement construction. In some cases, however, the detonation impedance, the nature of the geologic medium, and/or operational requirements in emplacing the explosive may dictate that a more expensive explosive be used. For instance, ANFO is not suitable for use in a wet environment unless it is adequately encased. Also it is generally desirable to keep the charge dimensions small enough so that costly reinforcement of the cavity and/or casing will not be required.

The total cost of the selected explosive is:

$$\text{Explosives cost} = \left(\frac{Y_t}{E}\right) c = (Y_t)\left(\frac{c}{E}\right). \qquad (34)$$

In Eq. (34), Y_t is the total required quantity of explosives in tons TNT equivalent taken from the design, E is the cratering volume effectiveness of the selected explosive, c is the cost per ton of the selected explosive, and c/E is the cost per ton TNT equivalent of the selected explosive taken from the explosive selection table.

b. Emplacement Construction Cost

Emplacement construction will normally involve drilling. Drilling costs vary with hole size, charge configuration, and strength of the material to be drilled. In areas of deep overburden it may be necessary to install surface casing to preserve the emplacement hole. Casing might also be required for projects involving underwater excavation. The cavity dimensions required for various sizes of cylindrical charges may be determined from Fig. 39 in which cavity diameters for various ratios of charge length to diameter (h/d)* are plotted as functions of explosive charge volume. The explosive charge volume is the product of the charge size (Y) in tons TNT equivalent and charge volume per ton TNT equivalent, $2000/\rho E$, taken from the explosive selection table discussed above. The use of Fig. 39 is illustrated in Chapter 10.

After emplacement hole dimensions and casing requirements are determined, emplacement construction costs can be estimated. Local drilling and casing cost data should be used when available. Table 21 gives estimated emplacement hole unit drilling costs developed from data in NCG reports.[1,2] Figure 40 gives estimated casing costs. Depth of drilling in materials of various strengths are determined from the explosive excavation design profile with each emplacement cavity extending one-third the charge length [0.33(h/d)d] below the designed location of the center of detonation. Costs also should be included for stemming the access holes to the emplacement cavities with a free-flowing material that is carefully tamped as it is placed.

9.6 EXPLOSIVE EXCAVATION FIELD OPERATIONS

Explosive excavation field operations are discussed in Chapter 7. Those cost elements related to emplacement construction, explosives procurement, explosives services, and stemming are covered in the preceding paragraph. Cost considerations regarding detonation hazards are discussed in a subsequent paragraph on safety measures. This portion of the chapter deals with other field operations elements that may influence explosive excavation costs.

As discussed in Chapter 7, it may be necessary to provide a cleared, secure

*The letter "h" (connoting height) is used in this ratio to avoid confusion of the letter "l" with the numeral "1."

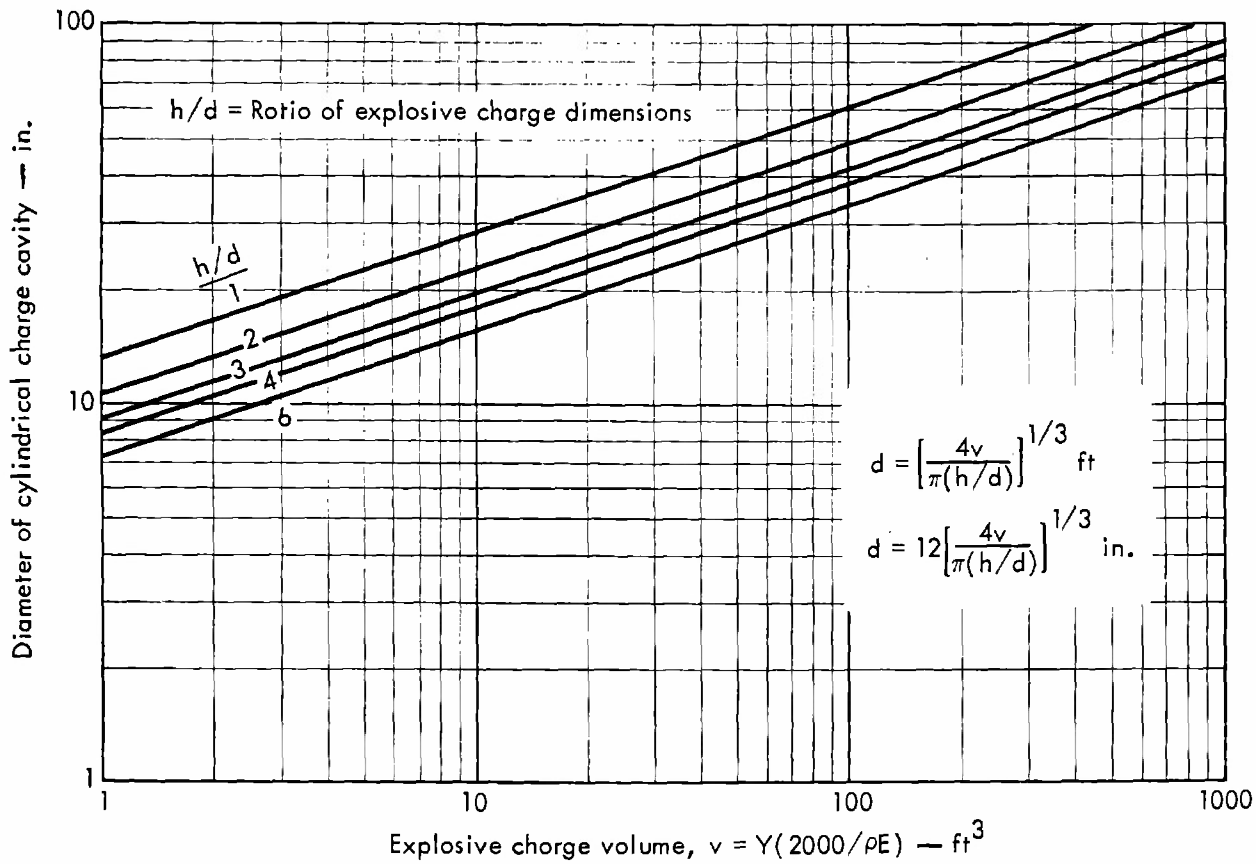

Fig. 39. Cylindrical charge cavity dimensions.

Table 21. Estimated drilling costs.

Material classification	Unconfined compressive strength (psi)	Rig type	Unit drilling costs for various hole sizes[a] ($/ft) Diameter (in.) 12	24	36	48	60	72
High-strength rock	>16,000	Drive-in rotary[b]	15-30	25-40	35-80	50-100	70-140	95-185
Intermediate-strength rock	4,000 to 16,000	Drive-in rotary[b]	8-20	15-30	25-45	35-60	45-80	70-140
Weak rock	1,000 to 4,000	Drive-in rotary[b]	5-10	8-15	10-30	20-40	30-45	35-70
Common excavation	<1,000	Drive-in auger[c]	<3	—	<4	<5	<6	<7

[a]Data are based on drilling at least 300 linear ft of hole. Rig (24-hr day-rate), cutter and drilling fluid costs are included, but no allowance is included for loss of circulation fluid. Rig day-rate includes equipment rental, profit, overhead, depreciation and crew salaries. Cost of mobilization, demobilization, site preparation, rig set up, and rig tear down are not included.

[b]Mobilization and demobilization costs about $1.20 per mi plus rig standby costs during travel time (site preparation costs about $1500 to $2500).

[c]Mobilization and demobilization costs about $0.70 per mi plus rig standby costs during travel time (site preparation costs considered negligible).

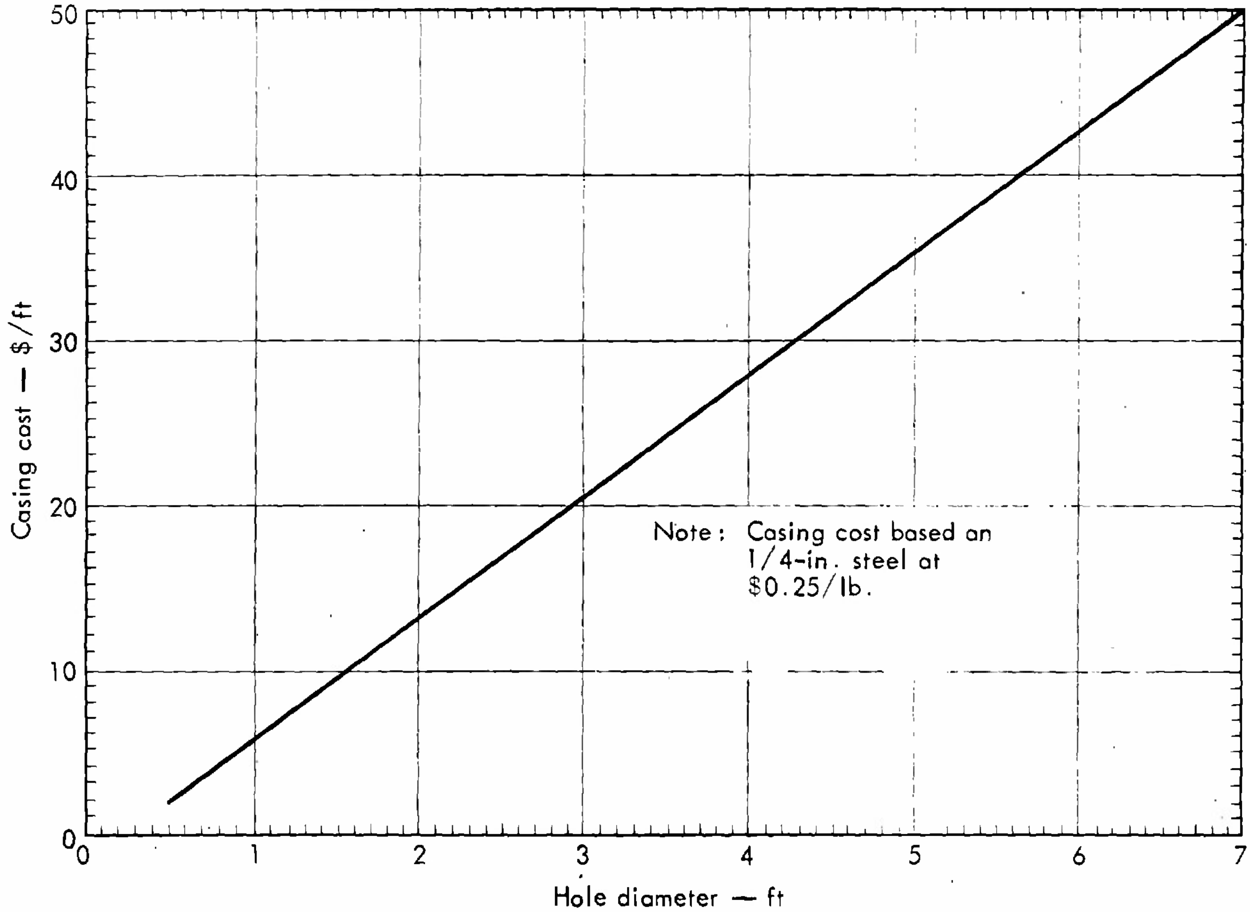

Fig. 40. Cost of casing.

temporary storage area for explosives and explosive accessories. An area of this type would probably also be required if the project were done conventionally; however, it might have to be larger if explosive excavation is used. Minimum safe separation distances can be determined from the "Applicable Publications" listed in Appendix F. Buried 55-gal drums may be satisfactory for separate lockable bunkers for initiators and boosters. The work space for assembly of components should be sheltered and be a minimum of 60 ft^2.

Chapter 7 discussed the need and location requirements for a Control Point (CP). The facility should be a minimum of 220 ft^2 and may be housed in an existing or portable building or trailer. Communications should include access to commercial telephone systems and radio or wire access to all critical points. Usually, the radio network would consist of a base station at the CP, five to eight mobile or fixed stations, and four to eight portable units. If not available within the construction organization, radios can be obtained on a rental basis.

In addition, water supply, electric power, sanitary facilities and other logistical support are required to meet construction and personnel needs. The extent of such support will depend upon the project and its location. This support is similar to that required for a conventional construction project. Documentary still and motion photography would be provided as desired.

9.7 SAFETY MEASURES, CLAIMS, REPARATIONS, AND DAMAGE REPAIRS

Each explosive excavation design should be checked for safety in regard to detonation effects. This portion of the chapter provides procedures for estimating seismic and airblast damage costs. It also discusses the validity of the data, its use in determining how to proceed with explosive excavation planning, and cost considerations involved in the process.

For preliminary safety analyses, an assessment of potential seismic and airblast damage and damage-related costs should be sufficient. Figures 41 and 42, which combine several of the safety analysis steps given in Chapter 6, are provided to assist the engineer in these preliminary analyses. To use the figures,

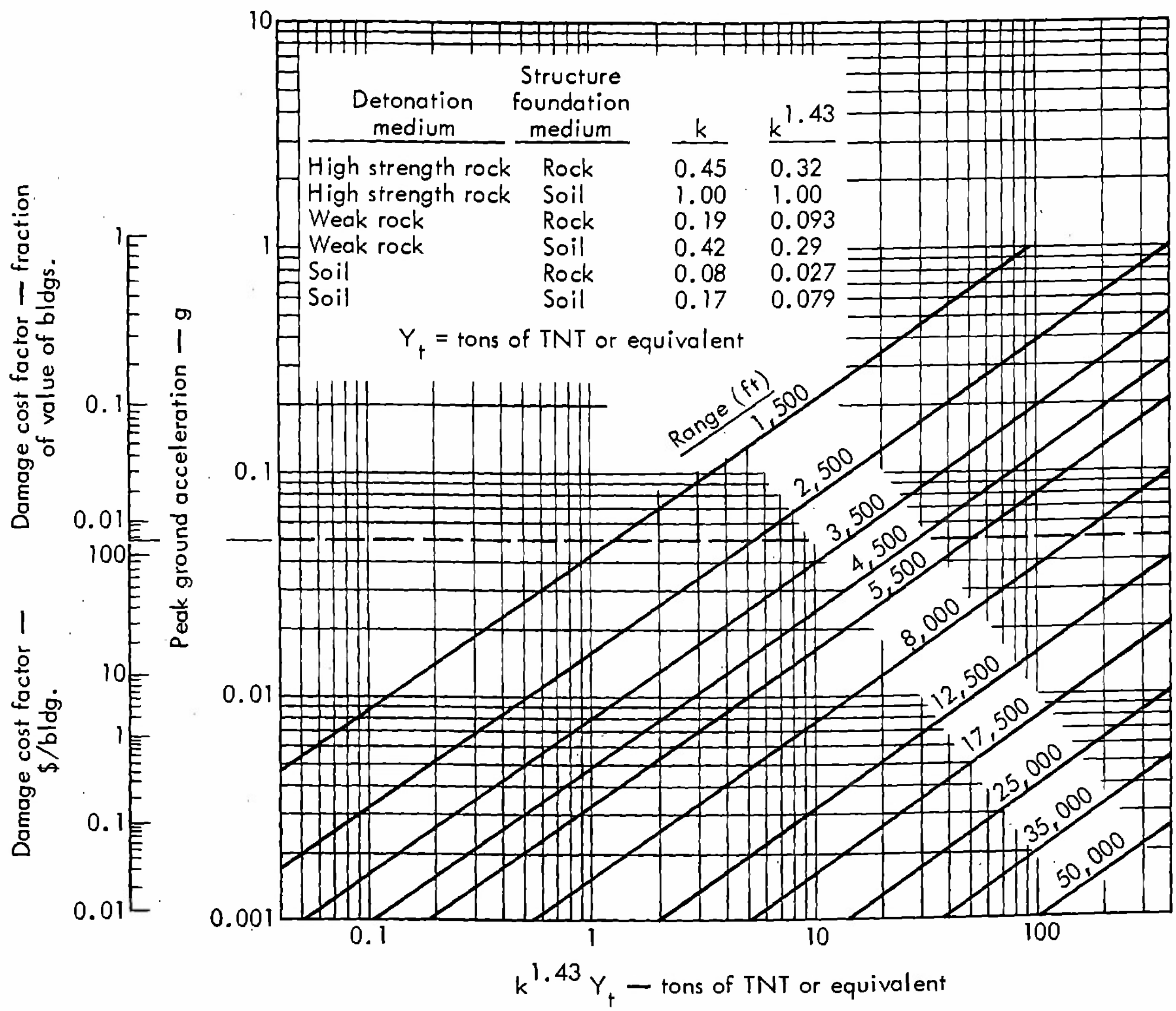

Fig. 41. Ground shock prediction data.

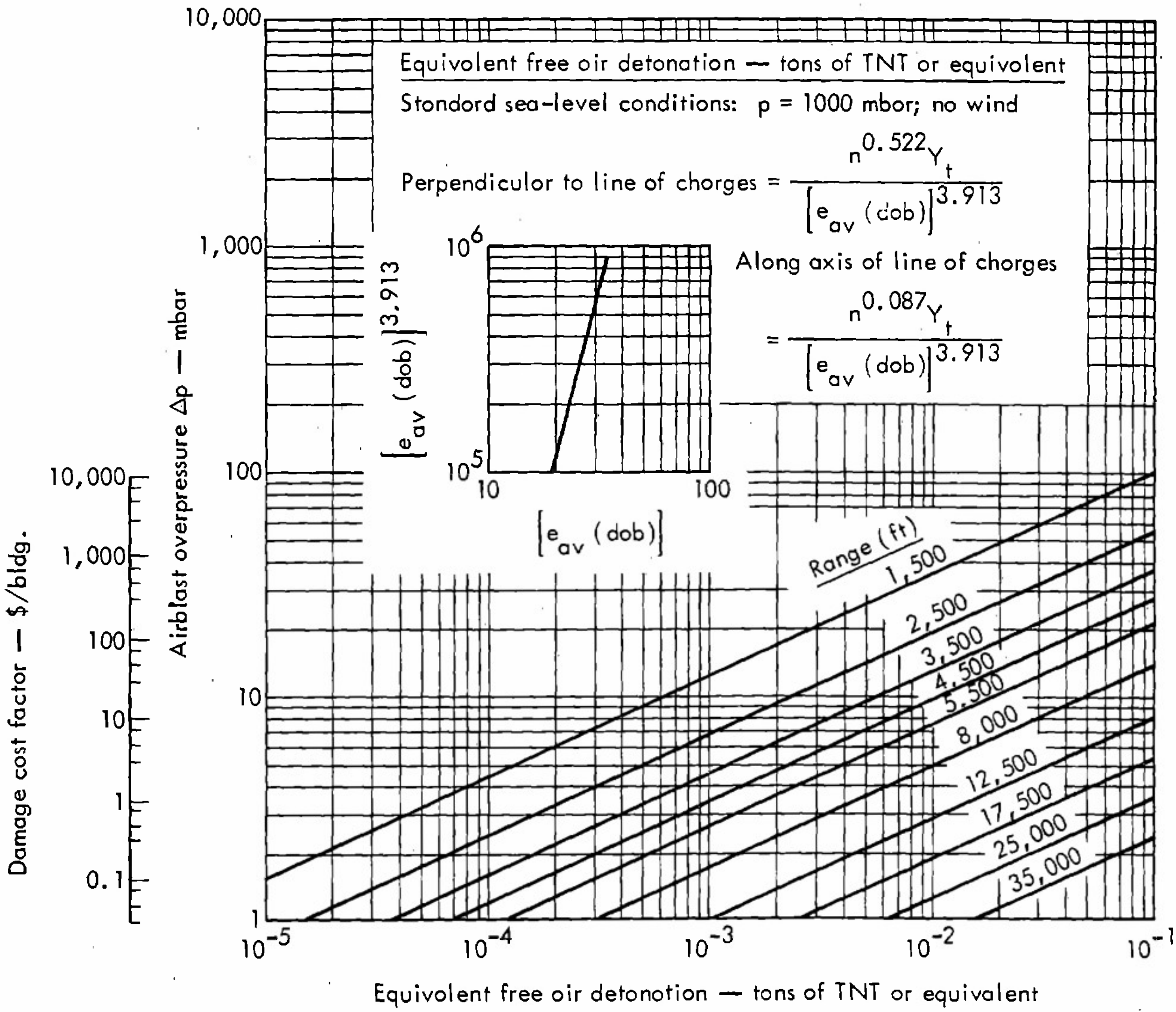

Fig. 42. Airblast prediction data.

one must calculate the following three quantities:

$Y_t k^{1.43}$ = Equivalent total detonation charge for detonation in high strength rock and structures on soil

$\frac{Y_t n^{0.522}}{[e_{av}(dob)]^{3.913}}$ = Equivalent free air detonation for airblast effects perpendicular to the line of charges

and

$\frac{Y_t n^{0.087}}{[e_{av}(dob)]^{3.913}}$ = Equivalent free air detonation for airblast effects along the axis of the line of charges

in which

Y_t = total weight of explosives detonated in tons TNT equivalent

k = factor from Figs. 41 or 26

n = number of charges in the detonation

dob = depth of burial for a 1-ton TNT charge

e_{av} = average enhancement of single-charge crater dimensions used in the design

and

$$e = \left[\frac{Y_{e=1}}{Y_r}\right]^{0.3} = \frac{DOB}{(DOB)_{e=1}}$$

$$= \frac{R_a}{(R_a)_{e=1}} = \frac{D}{D_{e=1}}.$$

Using $Y_t k^{1.43}$ as the abscissa, one can read directly from Fig. 41 the estimated ground acceleration and seismic damage cost factor per building at various average radii of annular zones surrounding the detonation. Using

$$\frac{Y_t n^{0.522}}{[e_{av}(dob)]^{3.913}} \text{ and } \frac{Y_t n^{0.087}}{[e_{av}(dob)]^{3.913}}$$

as abscissas, one can read directly from Fig. 42 the estimated overpressure, and airblast damage cost factor per building at various average ranges from the detonation. The inset on that figure can be used to determine $[e_{av}(dob)]^{3.913}$. The example analysis in Chapter 10 illustrates the use of these figures.

The damage cost factors determined from Figs. 41 and 42 cannot be used to predict the damage to any one building. At a given distance from the detonation, one building may be damaged by ground shock, airblast, or both, whereas another may experience no damage. The factors are statistical in nature and are more applicable for estimating total damage costs for a group of buildings. The total estimated damage cost for a particular zone is the damage cost factor at the average range of the zone times the number of buildings in the zone except for damage from accelerations greater than 0.05 g. This case is explained below. If the number of buildings is unknown, an estimate can be based on the population and the average number of inhabitants per residential type unit. The damage cost factors include claims processing costs.

The damage cost factors in Fig. 41 were developed from data previously reported by NCG.[3] They are fixed values for architectural damage for accelerations less than 0.05 g. For accelerations greater than 0.05 g, the damage cost factors are expressed as a fraction of the value of the buildings. Also, the factors are not applicable for estimating damage to engineered structures, such as high-rise buildings, dams, bridges, etc. Estimates of susceptibility of engineered structures to damage may require the expertise of a structural engineer.

The damage cost factors in Fig. 42 were developed from empirical data resulting from investigation of an accidental explosion of 56 tons of high explosive at the Medina facility in San Antonio, Texas on 13 November 1963.[4,5] The overpressure values are for standard atmospheric conditions. The cost factors include about a 40% allowance for damage to bric-a-brac. The figure is based on data for a built-up area and should give higher estimated airblast damage costs than would be expected from explosive excavations in rural areas.

One of the objectives of explosive excavation design is to minimize damage and damage related costs. If the preliminary safety assessment indicates possible damage to close-in structures or total damage costs are unacceptable, the following measures may be considered:

(1) Acquire the assistance of explosive excavation, seismic motion, or airblast effects consultants.

(2) Redesign the explosive excavation so that the number of charges or the amount of explosives detonated at any one time is reduced. A sequential detonation using a 25-msec delay between 1-ton charges has been found to reduce the seismic motion and airblast effects to those of a 1-ton single charge.

(3) Protect structures susceptible to damage by taping, boarding, sandbagging or removing windows, and sandbagging or bracing buildings, historical monuments, statues, etc.

(4) Include nearby structures within the acquisition right-of-way.

(5) Protect people from broken glass, plaster, and bric-a-brac fragments and other hazards by evacuation of buildings or areas as necessary, and conduct a public information program to inform the public of the project benefits, the accompanying explosive excavation hazards, and the measures taken to minimize them.

Once the final design is developed, a detailed safety analysis should be made based on the procedures given in Chapter 6. Detailed safety measures, as discussed above, should be outlined and costed. If some damage is to be accepted, damage claim costs for unprotected buildings may have to be based on data in Figs. 41 and 42 unless better estimating criteria are available. In addition, structures susceptible to damage should be surveyed both prior to and after the detonation in support of claims investigation activities.

9.8 DESIGN MODIFICATIONS AND FIELD CHANGES

An explosive excavation project differs from other projects in that the design of postshot construction would have to be based on predicted postshot engineering properties of the material in the vicinity of the excavation or wait until results of the explosions are determined. Topographic or hydrographic surveys may be needed to determine whether remedial conventional excavation is required to convert the explosive excavation into a useful facility. Engineering properties investigations may be needed for design or verification of design assumptions for structures to be constructed in or near the excavation. Necessary modifications in the design would be made and cost estimates adjusted accordingly.

9.9 CONTINGENCIES, SUPERVISION, AND ADMINISTRATION

Contingency, supervision, and administration costs are estimated in the normal manner based on the confidence the estimator has in the basic design data or assumptions. Allowances for preshot construction would probably be the same as those used for a conventional construction project. It may be desirable, however, to use a slightly higher contingency for postshot work especially if it depends to a large degree on the resulting crater configuration and the engineering properties of the disturbed material in the various crater zones.

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4. J.W. Reed, B.J. Pape, J.E. Minor, and R.C. DeHart, "Evaluation of Window Pane Damage Intensity in San Antonio Resulting from Medina Facility Explosion on November 13, 1963," Ann. N. Y. Acad. Sci. 152, 565 (1968).
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Chapter 10

Project Analyses

10.1 SCOPE

The purpose of this chapter is to illustrate how the information and technical data contained in this report may be used to analyze the practicability of explosive excavation techniques for a particular project. This is accomplished by means of an example analysis of a hypothetical river bend cutoff project. The steps followed in analyzing the project are shown in Fig. 43. Each is discussed in turn in this chapter. The analysis of other projects would generally follow these same procedures.

10.2 PROJECT PURPOSE AND DESCRIPTION

The proposed river bend cutoff is depicted in Fig. 44. Studies have indicated that it is necessary to straighten the watercourse in this area in connection with the overall flood control plan for the river valley. The proposed explosive excavation would provide a 750-ft long, 24-ft deep cutoff channel in an excavation having a maximum depth of cut of 31 ft.

10.3 PROJECT AND SITE PARAMETERS

a. Project Requirements

The project parameters to be considered in designing an explosive excavation for a watercourse of this nature are discussed in Chapter 3. For this project, studies have indicated that a 24-ft deep, hyperbolic-shaped, unlined channel provided by a single row-charge crater would be sufficient to carry the design flow in the 2-ft drop across the cutoff. The excavation should be alined so as to straighten the watercourse as much as possible (Fig. 44). There is no requirement to dam the stream to prevent overflow into the meander that is to be cut off.

b. Site Features

A river was forced to seek a path around a basaltic flow (Fig. 44). A cutoff is to be excavated across the flow. The basalt has a density of 160 to 165 lb/ft^3, an unconfined compressive strength of about 15,000 psi, and a seismic velocity of 9,000 to 11,000 ft/sec. The flow is massive with essentially no bedding or major structural discontinuities; however, some joints exist. The water table at the project site is generally at the level of the water in the river.

Some cattle grazing takes place in the river valley, but otherwise the land within 4 mi of the project is undeveloped. The area east of the river and immediately south of the project site is State land reserved for recreational campsite development. Information about population and structures more remote from the project site is given in Section 10.6.

10.4 SITE CRATERING CHARACTERISTICS

Because large-scale high explosive cratering tests have been conducted in basalt formations similar to the basalt involved in this project, site calibration

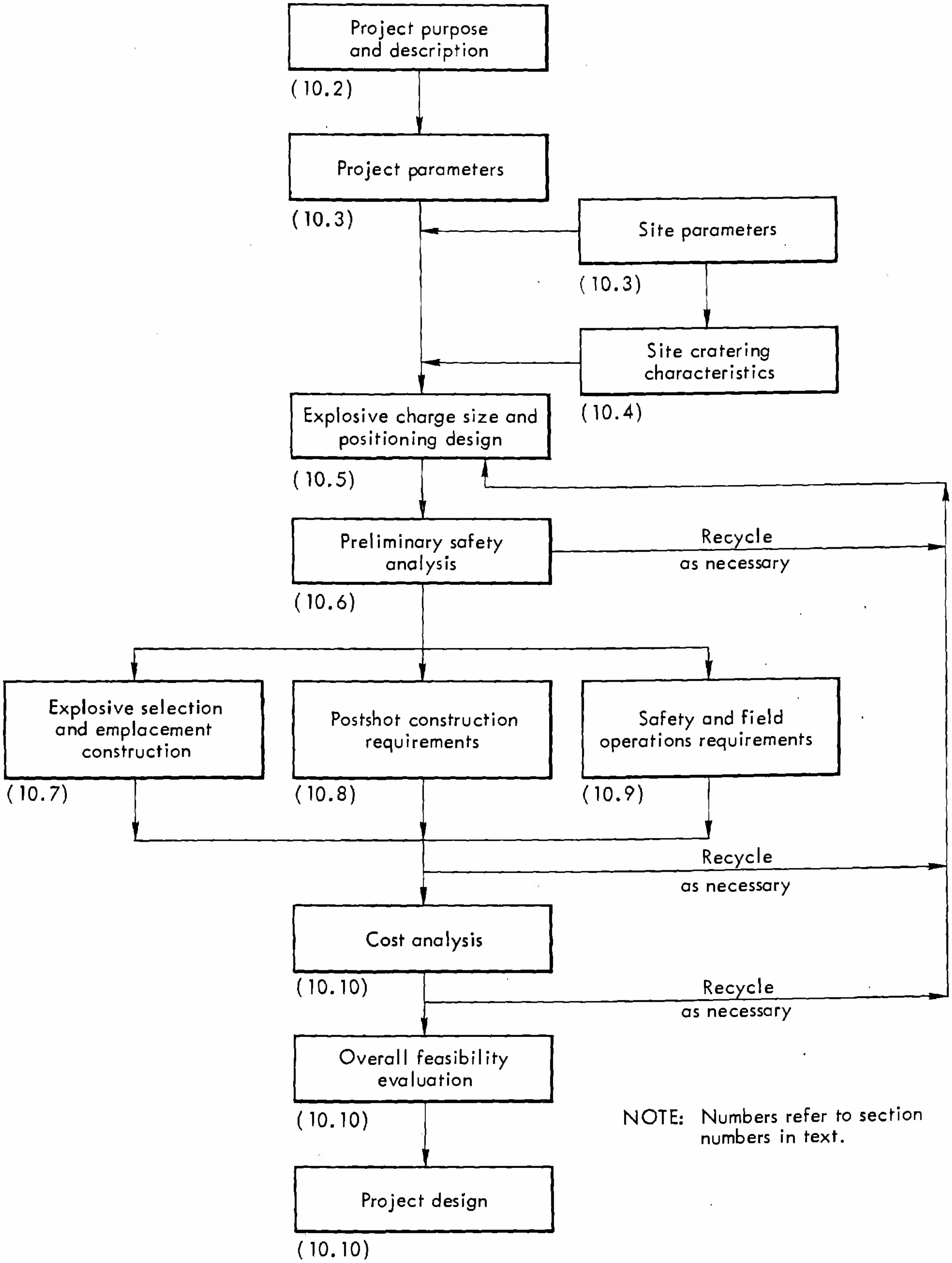

Fig. 43. Project analysis sequence.

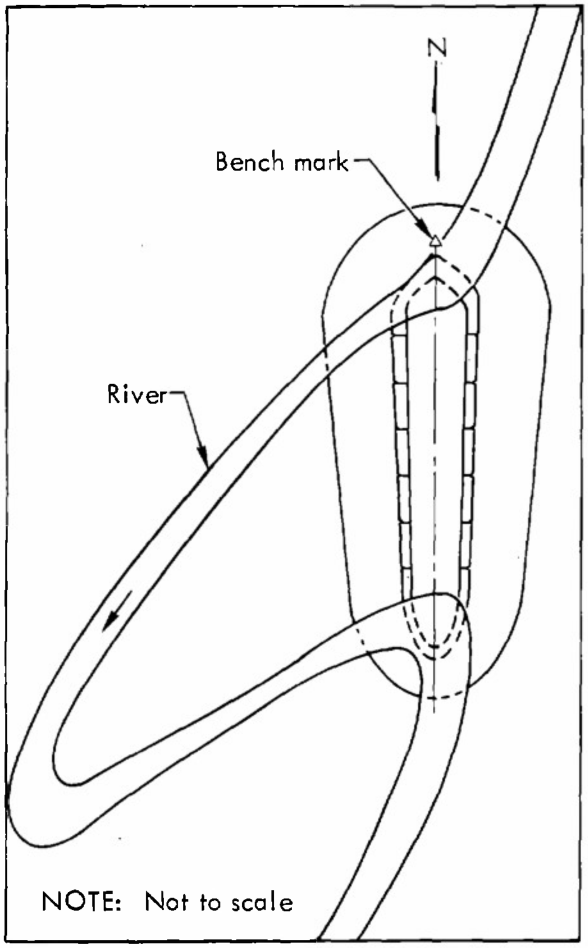

Fig. 44. Sketch of project area.

tests are not considered necessary. The data for dry rock from Fig. 17 of this report are used to design the explosive excavation.

10.5 EXPLOSIVE CHARGE SIZE AND POSITIONING

a. Profile Showing Depth of Cut

The centerline profile of the proposed alinement is shown in Fig. 45 as Line ABCDEFG. Station 0+00 is located at a bench mark established across the river at the north end of the proposed cutoff (Fig. 44), and the cutoff centerline is on an azimuth of 180° from the bench mark. The required depth of excavation is shown in Fig. 45 as Line HH'. To take into account the water overburden in accordance with Chapter 5, Section 5.6, Line IJCDEKL is the hypothetical surface used in the design of the explosive excavation.

b. Selection of Charge Size and Design Method

Unless project or safety requirements dictate otherwise, generally the larger the charge size, the more economical the explosive excavation. Thus, the availability of drilling equipment is an important aspect of the design. An investigation disclosed that several drilling contractors in a city 50 mi from the project site have equipment capable of drilling holes up to 54 in. in diameter in high strength rock. From Fig. 39, a borehole 54 in. in diameter could contain a cylindrical explosive charge of about 430 ft^3 with a height-to-diameter ratio of six, the maximum h/d considered feasible for explosive excavation at the current state of the technology. ANFO is eliminated from consideration because of the water which is present in the joints in the basalt formation. From Table 20, one ton TNT equivalent of 2% Al-AN slurry, the explosive still under consideration nearest the top of the table, occupies 20.57 ft^3. The amount of that explosive which can be emplaced in a 54-in. borehole (h/d = 6) is:

$$Y = \frac{430}{20.57} = 20.9 \text{ tons of TNT or equivalent.}$$

On this basis, 20 tons of TNT or equivalent is selected as the maximum charge weight for the preliminary design. The constant-charge-weight method of design is used because all boreholes will be of the same

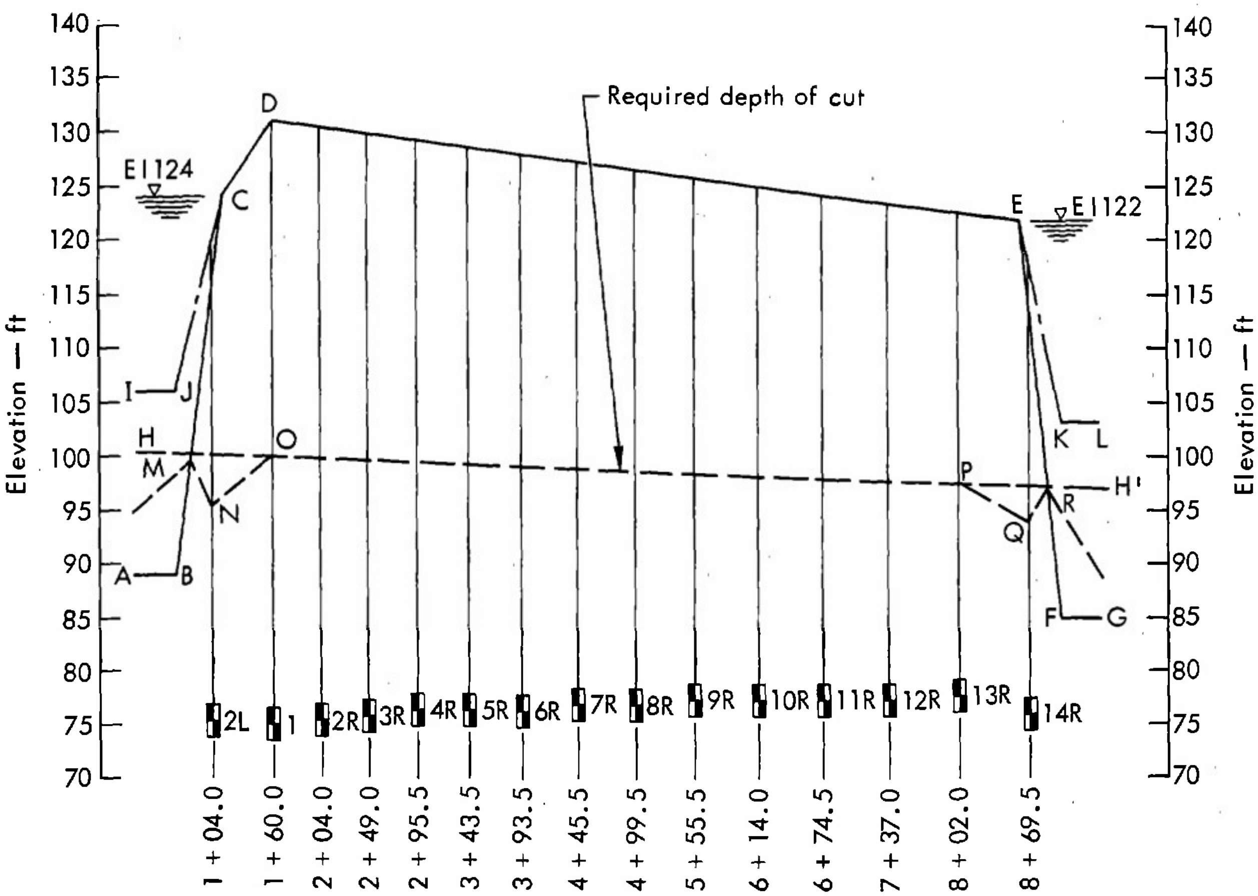

Fig. 45. Project profile showing stations, elevations, required depth of cut, and charge positioning.

diameter and because this method usually leads to a more economical solution than the constant enhancement method.

c. Constant-Charge-Weight Design

(1) Charge No. 1 at Location of Deepest Cut

The design is started by determining the charge positioning data for charge No. 1 to be placed at the cross section of maximum depth of cut (31 ft from Fig. 45) which is at station 1+60.0. From Fig. 17, a single charge of 43.5 tons TNT equivalent would be required for that depth of cut. For a 20-ton TNT equivalent charge at that point, the spacing relationship is:

$$S_1/R_{a_1} = 1.4\,(Y_r/Y_s)^{0.6}$$

$$= 1.4\,(20/43.5)^{0.6} = 0.88.$$

From Fig. 17, R_a is 49 ft, and

$$S_1 = 0.88 \times 49 = 43 \text{ ft.}$$

The 20-ton charge is located at the depth of burial (DOB) for a 43.5-ton charge which, from Fig. 17, is 56 ft.

(2) Proceeding to Right of Charge No. 1

The positioning data for charge No. 2 is determined from the depth of cut required 43 ft, S_1, to the right of charge

No.1. From Fig. 45, the depth of cut is 30.5 ft. Repeating the above procedures:

$$S_2/R_{a_2} = 1.4\,(Y_r/Y_s)^{0.6}$$

$$= 1.4\,(20/41)^{0.6} = 0.91$$

and

$$S_2 = 0.91 \times 49 = 44.5 \text{ ft.}$$

The actual spacing between Y_1 and Y_2 is:

$$S_{1-2} = (S_1 + S_2)/2 = (43 + 44.5)/2 = 44 \text{ ft.}$$

The depth of burial is 55 ft (from Fig. 17).

The depth of cut for the next charge is determined for the cross section 44.5 ft, S_2, to the right of charge No. 2. From Fig. 45, it is 30 ft. The spacing relationship is:

$$S_3/R_{a_3} = 1.4\,(Y_r/Y_s)^{0.6}$$

$$= 1.4\,(20/39)^{0.6} = 0.94$$

and

$$S_3 = 0.94 \times 49 = 46 \text{ ft.}$$

The depth of burial is 54 ft. The actual spacing is:

$$S_{2-3} = (S_2 + S_e)/2 = (44.5 + 46)/2 = 45 \text{ ft.}$$

The depth of cut for the next charge is determined for the cross section 46 ft, S_3, to the right of charge No. 3. The design data for that and the other charges are recorded in Table 22. Charge positioning is shown in Fig. 45. Data are given to the nearest 0.5 ft to show the procedure for designing an explosive excavation in nonlevel terrain; however, positioning the charges to the nearest foot should be adequate for practical purposes.

(3) Proceeding to Left of Charge No. 1

The positioning data for charge No. 2L is determined from the depth of cut required 43 ft, S_1, to the left of charge No. 1. In this case, only one charge to the left of charge No. 1 is required. Data are given in Table 22. If more charges were required, their positioning data would be determined as illustrated above but moving along the profile to the left instead of to the right.

(4) Summary of Preliminary Design

The proposed explosive excavation could be accomplished with 15 charges each of 20 tons of TNT or equivalent or a total equivalent of 300 tons of TNT. The predicted profile of the row-crater bottom is Line MNOPQR in Fig. 45. The estimated volume of excavation is 52,800 yd^3.

10.6 PRELIMINARY SAFETY ANALYSIS

A preliminary safety analysis for a single detonation in basalt is made based on the procedures given in Section 9.7. Residential buildings in the region generally are located in the river valleys and have their foundations on soil.

From Figs. 41 or 26 for the above conditions:

$$k = 1.00 \text{ and } k^{1.43} = 1.00.$$

Table 22. Project design data (constant-charge-weight method).

Cross section for determining D_{ar} (Fig. 45) (station)	Depth of cut D_{ar} (from Fig. 45) (ft)	Single charge weight Y_s (from Fig. 17) (tons TNT or equiv.)	Depth of burial, (from Fig. 17) (ft)	Charge No.	Charge weight Y_r (tons TNT or equiv.)	$S/R_a = 1.4\left[\frac{Y_r}{Y_s}\right]^{0.6}$ when $S/R_a > 1.4$, e < 1, use S/R_a = 1.4 and change Y_r to $Y_r = Y_s$.	$S_n = R_a \times (S_n/R_a)$ (see note a) (ft)	$\frac{S_n + S_{n-1}}{2}$ (ft)	Charge station (plotted on Fig. 45)
				Charge No. 1 at location of deepest cut					
Max. cut 1+60.0	31	43.5	56	1	20	0.88	43	—	Max. cut 1+60.0
				Proceeding to right of charge No.1					
Charge $station_{n-1}$ plus S_{n-1}									Charge $station_{n-1}$ plus $\frac{S_n + S_{n-1}}{2}$
2+03.0	30.5	41	55	2R	20	0.91	44.5	44	2+04.0
2+48.5	30	39	54	3R	20	0.94	46	45	2+49.0
2+95.0	29.5	37	53	4R	20	0.96	47	46.5	2+95.5
3+42.5	29	35	52	5R	20	1.00	49	48	3+43.5
3+92.5	28.5	33	51.5	6R	20	1.04	51	50	3+93.5
4+44.5	28	31	50.5	7R	20	1.08	53	52	4+45.5
4+98.5	27.5	29	49.5	8R	20	1.12	55	54	4+99.5
5+54.5	27	27	48.5	9R	20	1.17	57.5	56	5+55.5
6+13.0	26.5	25.5	47.5	10R	20	1.21	59.5	58.5	6+14.0
6+73.5	26	24	47	11R	20	1.26	61.5	60.5	6+74.5
7+36.0	25.5	22.5	46	12R	20	1.30	63.5	62.5	7+37.0
8+00.5	25	21	45	13R	20	1.36	66.5	65	8+02.0
8+68.5	—	20	44.5	14R	20	1.40	68.5	67.5	8+69.5[b]
				Proceeding to left of charge No. 1					
Charge $station_{n-1}$ minus S_{n-1}									Charge $station_{n-1}$ minus $\frac{S_n + S_{n-1}}{2}$
1+17.0	24.5	20	44.5	2L	20	1.40	68.5	56	1+04.0[b]
			$\sum$ 744.5		$\sum$ 300				

[a] R_a is single crater radius of Y_r, and n is charge number. R_a is 49 ft for 20-ton TNT equivalent charge.

[b] An estimate of cratering results to right of station 8+69.5 and to left of station 1+04.5 may be made based on the supplemental single crater parameters in Table 2.

From the design (Fig. 45 and Table 22):

$$Y_r = 20 \text{ tons of TNT or equivalent}$$

$$n = 15$$

$$y_t = nY_r = 300 \text{ tons of TNT or equivalent.}$$

$$e_{av}(dob) = \frac{(DOB)_{av}}{Y_r^{0.3}} = \frac{\sum (DOB's)}{mY_r^{0.3}}$$

$$= \frac{744.5}{15(20)^{0.3}} = 20.2 .$$

Substituting the above in the expressions found in Section 9.7 gives:

$$Y_t k^{1.43} = 300 ,$$

$$\frac{Y_t n^{0.522}}{e_{av}(dob)^{3.913}} = 9.61 \times 10^{-3} ,$$

$$\frac{Y_t n^{0.087}}{e_{av}(dob)^{3.913}} = 2.97 \times 10^{-3} .$$

Table 23. Project preliminary safety analysis data.

Annular zone		Seismic data		Airblast data			
				Perpendicular to line of charges		Along axis of line of charges	
Radii (thousands of feet)	Average radius (thousands of feet)	Peak ground acceleration (g)	Damage cost factor[a] ($/bldg)	Overpressure Δp (mbar)	Damage cost factor[a] ($/bldg)	Overpressure Δp (mbar)	Damage cost factor[a] ($/bldg)
2.0- 3.0	2.5	8.84	14,800[b]	19.5	125	11.5	28
3.0- 4.0	3.5	0.43	6,600[b]	13	40	7.5	8.20
4.0- 5.0	4.5	0.26	3,200[b]	9.6	17	5.6	3.70
5.0- 6.0	5.5	0.17	1,600[b]	7.5	8	4.4	1.80
6.0-10.0	8.0	0.082	400[b]	4.9	2.50	2.9	0.60
10.0-15.0	12.5	0.034	70	2.8	0.53	1.65	0.13
15.0-20.0	17.5	0.017	20	1.9	0.18	1.1	0.04
20.0-30.0	25.0	0.0084	5.40	1.25	0.055	<1	Negligible
30.0-40.0	35.0	0.0043	0.33	<1	Negligible	<1	Negligible
40.0-60.0	50.0	0.0021	0.05	<1	Negligible	<1	Negligible

[a]Damage cost factors are in dollars per residential type structure.

[b]Average value of a residence in the area is $20,000. Damage cost factor in $/bldg is equal to $/V (from Fig. 41) times $20,000.

With the above quantities, estimated seismic and airblast intensities and damage cost factors obtained from Figs. 41 and 42 are listed in Table 23.

The nearest structures to the detonation are relatively new farm buildings located in the river valley about 21,000 ft south of the project site. If the detonation is executed when there is no sound or airblast wave speed increase in the southerly direction, no damage to those farm buildings would be anticipated. From Table 12, no damage is predicted for seismic motions less than 0.02 g. From Table 15, some window breakage, especially to large store windows, could be expected at the 2-mbar overpressure level, but the probability of damage would decrease with range from the detonation. Table 23 indicates that beyond 17,500 ft from the detonation, peak ground accelerations would be less than 0.02 g, and airblast overpressures would be less than 2 mbar.

Although unlikely, complaints alleging damage could arise from owners of structures located beyond 17,000 ft from the detonation. The nearby inhabitants reside in the river valley north and south of the project location. The nearest buildings to the east and west are over 20 mi from the site. As indicated in Table 24, the cost of administratively handling possible complaints is estimated to be about $2400. This amounts to about 4-1/2 cents/yd^3 of excavation. The estimate is based on seismic damage cost factors because Table 23 indicates that in this project the airblast damage cost factor at a given range is negligible compared to that for ground shock.

In view of the above, it is not considered necessary to revise the explosive design because of safety problems.

Table 24. Project damage claim cost estimate.

Range from detonation (1,000's of ft)	Average range (1,000's of ft)	Number of people	Number of buildings[a]	Damage cost factor[b] ($/bldg)	Cost[c] ($)
20-30	25	400	100	5.40	540
30-40	35	12,000	3,000	0.33	990
40-60	50	69,600	17,400	0.05	870
				Total	$2,400

[a]Based on number of people; local data indicate that there is about one building per four people.

[b]From Table 23.

[c]Number of buildings times damage cost factor.

10.7 EXPLOSIVE SELECTION AND EMPLACEMENT CONSTRUCTION

For this example project analysis, it is assumed that Table 20 was developed from data obtained from potential explosive suppliers located 400 mi from the project site. The table includes the cost of transporting the explosives and explosive services. This table assists in explosive selection for the project. ANFO was eliminated from consideration because of the water which is present in the joints in the basalt formation. AN slurry was eliminated from consideration because it would require larger and, therefore, more costly emplacement cavities than 8% Al-AN slurry which has a lower "down-hole" cost per ton TNT equivalent. Combined emplacement construction and explosives costs for the remaining explosives are compared in Table 25.

Certain emplacement construction considerations are discussed in Section 10.5b. For this project, it is not considered necessary to case the emplacement holes. Adequate stemming material is available nearby, and it is estimated that emplacement holes can be backfilled and hand-tamped at $\$4.00/yd^3$. The quantity of stemming material required is based on the volume of the drill holes above the Zero Points minus two-thirds the height of the explosive cavities.

Based on Table 25, the use of 2% Al-AN slurry blasting agent is recommended for the project because it would result in the least explosive excavation costs. The quantity of the explosive required for each emplacement hole is:

20 tons of TNT or equivalent/E = 20/1.2

= 16 2/3 tons of 2% Al-AN slurry.

The design could be revised to determine whether use of larger sized charges would lead to a more economical solution. The maximum hole diameter capability of readily available drilling equipment is 54 in. From Section 10.5b, a charge

Table 25. Explosive selection and explosive and emplacement construction cost estimates (fifteen 20-ton charges of TNT or equivalent).

Item		Al-AN slurry blasting agents 2%	8%	20%	35%
From Table 20: Required cavity volume = 20 tons TNT equivalent × (2000/pE)	(cu ft)	411.4	344.2	313.8	236.4
From Fig. 39 (charge h/d = 6 used to reduce drilling costs): Required cavity diameter	(in.)	54	50	49	45
Subdrilling (below ZP) = 0.33 × (h/d)d	(ft)	9	9	9	8
Explosive costs ("down-hole"): From design Fig. 45: Quantity (tons TNT or equivalent)		300	300	300	300
From Table 20: "Down-hole" unit cost ($/ton TNT or equivalent)		208	250	288	328
Subtotal: Explosives cost ("down-hole")		$62,400	$75,000	$86,400	$98,400
Emplacement construction costs: From Table 22: Drilling to ZP's (744.5 - 8.5 in water)	(ft)	732	732	732	732
From above: subdrilling below ZP's (15 holes)	(ft)	135	135	135	120
Total drilling	(ft)	867	867	867	852
From Table 21: Estimated unit drilling cost	($/ft)	65	55	55	45
Subtotal: Drilling costs		$56,355	$47,685	$47,685	$38,340
Casing costs (none required)		—	—	—	—
Stemming costs: $\frac{\pi}{27}\left(\frac{d}{2}\right)^2$ × (drilling to ZP's - 2 × subdrilling) at $4.00/yd^3		$ 1,090	$ 933	$ 895	$ 816
Site preparation costs (est)		$ 3,000	$ 3,000	$ 3,000	$ 3,000
Mobilization and demobilization (100-mi round trip at $1.20/mi and 2 days rig time at $600/day)		$ 1,320	$ 1,320	$ 1,320	$ 1,320
Subtotal: Emplacement construction costs		$61,765	$52,938	$52,900	$43,476
Total explosives and emplacement construction costs		$124,165	$127,938	$139,300	$141,876
Cost per yd^3 explosive excavation (52,800 yd^3)	($/yd^3)	2.35	2.42	2.64	2.69

cavity with a diameter of 54 in. and an h/d ratio of 6 could contain:

$$Y = 20.9/E = 20.9/1.2$$
$$= 17.4 \text{ tons of } 2\% \text{ Al-AN slurry.}$$

The change is not considered significant enough to warrant redesign. However, the drilling cost estimates in Table 25 are based on drilling the entire length of the emplacement holes at full-charge diameter. A check would be made to find out whether the drilling contractors could develop an underreaming capability. If so, an estimate would be made to determine whether drilling a smaller diameter access hole and underreaming the explosive cavity to the required diameter would be more economical than drilling a full-charge-diameter access hole.

The following are the emplacement hole construction and explosives emplacement time estimates:

Item	Working days
Drilling 867 ft of hole at 15 ft/day (10-hr day)	= 58
Rig setup-teardown, 15 holes at 1 day/hole	= 15
Explosives emplacing and firing (15 × 16-2/3 tons) at 16-2/3 tons/day	= 15
Total	88 × 7/5 = 123 calendar days, say 4 months.

10.8 PROVISIONS FOR POSTSHOT CONSTRUCTION

No significant postshot construction, such as that discussed in Chapter 8, is envisioned for this project. Based on project requirements, an unlined cutoff channel is acceptable. The design (Table 22 and Figs. 44 and 45) indicates that the crest of the end lips will be below project depth. Thus, removal of lip material to meet cutoff channel specifications is not anticipated. The ejecta may not completely block the meander, but this is not necessary. It should be possible to accomplish any remedial excavation or fill work within the contingency estimated for the project.

10.9 SAFETY AND FIELD OPERATIONS CONSIDERATIONS

a. Safety Requirements

Based on a review of pertinent parts of Chapter 6 and the preliminary safety analysis in Section 10.6, the safety requirements for this project are considered to be minor. During the construction season, surface winds are usually mild, and generally blow northward in the river valley during the daytime and southward at night. Winds above the ridge lines are prevailing westerlies. It is estimated that the detonation could be executed 75% of the time during the construction season under atmospheric conditions which will not significantly extend the range of airblast overpressures as shown in Table 23. Also, Table 23 indicates that the level of airblast damage is relatively minor compared to ground shock effects at a given range. No requirements exist for the preshot survey of structures or evacuation of inhabitants. The main requirement is to prevent entry of people and animals into a designated limited access area during the detonation. This would be accomplished by a public relations program and by the guard system established for field operations. It is anticipated that owners would keep their cattle outside the limited access areas during the detonation. As discussed in Section 10.6, \$2400 is allocated for administratively handling damage complaints. Another \$600 is allowed for emplacing upstream and downstream fish barriers during the detonation. It is anticipated that other safety costs, if any, can be absorbed in the allowances for contingencies, supervision, and administration.

b. Field Operations Requirements

Field operations are discussed in Chapter 7 and Section 9.6. Trailers are available on a rental basis from dealers in the city 50 mi from the project site. Radio communications equipment is available from a previous project. Estimated cost for a trailer, sanitary facilities, electric power and logistical support is \$1.00/mi for mobilization and demobilization, \$8.00/day for rental, and \$120 for CP site preparation. Total cost is estimated as follows:

Mobilization, 100 mi. round trip	\$ 100
Rental, 165 days (Section 10.7)	1320
Set up	120
Total	\$1540

It is proposed to locate the CP generally off the end of the line of charges and on the hillside south-southeast of the site where the basement complex rock outcrops in several places. Maximum missile range is estimated from Fig. 35 for one 20-ton of TNT or equivalent charge at the shallowest depth of burial which, from Table 22, is 44.5 ft:

Abscissa: $DOB/Y^{1/3} = 44.5/(20)^{1/3}$
$= 16.4 \text{ ft/ton}^{1/3}$

Ordinate from Fig. 35: $R/Y^{1/6} = 1350 \text{ ft/ton}^{1/6}$

and $R = 1350\ (20)^{1/6} = 2220$ ft.

From Table 23, the airblast overpressure along the axis of the line of charges at that range would be about 11.5 mbar. From Table 15, that is less than the 13-mbar overpressure corresponding to probable structural damage. The trailer procured for the CP should have small window pane size, and it would be preferable that the windows could be opened. During the detonation, the windows would be opened, taped, or boarded. From the inset table in Fig. 41, $k^{1.43}$ is 0.32 for a structure on rock, and

$$Y_t k^{1.43} = 300(0.32) = 96.$$

From Fig. 41, the ground acceleration at 2220 ft from the detonation would be about 0.4 g. In the past, trailer CP's were located where ground accelerations were as much as 1.0 g. The CP, therefore, would be located at least 2220 ft south-southeast of the project site. An observer area for unprotected personnel would be located twice that distance (at least 4500 ft or 1 mi) from the detonation as recommended in Section 6.7. Both the CP and observer areas could be located on the State land reserved for recreational campsite development. The State is interested in the project, and it is anticipated that an agreement for temporary use of the land without cost could be reached.

10.10 SUMMARY

A summary estimate of explosive excavation costs, less mobilization, as discussed for the river bend cutoff project is presented in Table 26. Mobilization, construction camp, and access road construction costs would be about the same as those required if conventional means of excavation were used for the project. The project construction time and costs required for explosive excavation would be compared to those required for other means of accomplishing the project. Finally, contract documents would be prepared for the most favorable approach.

Table 26. Explosive excavation project cost estimate.

Feature or item	Unit	Quantity	Unit price	Amount
Mobilization, camps and access roads	LS	—	—	Same as for conventional execution
Explosives (Section 10.7) ("down-hole" including arming and firing)	ton	250	$250.00	$ 62,500
Emplacement construction (Section 10.7)				
Rig mobilization and demobilization	LS	—	—	1,320
Site preparation	LS	—	—	3,000
Drilling	ft	867	65.00	56,355
Stemming	LS	—	—	1,090
Explosive excavation field operations (Section 10.9b)				
CP facilities rental	days	165	8.00	960
CP facilities mobilization	mi	100	1.00	100
CP site preparation	LS	—	—	120
Safety (fish barrier)	LS	—	—	600
Claims (administrative handling of complaints, Sections 10.6 and 10.9a)	LS	—	—	2,400
Postshot construction (Section 10.8)				—
Subtotal				$128,445
($\$128,445/52,800\ yd^3 = \$2.43/yd^3$)				
Contingencies (20%)				25,689
Subtotal				$154,134
Engineering, design, supervision, and administration (20%)				30,827
Subtotal				$184,961
				or $185,000

Chapter 11

Explosive Excavation Research

11.1 SCOPE

This chapter presents a concise summary of research needed to make explosive excavation a more competitive construction tool. While the information presented in this report is adequate for designing straightforward projects such as those using simple single or double row crater shapes, most applications require some subtle or extreme modification of these simpler concepts. If these applications are to be accomplished economically, general design methods must be extended to cover them.

The final objective of a continuing research program is seen as the development of explosive excavation technology to the point that it is a familiar and commonly accepted construction technique among engineers so that it is available and used in those cases in which it is more advantageous than other methods. A productive program should include theoretical cratering research, project definition, and liaison work with potential project sponsors, field experiments, and investigation of engineering properties of explosive excavations. Six categories for research have been defined where additional data and cratering experience are critical to the practical development of explosive excavation. These are (1) topography and geologic media, (2) charge shape, emplacement patterns, and firing techniques, (3) explosives, (4) emplacement methods, (5) engineering behavior of craters, and (6) environmental effects and safety. As each of these topics is discussed in the paragraphs that follow, a brief statement of the state-of-the-art is given that in some cases refers to specific experiments. These experiments are discussed with pertinent findings in Appendixes A and B of this report. The reader is referred to these appendixes for more specific information about the data upon which this report is based.

11.2 TOPOGRAPHY AND GEOLOGIC MEDIA

Chemical explosive cratering tests have been conducted in dry soil, dry basalt, rhyolite, saturated clay shale, coral overlain by water, granite (at very shallow depths of burial), sandstone, dry sand, and clay. Actual cratering curves (dimensions as a function of depth of burial) exist only for a dry rock (mostly basalt data), a dry soil (alluvium), and a saturated clay shale. In the ton's charge weight range, $Y^{0.3}$ has been established as the best scaling to use.

Varying terrain row-charge detonations were successfully accomplished in the Pre-Gondola and Trinidad series. It is expected that additional experience and data in this area will be acquired as application projects are proposed and small-scale calibration tests are accomplished on each site where there is a new media and a new topographic feature in the project. Listed by priority the following research is recommended:

(1) Experience in a wet competent medium. Conduct single-charge detonations

in a wet competent rock, such as sandstone, high strength dolomite or granite, to establish cratering curves.

(2) Ejecta control. Perform single-charge and row-charge sidehill detonation experiments to determine proper burial distance from nearest free surface to limit throwout to an area determined by the project application (e.g., road cut, expedient dam, or quarry).

(3) Underwater cratering. Conduct underwater cratering tests in high strength rock, such as granite or basalt.

(4) Design concepts for complex terrain situations. Test new varying terrain row design techniques other than those presented in Chapter 5.

11.3 CHARGE SHAPES, EMPLACEMENT PATTERNS, AND FIRING TECHNIQUES

Many small-charge-weight modeling experiments have been conducted with objectives related to charge shape, emplacement patterns, and firing procedures. Charge weights have been less than 500 lb in most cases.

Five-hundred-pound cylindrical charges with length-to-diameter ratios of up to 9 have been detonated at cratering depths in clay shale. These tests indicate that cylindrical geometries can be used with length-to-diameter ratios up to 6 without severe loss of crater volume. The use of longer, smaller diameter charges reduces the cost of charge emplacement.

The emplacement patterns that have been tried include the two-row array, three-row array, the four-charge square array, the square array with a charge in the center, two rows connected in a "T" shape, and a widely spaced three-row array with the outside rows fired first, followed by the center row after all ejecta has settled. The latter test used 1-ton charges. The four-charge square array pattern was detonated in the Project Tugboat berthing basin excavation using 10 tons per charge. The two-row array was used at Trinidad with mixed weights of 1 and 2 tons per charge. All of the other arrays were detonated with charge weights of 64 lb or less.

Prior to Project Trinidad there had been only one experimental series of significant charge size that provided data on the loss of volume excavated by a row of charges when a delay time is introduced between initiation of successive charges in the row. Sixty-four pound charges were used in these detonations and delays ranged to 25 msec. There was indication that significant volume loss does occur at the larger time delays. The reason for using time delays is to reduce the airblast and ground motion accompanying the detonations. Some of the experiments at Project Trinidad are expected to provide additional data in this area.

Listed by priority, further research is recommended as follows:

(1) Charge shape. Determine the effect of charge geometry changes at larger charge sizes. Concentrate on cylindrical charge shapes with high height-to-diameter ratios.

(2) Wide excavations. Conduct multi-row experiments at significant charge weights to produce wide craters.

(3) Long excavations. Develop within-row firing delay techniques. Use this

method to replace the necessity to use connecting rows in long excavations.

(4) Directional excavation. Perform simultaneous and delay firings of adjacent rows for rock ledge removal and sidehill road cut applications. Between-row spacing criteria is critical in this work.

(5) Mound velocity measurements. Make mound velocity measurements on multiple-charge detonations when possible. Conduct studies to try to determine both explosive and medium effects on mound velocity.

(6) Side slope control. Investigate presplitting, cushion blasting, and the formation of crushed zones in connection with row excavations to try to meet side slope and overbreak specifications.

11.4 EXPLOSIVES

A big decision to make on any project is "What explosive do I use? There is experience with TNT and nitromethane in soil, basalt, some granite (shallow depth), sandstone (shallow depth), rhyolote (one large detonation) and clay shale. There is also experience with aluminized ammonium nitrate slurries in clay shale, coral under water, and sandstone. A very limited amount of comparable data are available for crater dimensions and volume as a function of the kind of explosive used in a particular medium. Some sand test-pit work (few pounds per charge) indicates a considerable variation in volume excavated for different explosives. This work was extended to a few 500-lb detonations in clay shale where results were different. It is evident that the proper selection of an explosive for a project depends on many factors which are just beginning to be understood. Listed by priority the following work in this area is recommended:

(1) Explosive characteristics. Perform investigations and tests to establish those explosive characteristics that are most important to cratering in a specific medium. Develop the capability to tailor the explosive to the medium by having a clear understanding of the following for each explosive:

(a) Cratering effectiveness or performance

(b) Price per unit volume excavated

(c) Rock breaking effectiveness

(d) Mound velocities produced

(2) Explosive performance in a specific medium. Conduct experiments to determine the relative cratering performance of explosives (blasting agents) in a variety of media. Limit investigations to explosives that are relatively easily and safely handled. Give particular attention to ANFO.

(3) Economics. Conduct research to get the most crater volume for the least explosive volume at reasonable cost.

(4) Delay firing systems. Develop an electronic down-hole delay firing system.

11.5 CHARGE EMPLACEMENT METHODS

In most of the experiments summarized in Appendixes A and B, emplacement holes are drilled at a diameter at least as large as the charge diameter, or access holes were drilled and charge cavities were mined. Both methods of charge emplacement are costly. For projects in which a few large emplacement holes can be drilled easily, full-charge-diameter drilling may still be the best method. When this is done a canister can be placed in the hole to accept the explosives if necessary to prevent the loss of the explosive to faults and fractures in the medium. However, if explosive excavation is to be economical for a large number of projects the cost of emplacement must be reduced. A major effort is needed in this area of research. Four possibilities recommended for further work are:

(1) Hole springing. Determine the best explosive to use and the best techniques for successively springing large cavities at the bottom of small diameter access holes. Determine the media in which the technique can be made to work and in which the technique is not feasible.

(2) Underreaming to create a cavity. It appears that hardware to underream a pilot emplacement hole, especially for the larger cavities in high strength rock, is presently available but little cost experience exists for charge emplacement applications. Determine the drilling conditions and specifications for underreaming tools which will lead to cost advantages in charge emplacement.

(3) Comparative Data. Compare hole springing and underreaming with full-charge-diameter drilling on the basis of cost and engineering problems associated with emplacement.

(4) Stemming. Develop criteria for inexpensive and effective stemming.

11.6 ENGINEERING BEHAVIOR OF CRATERS

The engineering behavior of craters can be discussed in terms of seepage, stability, and settlement. The emphasis in research to date has been on stability and specific crater properties as they relate to stability. It has been determined that craters in dry soil and rock are initially stable.

The Pre-Buggy (soil), Pre-Schooner (basalt and rhyolite), and Pre-Gondola (clay shale) craters have been extensively investigated. This work has made it possible to predict, with some certainty in these media, the size and shape of the apparent and true craters, the shape and extent of the rupture zone, the particle size distribution and the bulking factor of the fallback and ejecta.

The current effort is to relate the known properties of craters to their seepage characteristics. Yet to be investigated is the problem of settlement in fallback and ejecta and its impact on use of the craters. The work ahead is directly related to the use of craters as engineering structures. Listed by priority, the following tasks are recommended:

(1) Settlement of fallback and ejecta. Determine the settlement characteristics of fallback and ejecta in sandstone craters (Project Trinidad). Develop a settlement prediction method based on the compressibility of rockfill material.

(2) Water seepage in craters. Determine the relationships between porosity, permeability, and seepage of water in craters. Several problems are posed. One is the problem of the ability of the crater to hold water for canal or storage applications. A second is the problem of build up of water pressures in the crater zones causing instability. A third is the estimation of rubble available for a rockfill dam. A fourth is the problem of drainage from a roadbed built in the bottom of a crater.

(3) Material strength. Continue to research methods of measuring the strength of materials "in-mass." This measurement relates directly to stability.

(4) Crater zones. Expand knowledge of the extent and properties of crater zones to new materials. The most critical need is for data in a wet, high strength rock.

11.7 ENVIRONMENTAL EFFECTS AND SAFETY

The emphasis in the past in this area has been to make detailed measurements of the major effects, primarily ground shock and airblast. These programs were very expensive. The emphasis in the future should be to provide reasonable prediction techniques from a thorough analysis of all previous data and to provide for simply documenting the actual effects produced on a project at reasonable cost. Some comprehensive measurements will have to be made on the more complex multiple-row detonations. An additional concern is assessing the damage to the environment immediately adjacent to the project from ejecta.

New research requirements are as follows:

(1) Missile range. Develop prediction capability for determining missile range for multiple-charge cratering detonations, especially in directed blasting applications. Determine probable damage to trees in these kinds of projects.

(2) Airblast and seismic motion transducers for documentation. Develop a simple and inexpensive passive peak air overpressure transducer for use on explosive excavation projects. Conduct studies to determine whether there is any possibility of developing a similar transducer for measuring peak ground motion and structure velocity and displacement.

(3) Effects from detonation of complex designs. Make comprehensive airblast and ground motion measurements on at least one multiple-row detonation. Develop simplified procedures for the prediction of potentially damaging effects.

(4) Structural response. Continue developing damage criteria for structural response to explosively generated seismic motions.

11.8 SUMMARY

There are three primary incentives for using explosive excavation on a project. The most obvious one is when it is economical to do so. In situations in which it is extremely difficult to mobilize conventional earthmoving equipment, explosive excavation should be considered. A third less obvious reason for using explosive techniques is when a relatively fast method of excavation is desired to reduce the total effect on the local ecology or in an emergency situation. As earthmoving construction projects become more complex in the years to come with all of the attendant engineering, ecological, and social considerations, additional methods of accomplishing the job must be developed and made available to the designer. This report provides a new and unique method of excavation which should have extensive application in the future.

The simpler projects can be accomplished with the technology presented in this report. Additional research work is needed to extend the technology to a variety of complex project applications. If these kinds of projects are to be done, the costs must be reduced. The most critical cost factor is the cost of creating the underground charge emplacement cavity. Priority should be given to four general areas of research as listed below:

(1) Reducing the cost of getting an explosive charge underground. This includes being able to make the proper decision as to the cheapest explosive that will do the job, reducing the cost of producing a charge cavity, and determining the effect of charge geometries more favorable to lower cost.

(2) Developing criteria for design of charge layouts to achieve special project requirements. This involves developing cratering curves in as many media as possible and pursuing a variety of multiple-charge designs in conjunction with possible delayed firing techniques to achieve design objectives.

(3) Determining the engineering behavior of craters. The immediate need is the development of a method of predicting settlement and seepage in the fallback zone.

(4) Reducing the cost of fielding an operation by providing simple, effective and inexpensive ways of documenting the major effects.

Appendix A
Evolution of Explosive Excavation

A.1 PURPOSE

The purpose of this appendix is to provide a short historical sketch of the development of explosive excavation technology and to present the data on which the technology is based.

A.2 CONTRIBUTION OF PLOWSHARE PROGRAM

Plowshare is the name of the U. S. Atomic Energy Commission's program which has responsibility for conducting research and development related to peaceful applications of nuclear explosives. The chemical explosive cratering experiments conducted in connection with the Plowshare Excavation Program over the past several years have formed the basic technology for this report. These experiments greatly expanded the available data on cratering which had been generated by earlier military research programs. These experiments were designed primarily as models of nuclear experiments; i.e., the explosive charges were usually spherical in shape and the depth of burials usually slightly greater than optimum for crater dimensions. The large number and variety of experiments conducted under this program contributed greatly to an understanding of the cratering process and to charge location design problems. However, they did not contribute much to the technology required to reduce charge emplacement costs. Additionally, only two kinds of explosives were used in the larger yield (tons) experiments. Consequently, there is as yet very limited comparative knowledge about the cratering effectiveness of the large number of less expensive and more easily handled explosives and blasting agents on the market.

A.3 EXPLOSIVE CRATERING DATA BASE

Blasting is common practice in construction, quarrying, and mining. Many large blasts (tons of explosives) have been used to break and move earth materials. Two such large blasting projects have been described in Chapter 2. The design techniques used in the majority of these blasts differ from the method described in this report; i.e., that of burying and spacing charges on the basis of single-charge apparent crater dimensions. In normal blasting design a "powder factor" is used, which is the weight of explosive required to break up or move a unit weight of material. This kind of factor is only of incidental use in cratering application design as a check point and as a cost evaluation tool. It does not provide the basic crater dimensions used as the primary design parameters.

The blasting that has contributed the most crater dimension data has been experimental in nature and has been performed primarily by government agencies

(Department of Defense, Atomic Energy Commission, Air Force Weapons Laboratory, Defense Atomic Support Agency, Nuclear Cratering Group), certain contractors (Sandia Laboratories, Lawrence Radiation Laboratory, and Stanford Research Institute), and by foreign governments (principally France and the USSR). The work by NCG in connection with the AEC Plowshare Program is the most directly applicable to the subject of explosive excavation. Numerous small-yield laboratory-scale and field experiments have been reported. Yields range from grams to hundreds of pounds per charge. There are too many of these to list here. The relatively few experiments of larger yield which have been reported are listed in Tables A1 (single-charge detonations) and A2 (row-charge detonations). The listings are restricted to buried charges weighing ~1000 lb or more.

Each experiment was designed to achieve one or several specific objectives not always related to explosive excavation. Several of the larger chemical explosive cratering experiments were specifically aimed at calibrating a geologic medium or specific site, or in modeling a planned nuclear cratering experiment in some way. Emplacement construction and explosive selection were always done to satisfy criteria other than that which would apply to a construction application. Some of the experiments were conducted solely to gather data on explosion-generated effects, especially related to safety. Short discussions of the objectives and significant results of some of the more pertinent experiments listed in Tables A1 and A2 illustrate these points.

A.4 EARLY EXPERIMENTS

The early experiments had objectives related to the military program. Among these were Dugway and Jangle HE. All of the detonations in these experiments were at shallow depths of burial, and therefore incidentally contributed to the crater-dimensions-vs-depth-of-burial curves.

All experiments performed in connection with the Plowshare Program were named after conveyances. The first of these were Stagecoach, Buckboard, and Scooter which were executed by the Sandia Laboratories. Stagecoach and Scooter were conducted in the desert alluvium in Area 10 of the Nevada Test Site. The three 20-ton charges used in Stagecoach were emplaced at different depths in order to establish a cratering curve for the alluvium medium. Cratering curves had been established at small yields and a major objective of Stagecoach was to provide the scaling exponent to scale crater dimensions at small yields to those at the 20-ton level. Cube root ($Y^{1/3}$) scaling had been assumed, and it was found not to hold at all burial depths. This made scaling to larger yields uncertain and, of course, the design of a large nuclear experiment was the objective.

The Scooter experiment used a single charge of 500 tons at near optimum depth of burial. It had two major objectives: (1) to study the mechanics of crater formation and (2) to clarify scaling laws. The major result of this experiment was confirmation of 1/3.4 as the yield scaling exponent ($Y^{1/3.4}$) for desert alluvium.

Table A1. Dimensions of craters from buried single charges.[a]

Series name	Shot designation	Sponsor	Date	Medium	Type of explosive	Charge weight (lb)	Charge burial depth (ft)	Apparent crater radius (ft)	Apparent crater depth (ft)	Apparent crater volume (ft^3)
Dugway	108	DOD	7/10/51	Dry sand (Utah)	TNT	2,560	2.6	19	9.75	5,200
	109		7/10/51			2,560	7.0	24.75	8.5	8,200
	112		7/27/51			2,560	7.0	30	12	13,000
	115		8/8/51			40,000	17.5	75	23	180,000
	308		4/16/51	Dry clay (Utah)		2,560	2.6	20	12	5,400
	309		4/18/51			2,560	7.0	21.5	15.5	7,300
	312		5/4/51			2,560	7.0	26	15	13,000
	315		5/10/51			40,000	17.5	64	42	190,000
	318		5/10/51			320,000	35.0	120	60	1,100,000
	317		5/10/51			2,560	7.0	23	15.5	11,000
	319		5/10/51			2,560	7.0	23	13.5	7,800
	403		8/11/51	Wet clay (Utah)		2,560	5.0	41.75	12.75	29,000
	609		Unknown	Granite		2,560	5.0	25.2	10.2	Not reported
	610		Unknown			2,560	5.0	23.1	8.7	Not reported
	809		Unknown	Sandstone		1,080	3.75	19.0	8.6	Not reported
	812		Unknown			2,560	5.0	23.3	11.0	Not reported
	813		Unknown			10,000	7.9	39.4	16.1	Not reported
	815		Unknown			40,000	12.5	70.5	26.9	125,000
	816		Unknown			40,000	12.5	53.6	27.5	Not reported
	817		Unknown			320,000	25.0	94.8	47.0	512,000
	810		4/9/52			2,560	5.0	32.6	9.7	8,650
	811		4/30/52			2,560	5.0	25.1	10.5	7,050
	814		6/4/52			40,000	12.5	56.5	26.9	108,000
Jangle He	HE-1	DOD/SRI	8/25/51	Alluvium (NTS Area 10)	TNT	2,560	2.05	18.50	6.70	2,010
	HE-2		9/3/51			40,000	5.13	39.00	15.00	35,000
	HE-3		9/15/51			2,560	6.84	20.27	10.80	6,000
	HE-4		9/9/51			2,560	-2.05	6.90	1.90	110
	HE-5		9/31/51			2,560	4.10	19.40	7.50	4,000
	HE-6		10/2/51			2,560	3.01	19.80	6.10	3,600
	HE-7		10/4/51			2,560	2.60	19.00	6.70	3,300
Stagecoach	1	Sandia Lab	3/15/60	Alluvium (NTS Area 10)	TNT	40,120	80.0	57.0	7.9	49,145
	2		3/19/60			40,240	17.1	50.5	23.6	83,650
	3		3/25/60			40,070	34.2	58.6	29.2	144,600
Buckboard	1	Sandia Lab	6/23/60	Basalt (NTS)	TNT	1,000	24.6	—	—	—
	2		6/21/60			1,000	18.9	4.63	1.40	45
	3		6/30/60			1,000	14.7	15.65	5.20	1,800
	4		8/16/60			1,000	9.6	16.70	6.50	2,620
	5		7/1/60			1,000	4.8	15.00	7.50	1,890
	6		6/27/60			1,000	24.0	6.10	5.20	185
	7		6/30/60			1,000	18.6	10.67	3.80	654

Table A1 (continued)

Series name	Shot desig- nation	Sponsor	Date	Medium	Type of explosive	Charge weight (lb)	Charge burial depth (ft)	Appar- ent crater radius (ft)	Appar- ent crater depth (ft)	Apparent crater volume (ft^3)
Buckboard (cont)	8		6/24/60			1,000	14.7	16.92	8.80	3,500
	9		8/16/60			1,000	9.6	12.15	4.80	800
	10		7/6/60			1,000	4.8	15.80	7.00	2,660
	11		9/14/60			40,000	25.5	44.66	24.90	54,220
	12		9/27/60			40,000	42.7	57.00	34.70	135,000
	13		8/24/60			40,000	58.8	36.80	16.20	23,200
Scooter		Sandia Lab	10/13/60	Alluvium (NTS Area 10)	TNT	987,410	125.0	153.8	74.5	2,642,000
Pre-Buggy I	Test	NCG		Alluvium (NTS Area 5)	Nitro-methane	1,017	15.0	22.7	10.9	Not reported
	1		12/5/62			1,003	15.0	21.0	9.7	6,560
	2		12/10/62			1,011	16.6	21.8	9.1	7,560
	3		12/11/62			1,011	18.2	20.9	7.8	5,830
	4		12/13/62			1,009	19.8	20.6	9.4	6,530
	5		12/18/62			1,016	21.4	19.7	4.1	2,650
	6		12/19/62			1,015	19.6	20.7	8.3	6,080
Pre-Buggy II	F1	NCG	8/6/63	Alluvium (NTS Area 5)	Nitro-methane	1,000	19.8	22.7	11.8	7,860
	F2		8/6/63		Nitro-methane	1,000	19.8	21.2	11.8	6,030
	F3		8/6/63		TNT	950	18.5	21.1	11.0	6,950
	F4		8/6/63		TNT	950	18.33	22.1	10.8	7,560
Air Vent	1	DASA/ Sandia Lab	12/14/63	Playa (Frenchman Flat, NTS)	TNT	40,000	17.19	47.61	22.5	72,500
Pre-Schooner I	Alfa	NCG	2/6/64	Basalt (NTS)	Nitro-methane	39,250	58.0	50.3	22.9	75,800
	Bravo		2/13/64			39,450	50.2	49	25.5	73,900
	Charlie		2/25/64			39,840	66.1	Mound	-1.3	Mound
	Delta		2/27/64			39,590	41.8	46.1	25.6	64,800
Multiple threat cratering experiment		Air Force Weapons Lab		Basalt (Yakima Firing Center)	TNT					
	C2		6/25/65			4,000	2.2	14.0	5.3	1,500
	St1		6/11/65			4,000	-2.2	3.7	1.3	37
	St2a		7/14/65			4,000	-2.2	6.8	1.9	139
	St3a(C3)		6/23/65			4,000	-2.2	4.5	1.2	25
Pre-Schooner II		NCG	9/30/65	Rhyolite (Idaho)	Nitro-methane	171,000	71.1	95.2	60.7	669,000
CAPSA	1	Sandia Lab	8/16/66	Alluvium (Albuquerque)	TNT	1,000	15.0	18.82	7.10	4,045
	2		8/18/66			1,000	12.5	18.11	10.27	4,840
	3		8/24/66			1,000	10.0	17.89	10.49	4,557
	4		8/26/66			1,000	17.5	19.15	7.07	3,930
	5		8/31/66			1,000	15.0	19.67	7.10	4,172
	6		9/2/66			1,000	17.5	19.72	10.58	5,936
	7		9/13/66			1,000	10.0	16.12	9.12	3,661
	8		9/16/66			1,000	12.5	19.43	10.63	5,781
	9		5/21/68			1,000	12.5	18.45	10.43	5,339
	10		5/29/68		Nitro-methane	1,000	12.5	19.40	11.33	6,470
	11		6/13/68		Comp B	30,478	47.9	57.03	28.74	121,746

Table A1 (continued)

Series name	Shot desig-nation	Sponsor	Date	Medium	Type of explosive	Charge weight (lb)	Charge burial depth (ft)	Appar-ent crater radius (ft)	Appar-ent crater depth (ft)	Apparent crater volume (ft^3)
CAPSA (cont)	12		7/25/68		Nitro-methane	977	12.5	19.89	9.44	5,737
	13		7/25/68		Nitro-methane	981	15.0	19.33	9.65	5,182
Pre-Gondola I	Bravo	NCG	10/25/66	Bearpaw clay shale (Montana)	Nitro-methane	39,240	42.49	80.4	32.6	277,550
	Charlie		10/28/66			38,720	46.25	78.5	29.5	241,260
	Alfa		11/1/66			40,700	52.71	76.1	32.1	235,300
	Delta		11/4/66			40,480	56.87	65.1	25.2	133,880
	SC-4		6/21/66			1,000	12.2	24.5	13.0	
	SC-2		6/22/66			1,000	15.8	27.3	12.5	
	SC-1		6/20/66			1,000	19.1	7.1	2.8	
	SC-3		6/23/66			1,000	23.3	14.6	3.4	
Pre-Gondola III Phase I	A	NCG	7/25/68	Bearpaw clay shale (Montana)	Nitro-methane	2,000	6	21.5	11.5	
	B					2,000	12	25	13	
	C					2,000	14.3	27.2	14	
	D					2,000	16.8	25.8	12	
	E					2,000	19.5	27	11.7	Not reported
	F					2,000	22	25.4	11	
	G					2,000	24	18	6.4	
	H					2,000	26	0	0	
Tugboat Phase I	Alfa	NCG	11/6/69	Coral and water (Hawaii)	Aluminized ammonium nitrate slurry	2,000	16.33	56.2[b]	10[c]	
	Bravo		11/6/69			2,000	16.66	52.2[b]	11[c]	
	Charlie		11/4/69			1,975	20.12	68.0[b]	10[c]	
	Delta		11/5/69			1,950	24.74	53.2[b]	11[c]	
	Echo		11/7/69			20,200	41.3	129.0[b]	15[c]	
Trinidad[d]	B1	NCG	8/13/70	Interbedded sandstone and shales	ANFO	2,000	15.2	17	8.0	3,200
	B2		8/14/70			2,000	18.0	20	11.5	6,000
	B3		8/12/70			2,000	19.7	24	6.5	3,500
	B4		8/13/70		Aluminized (18-20%) ammonium nitrate slurry	2,000	15.9	23.5	12.8	9,100
	B5		8/11/70			2,000	18.6	23.2	13.0	8,100
	B6		8/10/70			2,000	20.9	21.5	11.5	7,000
	B7		8/12/70			2,000	22.6	20.2	6.0	3,500
	B8		8/11/70			2,000	28.1	Mound		

[a]Major portions of this table were taken from "Ten Years of High Explosive Cratering Research at Sandia Laboratory," by L. J. Vortman (see Bibliography at end of chapter).

[b]Radius at preshot coral surface.

[c]Depth below Mean Lower Low Water level (MLLW).

[d]All data are preliminary.

Table A2. Dimensions of craters from row charges.[a]

Series name	Shot designation	Sponsor	Date	Medium	Type of explosive	Charge weight (lb)	Charge burial depth (ft)	Spacing between charges (ft)	Average apparent crater width (ft)	Average apparent crater depth (ft)	Apparent crater volume (ft^3)	Apparent crater volume per lb of explosive (ft^3/lb)
Pre-Buggy I	A	NCG	1/16/63	Alluvium (NTS Area 5)	Nitromethane	5 ea 1,016	19.8	20.6	44.5	12.1	35,100	6.91
	B		1/23/63			5 ea 1,023	19.8	30.9	33.1	4.9	12,960	2.53
	C		1/31/63			5 ea 1,007	19.8	25.8	38.6	4.4	17,820	3.54
	D		2/7/63			5 ea 1,004	19.8	23.2	42.8	6.4	24,435	4.87
Pre-Buggy II	C	LRL	5/28/63	Alluvium (NTS Area 5)	Nitromethane	5 ea 1,000	19.8	20.6	49.5	15.4	41,420	8.28
	G		6/5/63			5 ea 1,000	19.8	23.2	47.3	13.7	43,060	8.61
	D		6/7/63			5 ea 1,000	19.8	25.74	46.8	11.7	47,775	9.56
	B		6/11/63			5 ea 1,000	23.0	20.6	46.2	7.6	23,950	4.79
	C'		6/13/63			5 ea 1,000	23.0	20.6	40.6	9.3	71,460	7.15
	H		8/2/63			13 ea 1,000	19.8	20.6 25.75 30.9	49.5 46.7 40.2	14.7 13.0 10.5[b]	107,780	8.29
Dugout	—	LRL	6/24/64	Basalt (NTS)	Nitromethane	5 ea 40,000	59.0	45	136.4	35.1	567,800	5.68
Pre-Gondola II		NCG	6/28/67	Bearpaw clay shale (Montana)	Nitromethane	274,820					1,312,631	4.78
						Charlie Crater[c]						
								105.5				
				Width and depth averaged from midpoint to midpoint between charges		E77,200	59.7		206.5	55.5	Volumes by section not reported	
								79.8				
						F39,400	49.4		152.5	37.5		
								79.9				
						G39,100	48.8		164.0	36.9		
								79.9				
						H79,120	59.9		214.5	57.0		
								79.9				
						I40,000	48.8		173.0	33.5		
Pre-Gondola III Phase I		NCG	9/68	Bearpaw clay shale (Montana)	Nitromethane	7 ea row 2,000 (2 rows)[d]	19.0	27.0	64.2	14.6	Not reported	
Pre-Gondola III Phase II		NCG	10/30/68	Bearpaw clay shale (Montana)	Nitromethane	300,500					2,350,000	5.57
						Charge I[e]						
								93.3				
				Width and depth measured at charges and midpoints		M60,000	5.6		196	54	Volumes by section not reported	
								86.0	190	55		
						N58,800	0.50		185	46		
								86.6	191	49		
						O60,200	55		182	53		
								86.0	187	47		
						P60,500	52		182	46		
								87.1	199	49		
						Q61,000	51		212	50		
								52.6	223	52		
						R60,600	50		202	57		
								52.1	217	52		
						S60,500	54		216	46		
Pre-Gondola III Phase III enhancement experiments	Row	NCG		Bearpaw clay shale (Montana)	Nitromethane							
	A-1		8/14/69			6 ea 2,000	19.3	27	64	15.0	Not reported	7.8[f]
	A-2		10/24/69			7 ea 2,000	21.0	23	68	17.5		8.5[f]
	A-3		10/24/69			5 ea 2,000	17.9	31	60	15.5		8.2[f]
	B-1		8/14/69			7 ea 2,000	23.0	19	76	17.5		7.7[f]
	B-2		10/27/69			6 ea 2,000	21.0	23	70	16.5		7.9[f]
	B-3		10/27/69			9 ea 2,000	26.0	15	86	22.0		8.7[f]

Table A2 (continued)

Series name	Shot designation	Sponsor	Date	Medium	Type of explosive	Charge weight (lb)	Charge burial depth (ft)	Spacing between charges (ft)	Average apparent crater width (ft)	Average apparent crater depth (ft)	Apparent crater volume (ft^3)	Apparent crater volume per lb of explosive (ft^3/lb)
Pre-Gondola III Phase III reservoir connection	Charge	NCG	10/6/69	Bearpaw clay shale (Montana)	Aluminized ammonium nitrate slurry	5 chgs					550,000	3.9
						30,000	37.0		240	52		
								60				
						70,000	52.0		225	66		
								65			Volumes by section not reported	
						20,000	37.0		185	44		
								55				
						10,000	30.0		124	37		
								50				
						10,000	25.0		110	38		
Tugboat Phase II	Outer channel row	NCG	4/23/70	Coral and water (Hawaii)	Aluminized ammonium nitrate slurry	4 chgs 20,000 each	42	100	163[g]	21[h]	2,173,100	—
	Inner channel row		4/28/70			4 chgs 20,000 each	42	120	208[g]	13.8[h]		—
	Square array		5/1/70			4 chgs 20,000 each	42	120	Square avg[g] 309 by 290	14.7[h]	1,545,500	19.3
Trinidad[i]	C1	NCG	9/28/70	Inter-bedded sandstones and shales	Aluminized (18-20%) ammonium nitrate slurry	5 chgs 2,000 each	18	32	48	12.8		5.8
	C2		9/29/70			5 chgs 2,000 each	20	25	51	14.1		4.7
	C3		10/1/70			7 chgs 2,000 each	23	18	67.4	18.9		6.2
	C4		10/1/70			5 chgs 2,000 each	20.6	25	50.8	10.8		4.3
	C5		10/2/70			5 chgs 2,000 each 2 parallel rows	20.2	25	53	12.8		4.8
	C6		9/30/70			5 chgs/row 2,000 each	20	25	Not reported	Not reported		4.8
	D1	NCG	11/17/70	Interbedded sandstones and shales	ANFO	9 chgs 200 to 2,000	13 to 24	$0.8R_a$		Max 20		5.6
	D2		11/18/70		5% Al-AN	5 chgs 2,000	18	$1.6R_a$		Max 10		4.0
	D3		11/19/70		35% Al-AN	12 chgs 2,000 and 4,000	19 and 25	1.4 and $1.1R_a$		Max 15		3.0 (avg) 5.3 (max)
	D4		12/16/70		20% Al-AN (two parallel rows)	32 chgs 2,000 and 4,000	17 to 25	0.9 to $1.25R_a$				5.5

[a] Major portions of this table were taken from "Ten Years of High Explosive Cratering Research at Sandia Laboratory," by L. J. Vortman (see Bibliography at end of chapter).

[b] The spacing between charges in this row varied. The first four spaces were 20.6 ft, the next four were 25.75 ft, and the last four were 30.9 ft. The dimensions reported are averages for that portion of the crater only.

[c] Already existing from Pre-Gondola I series.

[d] This was a 3-row-charge experiment; the two outside rows were fired simultaneously, then the center row was fired to remove the mound between the two rows and to produce a broad flat crater. Only average dimensions for the craters produced by the two outside rows are included here.

[e] Nearest charge location in Pre-Gondola II row.

[f] These factors are derived for the linear section of the crater only, excluding ends.

[g] At 12-ft depth contour.

[h] Depth below Mean Lower Low Water level (MLLW).

[i] All data are preliminary.

Because most of the Plowshare excavation applications were in rock media, some rock cratering data were needed. A new sea-level canal across the Isthmus of Panama was by this time the most obvious excavation application. Project Buckboard was the first attempt to get rock data at a significant yield. The site was the Buckboard Mesa basalt which was the closest thing to the Punta Sabana basalt from Panama that could be found on the Nevada Test Site. The data obtained from these tests were very scattered.

The explosive used in all of these early experiments and in two later experiments, Capsa and Air Vent, was TNT. The TNT was stacked in a spherical shape in an underground cavity. This method of emplacement was related to the objectives of the experiments and was very costly.

A.5 PRE-BUGGY, PRE-SCHOONER AND DUGOUT EXPERIMENTS

Following the execution of the nuclear cratering experiments, Sedan (100 kt) in alluvium, and Danny Boy (0.42 kt) in basalt, plans were being made for two major nuclear cratering experiments in the Plowshare Program. These were the Buggy row-charge experiment and the Schooner single-charge experiment. Buggy was originally planned for execution in alluvium near the Sedan crater, and Schooner was to be a high yield experiment similar to Sedan in a rock medium. There was a need for chemical explosive row-charge data in alluvium to permit the proper design of Buggy, and some additional chemical explosive cratering data in rock to establish the proper scaling to the larger yields for Schooner.

Pre-Buggy I and II had primary objectives of determining (1) the venting of radioactive tracers from row-charge cratering detonations compared to single-charge cratering detonations, and (2) the proper spacings and burial depths for charges in a row configuration and for a follow-on connecting row. The test results showed venting for row charges not to be greater than a factor of 2 larger than for single charges, and established the proper spacing between charges in a row to be approximately the radius of the crater produced by a single charge at optimum depth of burial. Neither of these results were very conclusive.

Pre-Schooner I and II had primary objectives of establishing good cratering curves in rock and determining the proper yield scaling to permit the design of Schooner. Pre-Schooner I established a satisfactory cratering curve for a dry basalt using $Y^{1/3.4}$ scaling. While these tests were in progress it became evident that the experiments in alluvium were not providing information which would benefit the design of future nuclear projects. Therefore, the site for Buggy was changed to a rock medium and a need arose for chemical explosive row-charge data in rock. Out of this need came the Dugout experiment. It was executed near the Pre-Schooner I sites in a dry basalt. Dugout was conservatively designed with charge spacings a little less than the corresponding Pre-Schooner I single-charge crater radius. The resulting crater had dimensions somewhat larger than single-charge dimensions because of the close spacing, and it was concluded that the Pre-Buggy row-charge design results could be extended to dry rock.

The Schooner site was selected off the Nevada Test Site on the Bruneau Plateau in southwestern Idaho. Consequently, the Pre-Schooner II experiment was designed to provide a scaling point at a higher yield (85 tons) and to calibrate the medium at the Bruneau Plateau site, which was a rhyolite having properties somewhat different from the basalt on Buckboard Mesa. Pre-Schooner II was executed as planned and provided the needed information, but the Schooner experiment was delayed for other reasons and eventually executed at NTS in a layered medium somewhat different from both the dry basalt and the rhyolite in Idaho.

The explosive used in the Pre-Buggy, Pre-Schooner and Dugout experiments was nitromethane. There were two primary reasons for its use: (1) it is a liquid and can be emplaced in a spherically mined cavity through fill lines after all construction (including stemming of the access hole) is complete, and (2) it has excellent explosive properties with total energy slightly greater than an equal weight of TNT. However, the cost of mining and sealing a spherical cavity kept emplacement costs high in those experiments in which it was used.

A.6 PRE-GONDOLA, TUGBOAT, AND TRINIDAD EXPERIMENTS (see also Appendix B)

Following the execution of Pre-Schooner II, the Interoceanic Canal Feasibility Studies were initiated with data collection activities on site in Panama and Colombia. Because nuclear excavation was being considered, the Plowshare Program was aiming toward supplying design information needed for the feasibility study. During site investigations it was discovered that about 20 mi of the proposed nuclear Route 17 through Panama was in a saturated clay shale material. Consequently, a series of nuclear experiments to be named Gondola were planned for execution in a clay shale.

The Pre-Gondola experiments were to provide the preliminary data for this new medium to facilitate the design of the nuclear experiments to follow. The series had several objectives among which were the usual cratering curves and the extension of row-charge design criteria to a new medium, except that in this instance the rows would be through varying terrain, would involve charge yields of varying sizes, and successive rows would connect to each other. Further, the rows were designed to provide a small navigation prism as an experimental exercise in producing a useful channel. The site selected for this work was Fort Peck, Montana.

Several explosives were used during the course of Pre-Gondola. Spherical-shaped charges of nitromethane in mined cavities was used on two of the large row detonations. Prior to the reservoir connection experiment, small-scale experiments were conducted to compare cratering the properties of several explosives and to determine the cratering effects of changing the charge shape from spherical to cylindrical and of varying the length-to-diameter ratios. These investigations resulted in the use of aluminized ammonium nitrate slurry charges in cylindrical cans with length-to-diameter ratios up to 2:1 for the reservoir connection experiment.

The extraordinary success of this series of experiments has led to the current program of chemical explosive excavation. Because of the lessons learned and because it is such a graphic depiction of several applications, the Pre-Gondola row-charge cratering work is described in detail in Appendix B. The two more recent experiments discussed below also included field tests which demonstrate the capability of explosive excavation. The demonstration aspects of these projects are also described in detail in Appendix B.

Project Tugboat provided cratering experience in a medium overlain by water. For several years there had been much talk about using large yield explosives to dig offshore harbors, but there were no experimental data to substantiate the idea. After the experience gained at the Pre-Gondola tests, it was determined that the next experiment should be one that would benefit a planned civil works project. The small boat harbor at Kawaihae Bay, Hawaii, was picked, and Project Tugboat was designed to provide the initial portion of the harbor. Project Tugboat provided single- and row-charge experience with a coral material in shallow water. A new square-array charge configuration was used to excavate the berthing basin.

The Trinidad Dam and Lake Project near Trinidad, Colorado, a civil works project under construction by the Albuquerque Engineer District, provided an opportunity to conduct cratering experiments in a layered dry sandstone material. This construction project includes relocation of a railway line which involves more than 40 sidehill cuts. The need for crater data in sandstone and the potential of explosive excavation for railway cuts prompted NCG to execute an extensive series of tests at the site during 1970. The tests at Trinidad (Project Trinidad) included the following objectives:

(1) To investigate the cratering characteristics of interbedded sandstone and shale

(2) To investigate row-charge cratering in nonlevel terrain

(3) To compare ground motion effects of single- and row-charges

(4) To compare the ground motion effects and cratering efficiencies of simultaneous and delayed row-charge detonations

(5) To compare the effectiveness and relative economics of two different explosives

(6) To test the feasibility of creating explosive emplacement cavities by means of "springing" charges detonated in small diameter access holes

(7) To develop a design and to execute a railway cut excavation on the new alinement of the railroad as a demonstration project

Although analyses are not yet complete, the work at Trinidad was highly beneficial from a cratering research standpoint and very encouraging in the progress toward making explosive excavation more cost-competitive with conventional methods. The design for the railway cut produced an excavation very nearly identical to the size and shape predicted (see Appendix B for discussion of this project). Follow-on tests are planned for 1971 at the Trinidad site.

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Appendix B
Demonstration Projects

B.1 INTRODUCTION

Three recent field experiments are discussed in this appendix to portray the capability of explosive excavation, and to explain in further detail the development of design techniques and the principal operational aspects of project execution. The Pre-Gondola III, Phase III reservoir connection, Project Tugboat, and the Project Trinidad railway cut experiments have been classified as demonstration projects because they illustrate the use of explosive excavation for projects of the type discussed in Chapter 2, specifically, canals and/or watercourses, harbors, and railway cuts. These demonstration projects mark the forward edge of the development of explosive excavation technology as set forth in this report.

B.2 PROJECT PRE-GONDOLA ROW CRATERS

a. General

Pre-Gondola is the name for a series of chemical explosive cratering experiments conducted by the Nuclear Cratering Group from 1966 through 1969 near Fort Peck, Montana. The experiments were designed to establish the cratering characteristics of saturated clay shale, to acquire row-charge cratering experience, and to demonstrate the feasibility of connecting an existing row crater to a body of water. A listing of both the single-charge and row-charge cratering experiments in the Pre-Gondola series is given in Appendix A. Only the three large row craters produced during the course of this work will be discussed in this section because these adequately trace the testing of various design concepts used at Fort Peck and best illustrate the excavation possibilities with large explosive charges for canals, certain harbors, and other water-edge-related construction projects. An aerial photograph showing two interconnected row-charge craters and the location of the third (Pre-Gondola III, Phase III reservoir connection) is shown in Fig. B1.

b. Pre-Gondola II

The first row crater excavation at Fort Peck was designated Pre-Gondola II and was detonated on 28 June 1967. The objectives of this 5-charge, 140-ton (nitromethane) experiment were to gain row cratering experience in clay shale, to determine the effects of using varying charge weights in a row, to test techniques for connecting a linear crater to an existing single-charge crater, and to attempt overexcavation at one end of the row for the purpose of accepting throwout from a follow-on connecting row crater. The Pre-Gondola II crater was connected to the shore side of the Pre-Gondola I Charlie crater.

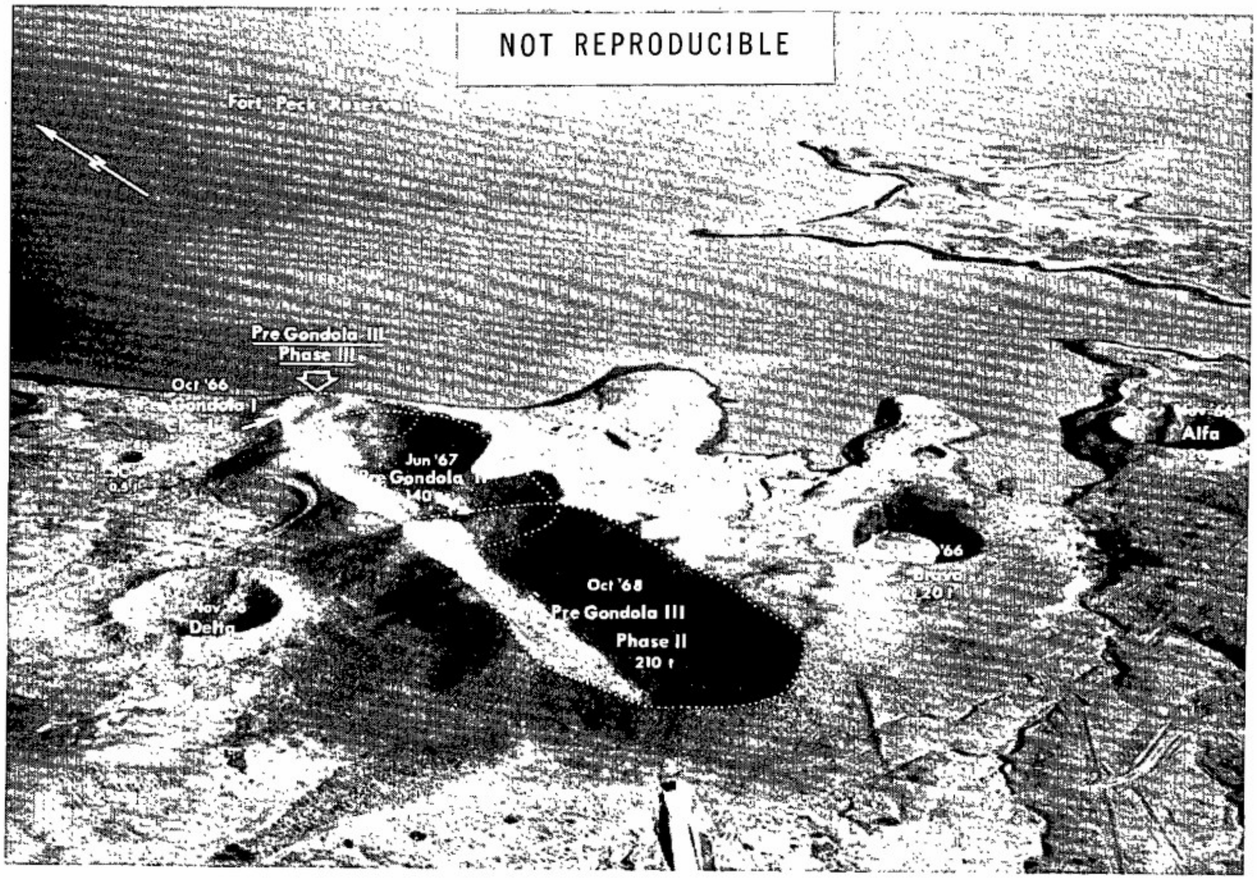


Fig. B1. Aerial view of Pre-Gondola row-crater site at Fort Peck, Montana before October 1969.

The design objective of the experiment was to excavate explosively a navigable channel 67 ft wide and 4 ft deep at a reservoir pool level of 2238-ft elevation. It was based on the concept that the same scaled depth of burial could be used for different size charges in a row. Charge containers were prepared by drilling 38-in. diameter access holes and mining out spherical cavities. A pneumatically applied mortar was used to line the cavities and to obtain the spherical shape within prescribed tolerances. A silicon-rubber membrane was bonded to the mortar lining making the cavities leak-proof. The access holes were stemmed with a concrete mixture designed to match the strength properties of the <u>in situ</u> shale. The spacing between charges was 80 ft, or approximately 1.0 R_a for a 20-ton single charge at the same depth. The charges were buried about 15% deeper than optimum for a single charge because it was anticipated that some interaction of the charges, termed "enhancement," would occur. The top cross section of Fig. B2 shows the charge configuration for Pre-Gondola II, the existing cross section of the Charlie crater, and the final profile.

Individual charge weights were 20 and 40 tons. The weight of connecting Charge E was selected as 40 tons to provide sufficient mound velocity to crater the hump of lip material. It was believed for this varying terrain situation that a 20-ton connection charge might have been marginal. At that time the assumed charge

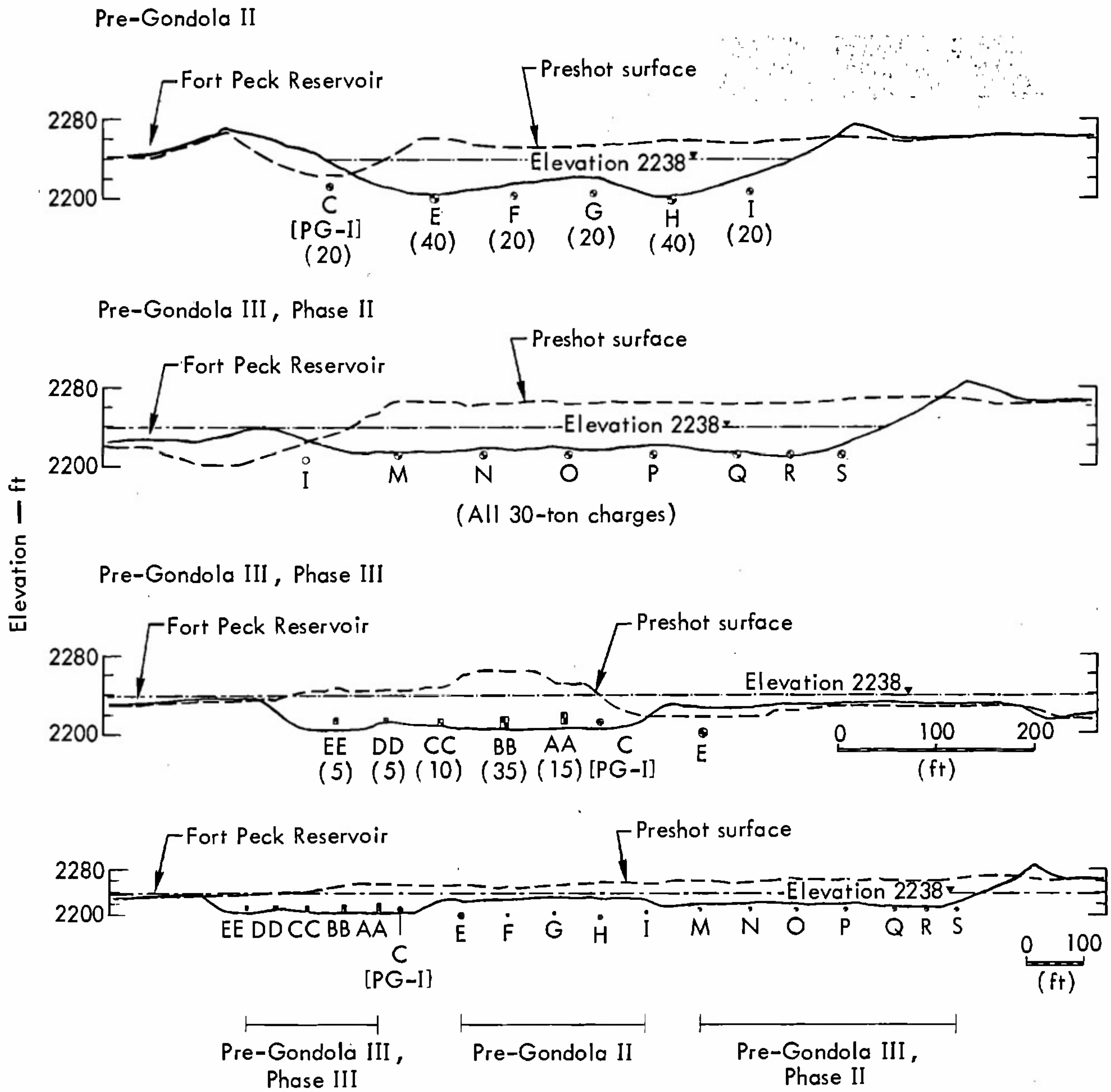


Fig. B2. Cross sections and charge emplacement for Pre-Gondola row craters.

increments for the family of nuclear explosives in the Plowshare Program was a factor of 2 and, therefore, a 40-ton charge was selected to maintain the nuclear model. The weight of Charge H was selected as 40 tons to attempt to overexcavate a section to receive the expected ejecta from the proposed Pre-Gondola III connecting row experiment. It was anticipated that the crater would be slightly deeper near the ends and that a considerable amount of ejecta would be deposited in the Charlie crater.

The experiment was successfully executed; however, it was difficult to relate the dimensions of the crater to charge size because of the presence of both 20-ton and 40-ton charges in the row. Subsequent row-charge experiments have shown that a more efficient excavation of material would have been achieved had a larger spacing

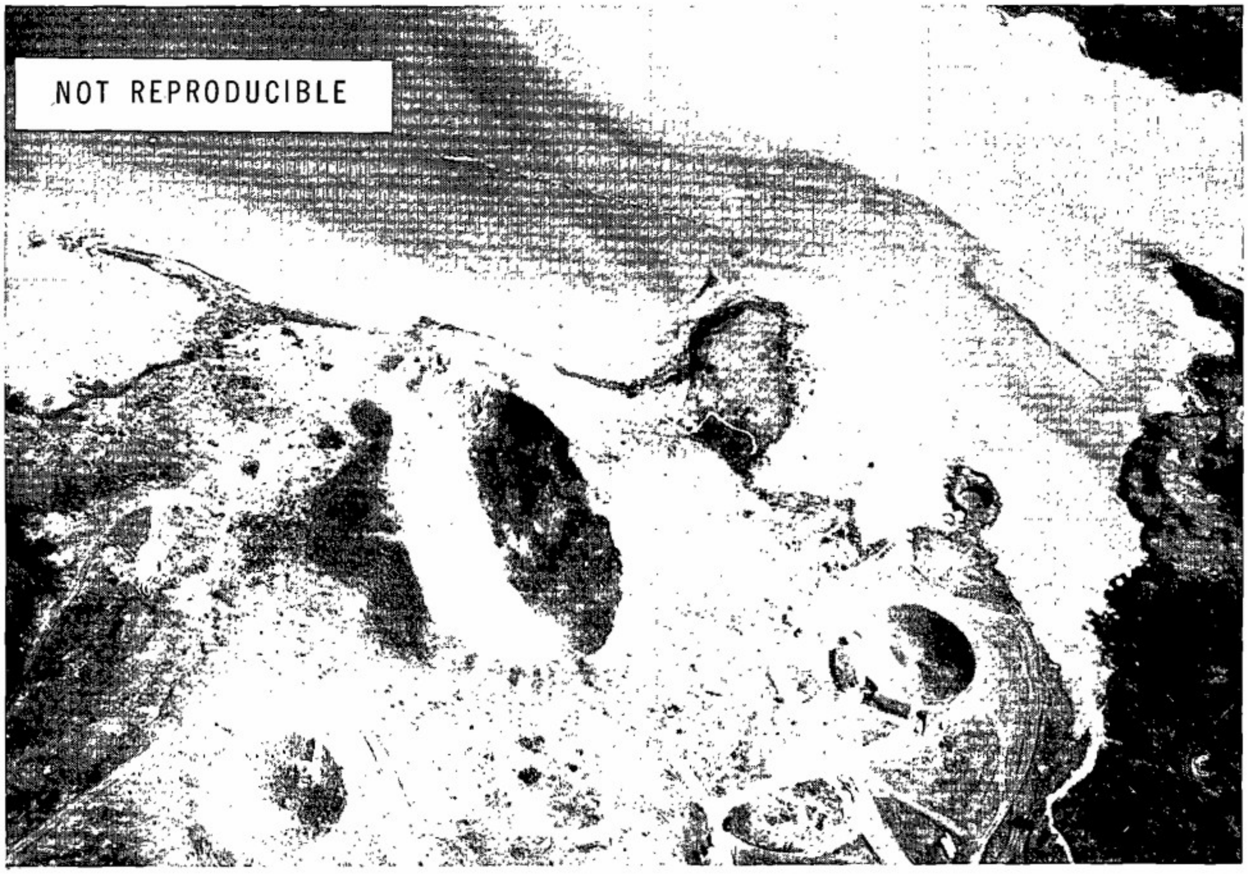


Fig. B3. Pre-Gondola II row crater in June 1967.

been provided between the 40-ton and 20-ton charges or had the 40-ton charges been placed at a greater depth of burial. The Pre-Gondola II row-charge experiment did demonstrate the feasibility of (1) connecting to a preexisting single-charge crater, and (2) employing different charge weights to overexcavate a portion of a row. Significant data were also obtained on seismic and airblast effects from a row-charge configuration. A picture of the Pre-Gondola II row crater is shown in Fig. B3.

c. Pre-Gondola III, Phase II

Based on the experience gained in Pre-Gondola II, it was decided to conduct a Pre-Gondola III, Phase II experiment designed to connect one row crater to another in clay shale. (Phase I of Pre-Gondola III was an experiment that tested a technique for obtaining craters with very flat side slopes.) At this time the only connecting row-charge cratering experiments that had been conducted under the joint Corps of Engineers—Atomic Energy Commission experimental program was the Pre-Buggy II series using 1000-lb nitromethane charges in alluvium.

To reduce some of the variables, the Pre-Gondola III, Phase II experiment was designed with equal weight charges buried at the same elevation. The charge increment of two had since been dropped and therefore the actual weight necessary to excavate for the 67 × 4-ft navigation prism was determined. The row consisted of seven

30-ton charges of nitromethane placed in the same manner as the previous experiments. From the cratering calibration data for the clay shale, it was decided that 30-ton charges at a nominal scaled burial depth of 150 $ft/kt^{1/3.4}$ would produce a crater of the necessary width and depth.

The design configuration of Pre-Gondola III, Phase II is shown in Fig. B2. The charges were placed at elevation 2210 ft with resulting burial depths ranging from 50 to 54 ft. The horizontal charge spacings were one crater radius for a single 30-ton charge at the burial depths actually used, except at the ends. Connecting Charge M was placed at a distance of approximately 1.1 crater radius from the nearest previous charge position, I, of the existing row crater. Laboratory-scale modeling experiments in a compacted sand medium had indicated that this spacing should be optimum for a smooth row crater connection. To provide an overexcavation at the end farthest from the connection point, the last three charges (Q, R and S) were spaced at 0.6 crater radius.

The detonation on 30 October 1968 produced an exceptionally smooth crater having an apparent length of 610 ft, an average width of 121 ft, and an average depth of 48 ft. In combination with the preexisting craters, it formed an excavation approximately 1100 ft long. The desired cross section was achieved; however, a mound of ejecta was deposited in the existing Pre-Gondola II crater and no significant overexcavation was achieved at the south end. Subsequent experiments have shown that the three charges at this location were not buried deep enough for the spacing used and that a greater depth of burial would have provided overexcavation. This experiment provided the first row-charge crater dimensions which could be compared directly with those from a single charge. Figure B4 shows the Pre-Gondola III, Phase II row crater.

d. Pre-Gondola III, Phase III Reservoir Connection

The Pre-Gondola III, Phase III reservoir connection experiment marked a distinct departure from the concept of nuclear modeling and served as the first demonstration project of chemical explosive excavation. The purpose of the project was to excavate a channel capable of accommodating a navigation prism through a hump of terrain separating the Pre-Gondola II and III interconnected row craters and the Fort Peck Reservoir. This was the first large scale row-charge experiment by NCG in varying terrain. An important secondary objective was to find ways of reducing costs associated with explosive excavation.

The project requirement was for a 67- × 4-ft navigation prism with the bottom of the prism at elevation 2234 ft, the same design objective used for the Pre-Gondola II and III row craters. Because of the very irregular topography and steep slopes through which the channel was to be excavated, existing row-charge design procedures were only partially applicable and considerable judgment was used in developing the explosive excavation design. The key to the design was the charge beneath the point of deepest cut. The size of this charge, Charge BB, was computed on the basis of having to

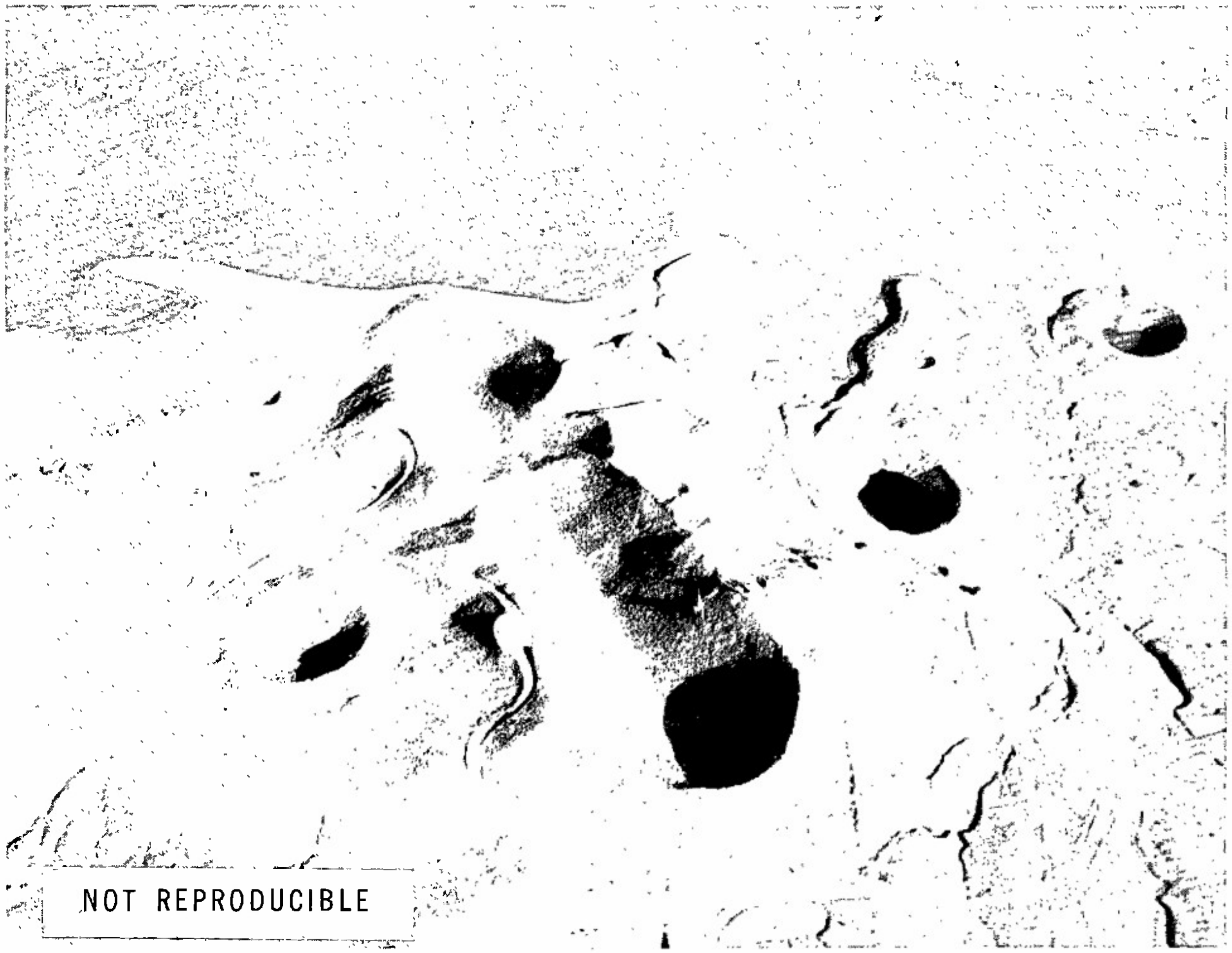


Fig. B4. Pre-Gondola III, Phase II row crater (foreground) connected to Pre-Gondola II crater, October 1968.

excavate a crater with a width of 67 ft at elevation 2234 ft, the bottom of the navigation prism. This computation was made by generally following the procedures now available in Chapter 5 and rounding the charge weight up to the nearest multiple of 5 tons. Adjacent charges were placed one crater radius out from Charge BB, computed to obtain the required width of cut at elevation 2234 ft, and rounded up to the nearest 5-ton increment. This procedure was repeated for the other charges. Charge AA was increased an additional 5 tons in order to increase the ejecta velocity of the material above it. The design as developed used a total of 70 tons of explosive in five charges of varying weights buried at varying depths. The charge layout and resulting crater cross section are shown in Fig. B2.

Since the Pre-Gondola III, Phase III reservoir connection was the first demonstration project of chemical explosive excavation as well as an experiment, an important objective was to reduce construction costs. Methods of accomplishing this included the use of cylindrical charge configurations with length-to-diameter ratios between 1:1 and 2:1 in place of the aluminum spheres used in all previous experiments, and the use of simplified stemming techniques. The concrete cylindrical containers were lowered to the bottom of the access shafts that were excavated by a crew using a bucket auger (Fig. B5). Steel liner plates (emplaced by the crew for safety purposes as the shafts were dug) were removed as the shafts were backfilled and compacted with material on site. None of these measures adversely affected the excavation. In fact, considerable overexcavation was achieved because the cratering effectiveness of the explosive was greater than that assumed in the design.

A metallized ammonium nitrate slurry blasting agent was used on the reservoir connection experiment instead of nitromethane (which had been used on all previous Pre-Gondola detonations) as a further effort toward reducing costs and engineering problems encountered in attempting to emplace large quantities of nitromethane explosive in ejected and fractured shale. The slurry proved an excellent explosive for this type of project because it is pumpable; it is compatible with and has a density greater than water, which allows it to displace water when pumped into a wet hole; and when mixed with a cross linking agent, it sets up a stiff gel consistency which may be left down-hole for days with no ill effects. The cratering effectiveness on a volume basis was greater than that of any explosive previously used.

The 70 tons of slurry were detonated on 6 October 1969 and produced a crater larger than that required by the design. Water was observed entering the crater

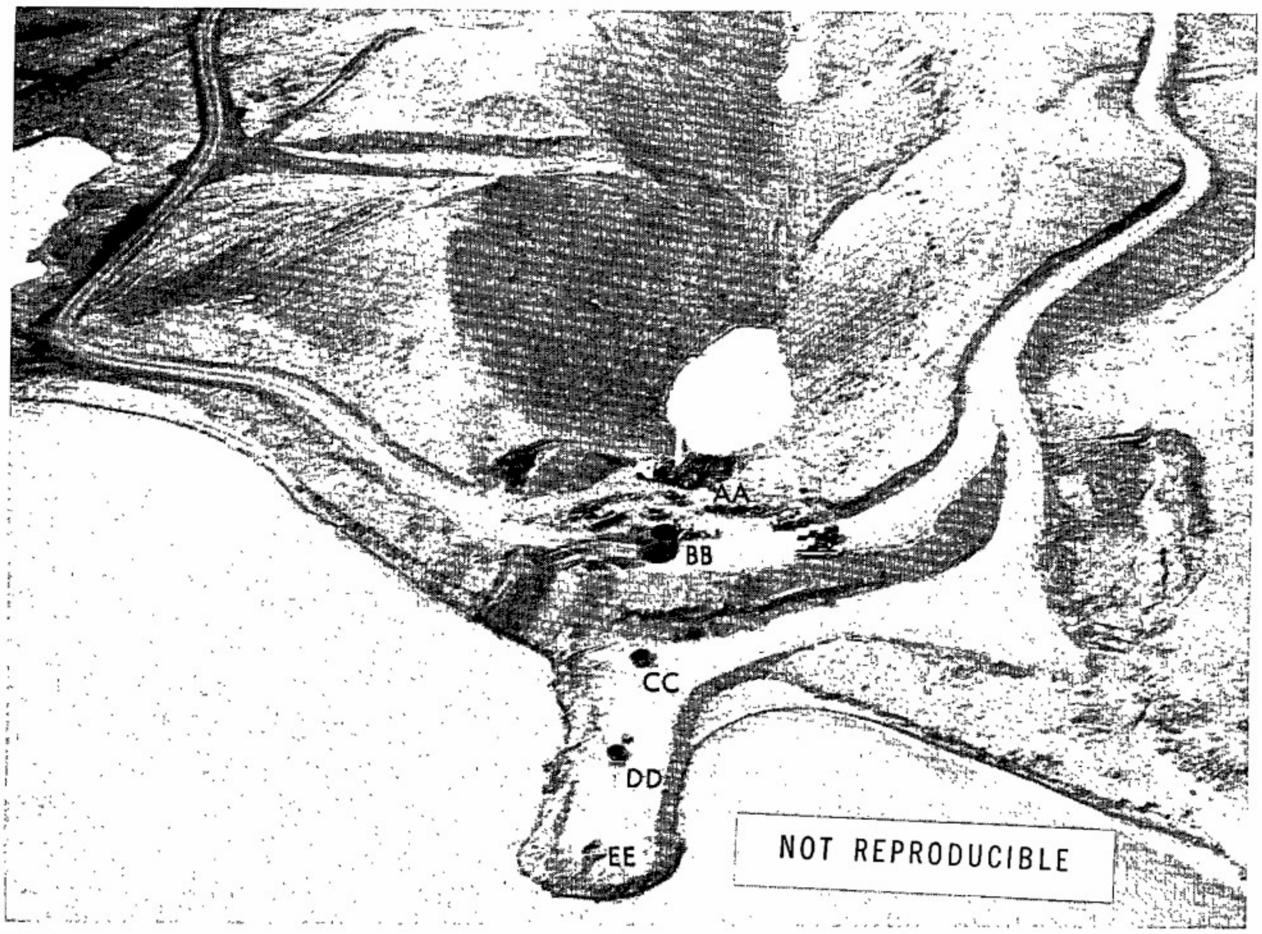


Fig. B5. Drilling for emplacement of charge AA, 15-ton charge (steel liner plates are in place for other four charges).

excavation after the base surge cloud cleared (Fig. B6), and the entire Pre-Gondola channel was filled in approximately 9 min. The crater formed by this detonation was approximately 400 ft long and averaged 150 ft wide with a water depth of 25 ft at centerline. Cross sections through each charge location and a longitudinal profile along the centerline are shown in Fig. B7.

e. Summary

The total Pre-Gondola water-filled channel, which includes the three row craters discussed in this section, is approximately 1370 ft long. The crater width at water level varies from a minimum of 100 ft to a maximum of 200 ft. The depth of water at the centerline varies from a minimum of 13 ft to a maximum of 39 ft, except at the entrance where the depth is approximately 7 ft. The final postshot configuration resulting from the three detonations is shown in Fig. B8.

In addition to providing the modeling data necessary to design a nuclear excavation detonation in similar material, considerable basic information on chemical explosive excavation was obtained from the Pre-Gondola row crater experiments in clay shale at Fort Peck. The technical programs provided valuable information related to

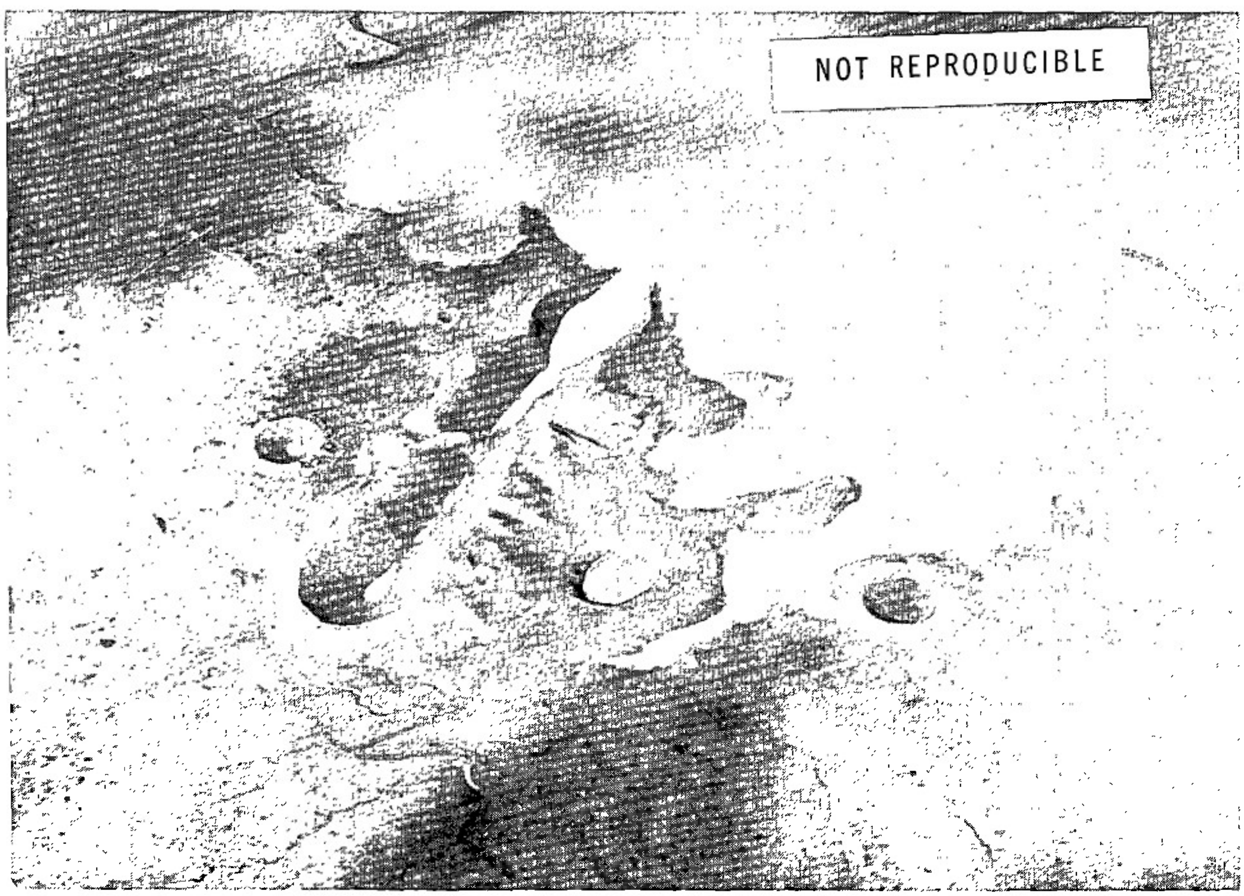


Fig. B6. Water filling Pre-Gondola row craters after reservoir connection detonation, 6 October 1969.

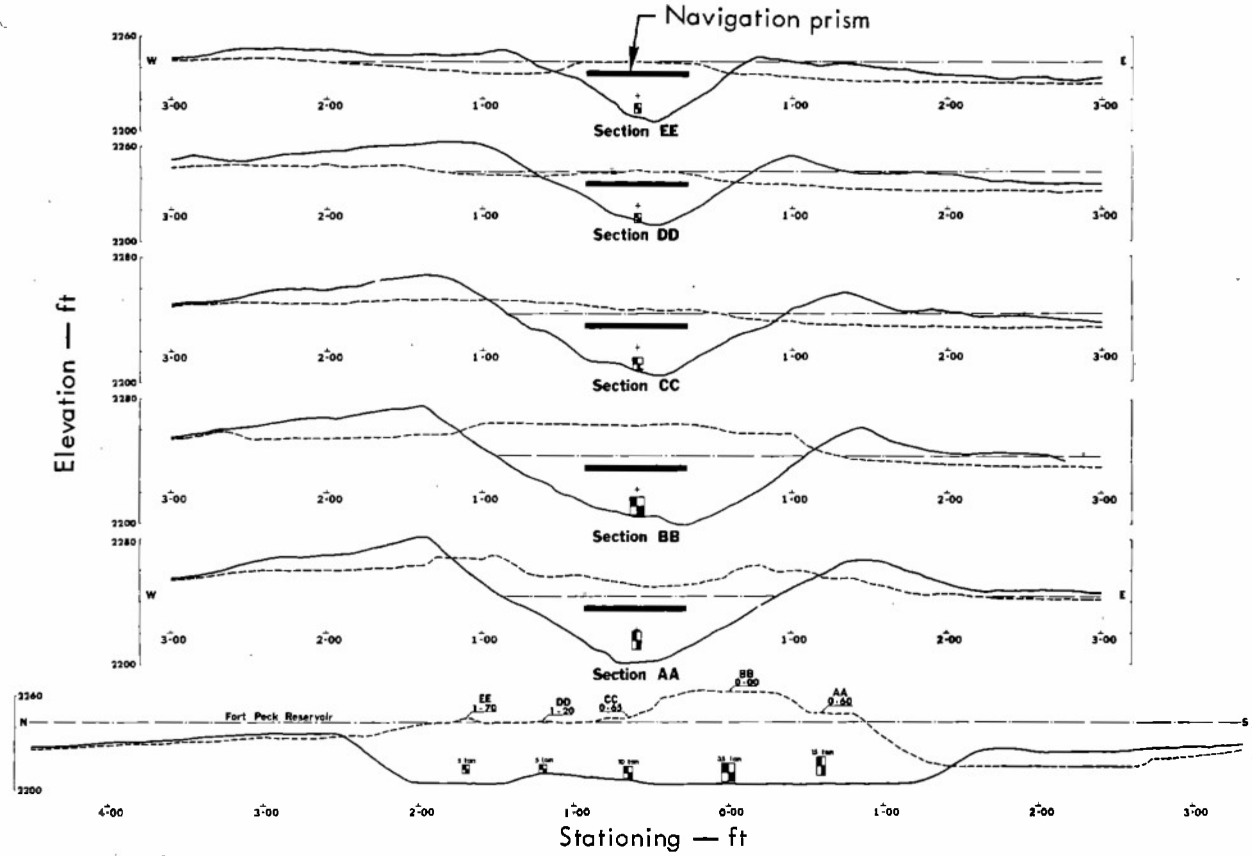


Fig. B7. Cross sections and longitudinal profile through Pre-Gondola III, Phase III reservoir connection.

airblast and ground motion effects, and significant information was obtained on the direction and magnitude of movements of material in the rupture and fallback zones.

Row cratering design techniques were refined to include a near full scale demonstration of a design for excavating through terrain of varying elevations. Experience was gained in connecting row craters; however, additional experimental work is still required to insure smooth connections. Construction, emplacement, and firing techniques were simplified and streamlined as the series progressed, providing for faster, safer, and more economical operations. The cratering effectiveness of the aluminized ammonium nitrate slurry used on the reservoir connection project proved to be about 1.6 times that of TNT. The Pre-Gondola row crater experiments provided important practical data in the development of explosive excavation technology.

B.3 PROJECT TUGBOAT

a. General

Tugboat is the name for a project carried out by the Nuclear Cratering Group in 1970 to demonstrate the practical application of explosive excavation to harbor construction. Joint Federal-State plans called for the construction of a small boat harbor offshore at Kawaihae Bay, Hawaii (Fig. B9). The harbor would consist of berthing basins connected to the open sea by an entrance channel.

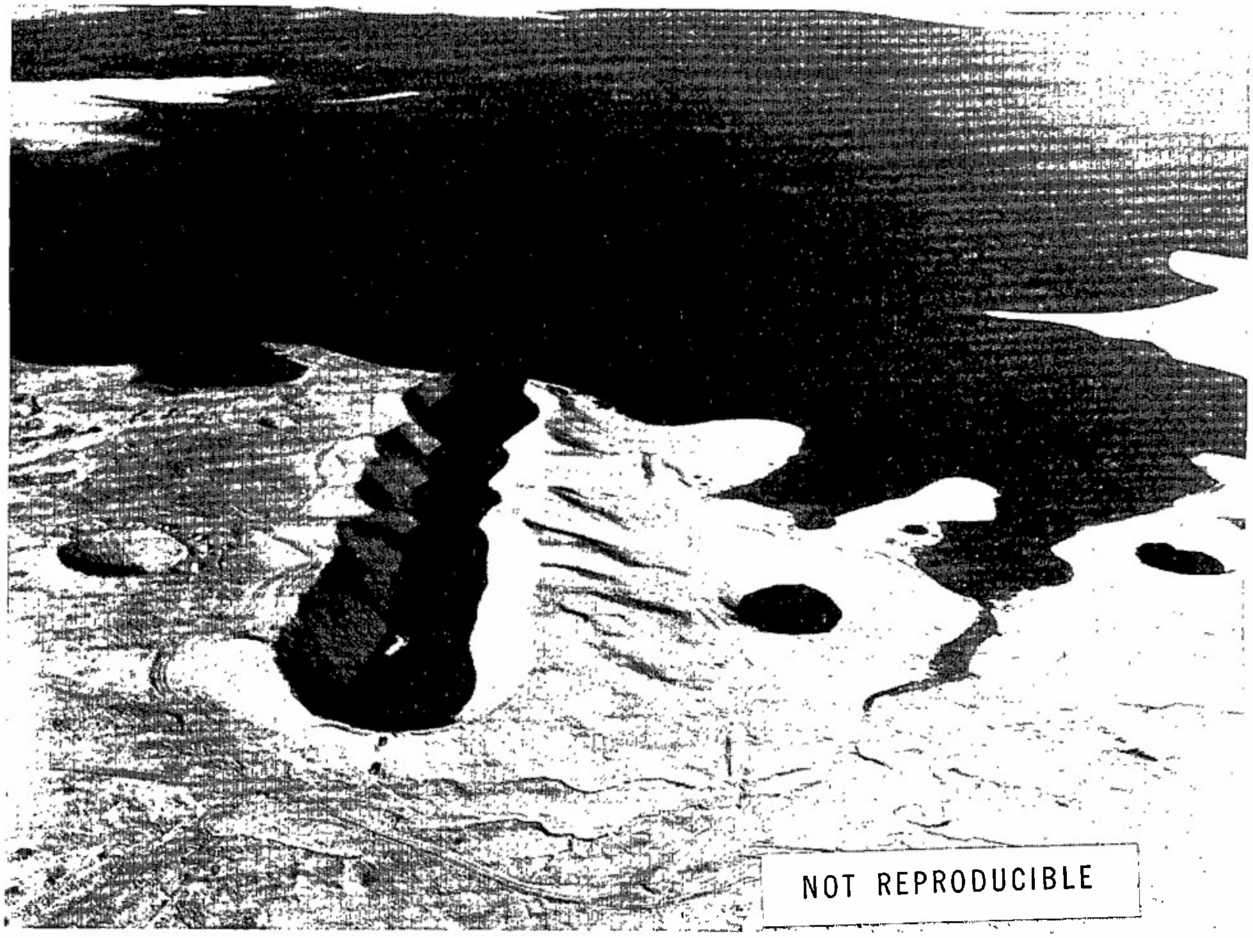


Fig. B8. Pre-Gondola row craters connected to Fort Peck reservoir (boat in channel is 42-ft tugboat).

It was decided that a large berthing basin (240 by 240 ft; 12 ft deep), and the principal entrance channel (120 ft wide and 12 ft deep) would be excavated using explosive cratering techniques, the excavation to consist of an underwater trench connecting deep water offshore to a large diameter crater closer inshore. The harbor would be expanded when Congress and the State provide the additional funds needed. Until that time, the berthing basin would be protected from the effects of ocean waves and currents by a breakwater construction on the fractured material. An aerial view of the project site, with an outline of the berthing basin, entrance channel, and breakwater, is shown in Fig. 13. An artist's cônception of the completed harbor project is shown in Fig. B10. Project Tugboat marks the first deliberate attempt to advance explosive excavation into a practical role; to conduct a prototype field experiment designed to test and to increase knowledge of explosive excavation and at the same time serve the functional needs of a civil works project.

b. Scope of Project

Tugboat was first planned to be executed in three phases preceded by a site investigations program. The investigations program provided site data and information

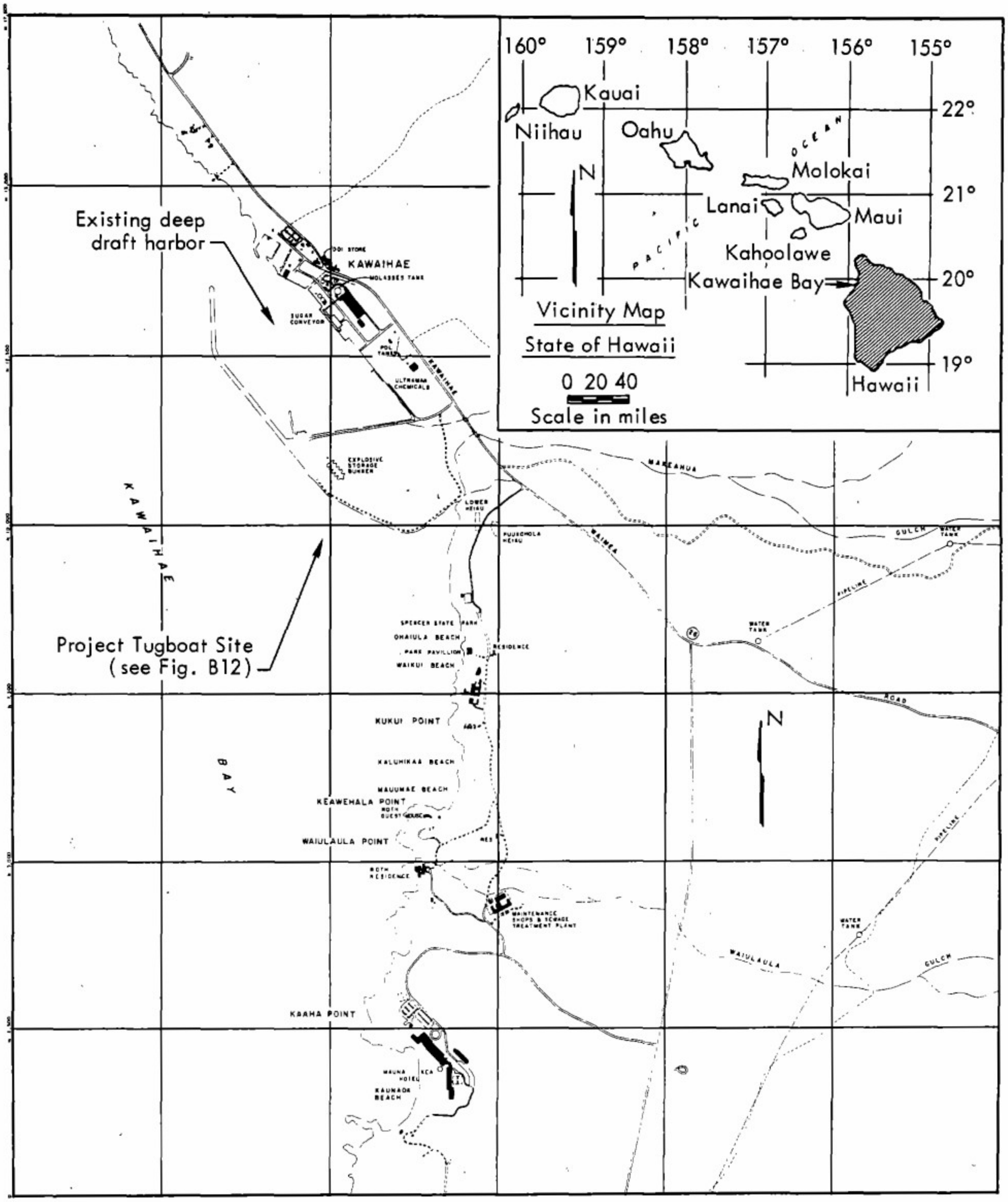


Fig. B9. General site area, Project Tugboat.

needed for excavation design and safety program definition. Phase I, Safety Calibration Series, involved the detonation of five cratering charges to provide airblast and ground shock data in the vicinity of the project site. These data were needed to determine the maximum quantity of explosive that could be safely detonated. The Phase I detonations also provided crater dimension data for underwater detonations in the coral

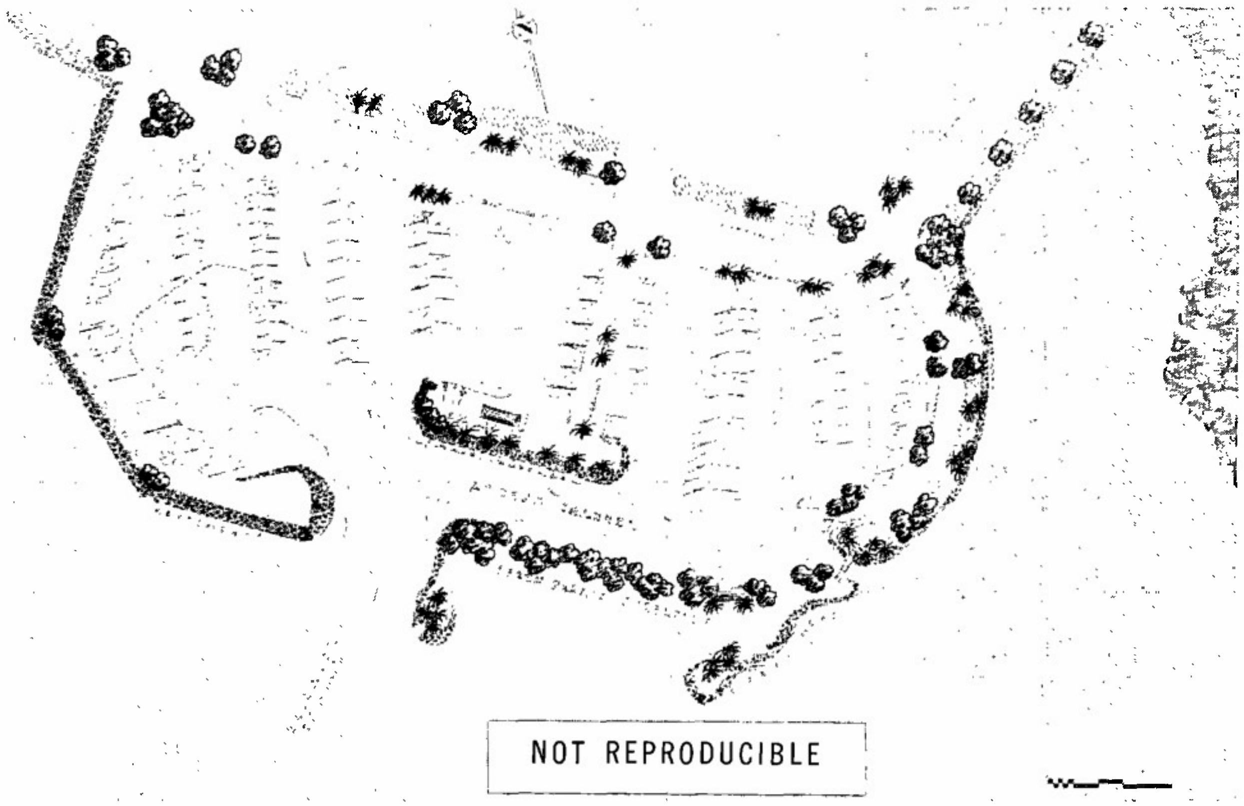


Fig. B10. Conceptual drawing of completed small boat harbor project, Kawaihae Bay, Hawaii.

medium to be used for design of the follow-on phases of the project. Phase II was initially planned as a row-charge entrance channel excavation with a total aggregate yield of approximately 100 tons of explosives to be followed by Phase III, a double row or an array-charge berthing basin excavation also using a 100-ton detonation.

Phases II and III were changed based on the results of Phase I. The revised design specified three individual 40-ton detonations for Phase II, followed by postshot engineering properties investigations and the construction of a breakwater near the lip of the berthing basin crater for Phase III.

c. Site Investigations

A number of programs were undertaken to determine specific types of site information and to provide topographic and hydrographic maps of the area. The general scope of each program is described below and a summary of results provided where appropriate.

(1) Geophysical Surveys

Geophysical refraction surveys were accomplished to provide interpretation of shock wave velocity and geological conditions to a depth below the region of interest (to 50 ft below bottom elevation). Soundings were made along the length of the entrance channel and in the berthing basin area. The medium was found to be coral to depths ranging from 68 to 111 ft below mean lower low water (MLLW). No basalt flows were detected in the region of interest (to 50 ft below bottom). Seismic velocities in the upper portion of the coral range from 5120 to 6660 ft/sec. The peak velocity measured in the coral was approximately 7240 ft/sec through a single continuous reef of coral.

(2) Hydrographic and Topographic Mapping

The bottom elevation and ground surface were surveyed and mapped in the detonation area. Water depths were found to be highly variable. The surface of the coral was found to be very irregular with maximum variation from high point to low point of about 10 ft.

(3) Geologic Investigations

This program included drilling, sampling, and laboratory testing. A series of 15 exploratory holes were drilled. Of a total of 687 ft of hole drilled, 123 ft of core was recovered. Only three pieces of core measured more than 1 ft in length. The coral material had an average dry density of 1.4 g/cm^3, and an average wet density of 1.8 g/cm^3. When crushed the coral had a dry density ranging from 2.2 to 2.8 g/cm^3. Compressive strengths ranged from 760 to 1738 psi. Porosity of the coral material was estimated to range from 37 to 64%. The material was classed as coral limestone; however, x-ray diffraction indicated that the coral was primarily aragonite. It was found to be relatively loose and soft throughout with voids and cavities comprising as much as 30% of stratigraphic sections.

(4) Archeological Explorations and Mapping

All archeological artifacts in the harbor area were located and mapped. These include two heiaus on the hill overlooking the site, Puukohola and Mailekina; a third heiau that is underwater and partially covered with silt; a stone seat adjacent to the shoreline; and ruins of the walls of John Young's principal house near the Puukohola Heiau. (The Puukohola and Mailekini Heiaus were protected by providing support with cable and timbers on corners and the steeper slopes prior to the detonations.)

(5) Wave Analysis

Wave action in the Kawaihae Bay area was analyzed to provide criteria for design of the explosively excavated harbor. The theoretical maximum waves anticipated to act on the breakwater are between 6 and 8.5 ft. The maximum wave height that can be expected at the mouth of the proposed entrance channel is 18 ft (nonbreaking wave). The critical direction and periods of waves are N67°30'W with a period of 8 sec for the breakwater, and N67°30'W with a period of 15 sec at the entrance of the channel.

(6) Site Meteorology

Meteorological data for the site were collected from records of the U. S. Weather Bureau and from records kept by the Corps at the existing Kawaihae deep draft harbor. The surface winds at Kawaihae blow either from the west or the east about three-fourths of the time. Westerly winds begin between 0900 and 1000 hr in the morning and usually last until 1900 to 2200 hr in the evening. The winds then shift to easterly and last until 3 to 4 hr after sunrise. During wind shift times, the winds are light and variable. Average temperature and rainfall data were collected from available records and studied prior to scheduling the detonations.

d. Phase I, Calibration Detonations

Phase I of Project Tugboat was executed during the week of 3 to 7 November 1969. Four 1-ton charges and one 10-ton charge were detonated individually. The sizes of the resulting craters were measured and used to determine the charge layout for Phase II. Seismic and airblast measurements made during Phase I were used to determine the maximum number of charges that could be safely detonated simultaneously during Phase II. Other notable data collected in conjunction with Phase I included fish kill information and wave height measurements.

Contrary to expectation, none of the craters produced during Phase I had lips which stood above water level. For each of these underwater cratering detonations the craters produced were shallower and of larger diameter than would be produced in a dry land medium. The characteristic shape results principally from consolidation of the fragmented unconsolidated coral. Material which would normally be thrown out and deposited around the crater (lip), in this case collapses, or is washed back into the deep portion of the crater. In this way, very wide, shallow craters are formed. These craters are well-suited to harbor excavations. The crater profile of the 10-ton detonation is shown in Fig. B11.

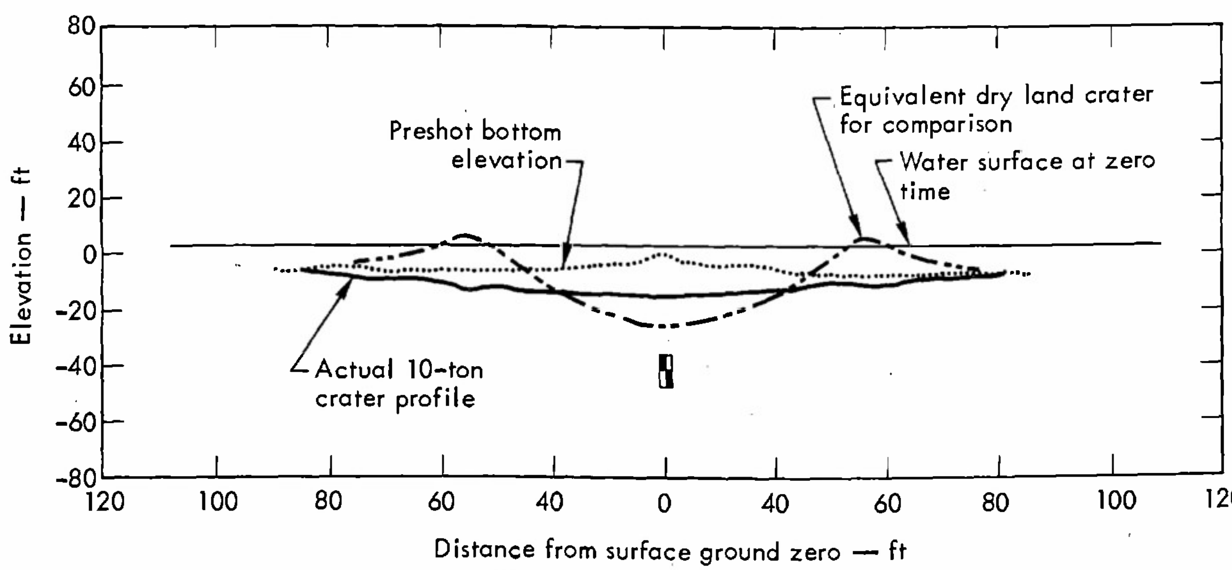


Fig. B11. Postshot crater profile, 10-ton detonation, Phase I, Project Tugboat.

e. Final Design

Several attractive possibilities for accomplishing the berthing basin and entrance channel excavations became apparent upon preliminary review of the Phase I data. These included accomplishing the entire project with a single detonation of about 100 tons or using two, three, or more lower yield detonations to further diminish the likelihood of seismic damage. It was clear that the total yield required was very much less than originally anticipated and that a revision of the initial design was in order.

Analysis of the seismic motion and structures response data led to a recommendation that the detonation yields be limited to 40 tons to insure a wider margin of safety and guarantee negligible risk of damage. This recommendation was accepted and a revised design prepared.

The new design called for three detonations of 40 tons each; each detonation to consist of four 10-ton charges buried 42 ft below MLLW. The first detonation included the four outermost charges in the entrance channel which were to be spaced 100 ft apart and detonated simultaneously. The second detonation included the remaining four charges in the entrance channel to be spaced 120 ft apart and sequentially detonated with 100-msec delays between successive charges. The third detonation was to be the simultaneous firing of four charges in a 120-ft square array for the berthing basin excavation. The charge layout pattern for Phase II is shown in Fig. B12.

f. Phase II (revised), Excavation Detonations

Figure B13 is a sketch showing certain details of the in-place charges. Holes for the charge containers were drilled with a barge-mounted-bucket auger. The 4-in. emplacement pipe was used to fill the container with 10 tons of aluminized ammonium nitrate slurry blasting agent after backfilling the hole. Primers and booster charges were lowered through the emplacement pipe and pushed into the main charge.

For the first detonation three 1-lb commercial boosters were positioned at quarter-points along the axis of each charge; for the second detonation the three boosters were clustered near the center of the charge. Because of fractional yields from two of the four charges in the first detonation and failure of two of the four charges to initiate at all in the second detonation (all difficulties were believed to be caused by inadequate booster charges), the commercial booster system for the third detonation was replaced by booster fabricated on-site consisting of a 3-in. tube, 10 ft long with a detonating cord primer along its length and filled with plastic explosive, C-4. This improvised system proved satisfactory and all four charges of the third detonation went full-yield on 1 May 1970. The detonation is shown in Fig. B14.

Investigation of the two charges which failed to detonate revealed in one case (Charge H) that the electric system had functioned; i.e., the cap had fired and the detonating cord backup system had been destroyed in the process. Since the existing emplacement pipe was badly damaged, this charge was primed by first driving a new 4-in.

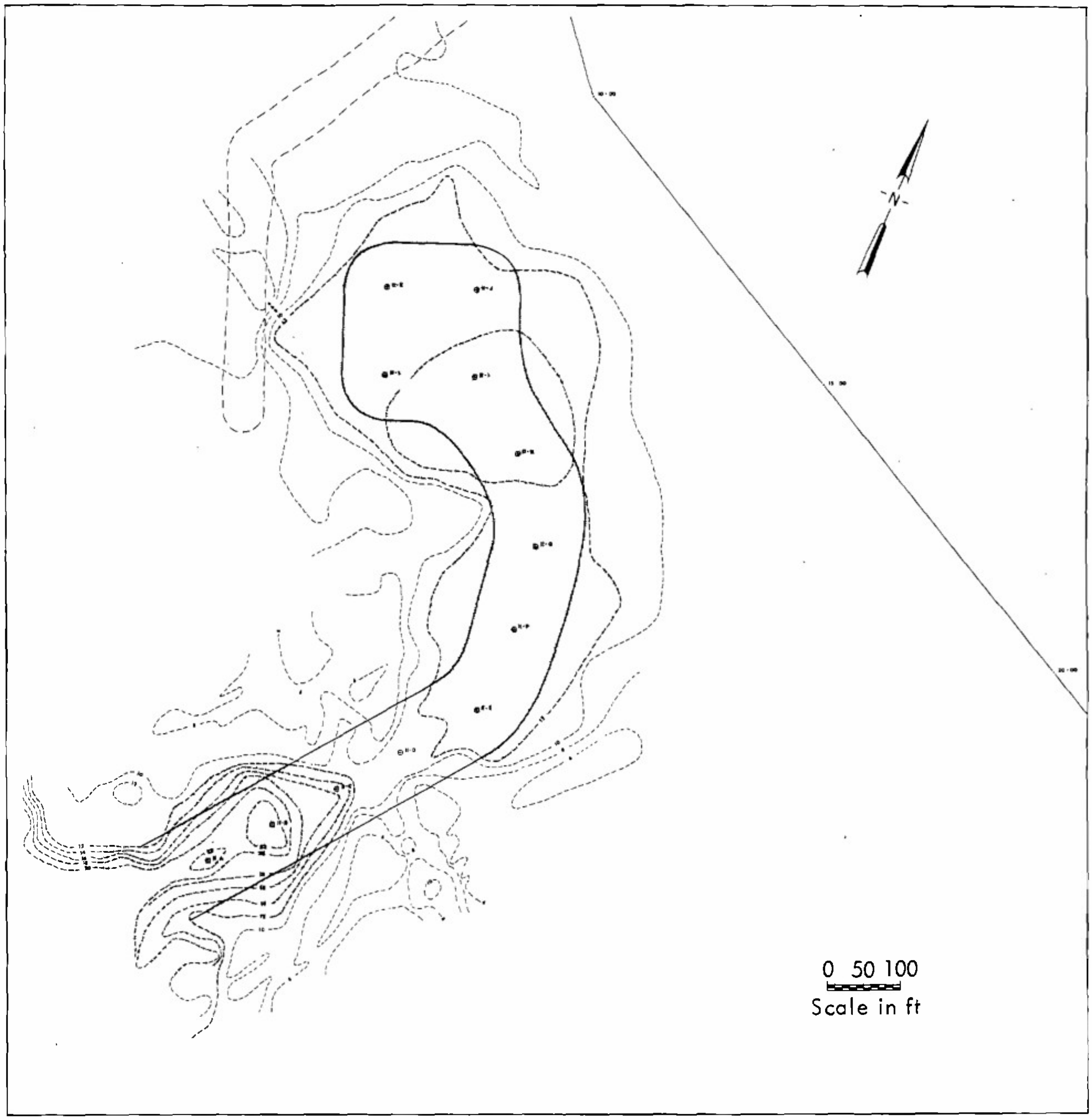
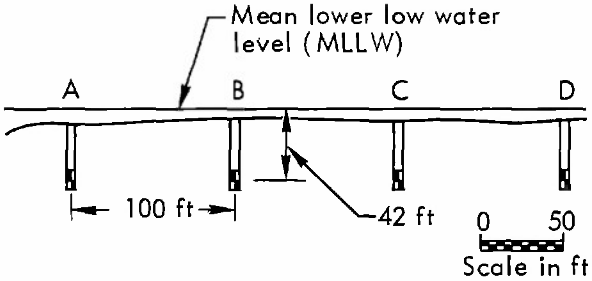


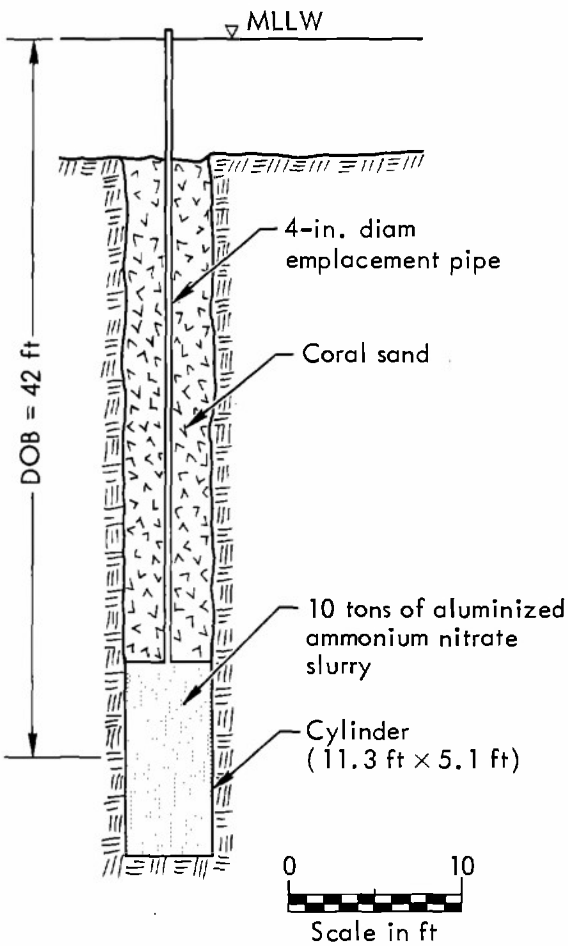
Fig. B12. Charge layout plan for Phase II and areas in which design depth was achieved by Phase II detonations, Project Tugboat.

pipe tipped with a sharp point down parallel to the old pipe and into the charge container. The tip of this pipe was knocked off and samples of the explosive were removed for testing. Then a priming charge made up similar to those which performed successfully on the third detonation was inserted into the pipe and pushed into the charge.

In the other case (Charge G) the electric system appeared not to have functioned, and the detonating cord system appeared to be intact. This latter system was rigged for firing in concert with the new primer for Charge H.



a. Section of first detonation



b. Typical charge section

Fig. B13. Charge emplacement details, Phase II, Project Tugboat.

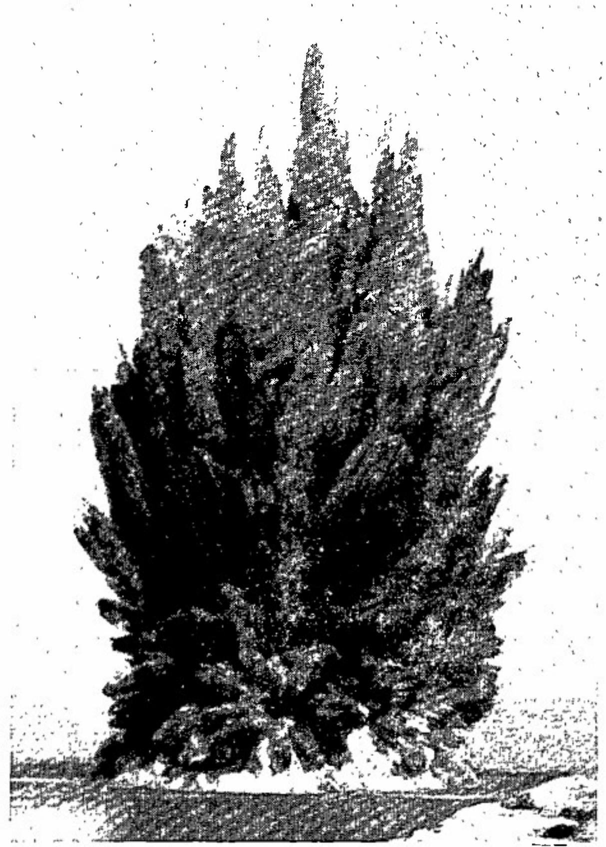


Fig. B14. Berthing basin detonation, Project Tugboat.

Upon firing, Charge H detonated at full-yield; however, Charge G did not. A prolonged vigorous bubbling noted over Charge G indicated that the slurry burned rather than detonated.

The degree to which the required project depth was achieved by the Phase II detonations is depicted by the shaded area in Fig. B12. Clearly the desired excavation would have been obtained had there been no malfunction in detonating the charges. Subsequently, the excavation deficiencies were remedied by using a total of sixteen 1/2-ton charges jetted into the bottom at places where the depth failed to meet the 12-ft criterion. The shaded area in Fig. B15 shows the area in which the design depth of 12 ft was finally achieved by explosive excavation.

Fig. B15. Planned area of excavation and final 12-ft depth contours after remedial detonations, Project Tugboat.

g. Phase III (revised), Postshot Evaluation and Construction

To complete Project Tugboat a breakwater to protect the basin excavation was constructed on the fractured material (see Fig. 13). The long-term behavior of this structure and of the cratered channels themselves will be studied.

h. Summary

Project Tugboat was a well-designed experiment plagued by unfortunate difficulties in firing some of the charges. These particular difficulties were corrected on-site to permit the final berthing basin excavation to go according to plan—and to achieve the expected results. Project Tugboat has confirmed the adaptability of explosive excavation, has shed more light on the practical aspects of project execution, and has come close to proving that instant excavations are at hand.

B.4 PROJECT TRINIDAD RAILWAY CUT

a. General

Project Trinidad consisted of an extensive program of chemical explosive cratering experiments conducted by the Nuclear Cratering Group during 1970 in connection with the Trinidad Dam and Lake Project in Colorado. These experiments were designed to provide additional field test data relating to several different aspects of explosive excavation and to culminate in the demonstration of a method of making a railway cut using a design developed as a result of the testing. The results of the Project Trinidad program will be published by NCG.[1] Only the Railway Cut, Detonation D4, is discussed here. Current plans are to conduct two additional railway cuts at Trinidad in 1971, each to test a different explosive excavation design concept from that used for the D4 cut.

b. Project Description

The location of the D4 railway cut is shown in Fig. B16. The D4 experiment involved the excavation of a 400-ft cut along the realinement of the Colorado and Wyoming Railroad. The terrain in the area of the cut had varying slopes. A portion of the cut was along a gentle sidehill. Design depth of the cut ranged from 15 to 20 ft, and the required width at subgrade elevation was 46 ft. Conventional excavation of the cut would have required the removal of a trapezoidal shaped section 400 ft long containing approximately 13,000 yd^3 of sandstone and shale.

c. Excavation Design

The explosive excavation design was prepared in accordance with the procedures presented in Chapter 5 of this report. The explosive array consisted of two parallel rows of charges. A representative cross section and plan view of the charge array is shown in Fig. B17. The north row consisted of 18 one-ton charges and the south row consisted of 12 two-ton and 2 one-ton charges, a total of 32 separate charges and 44 tons of explosive. The rows were alined parallel to the railway centerline but were offset 23 ft. Each row was then designed separately according to the terrain elevation along its alinement. A constant weight-varying enhancement design was used for both rows. The separation between the rows was fixed at 46 ft along the entire length of the cut. As a consequence of the fixed separation between rows, the row separation expressed in terms of the row crater width varied with the depth of the cut. At the deepest portion of the cut the row separation was 1.45 times the average half-width of a

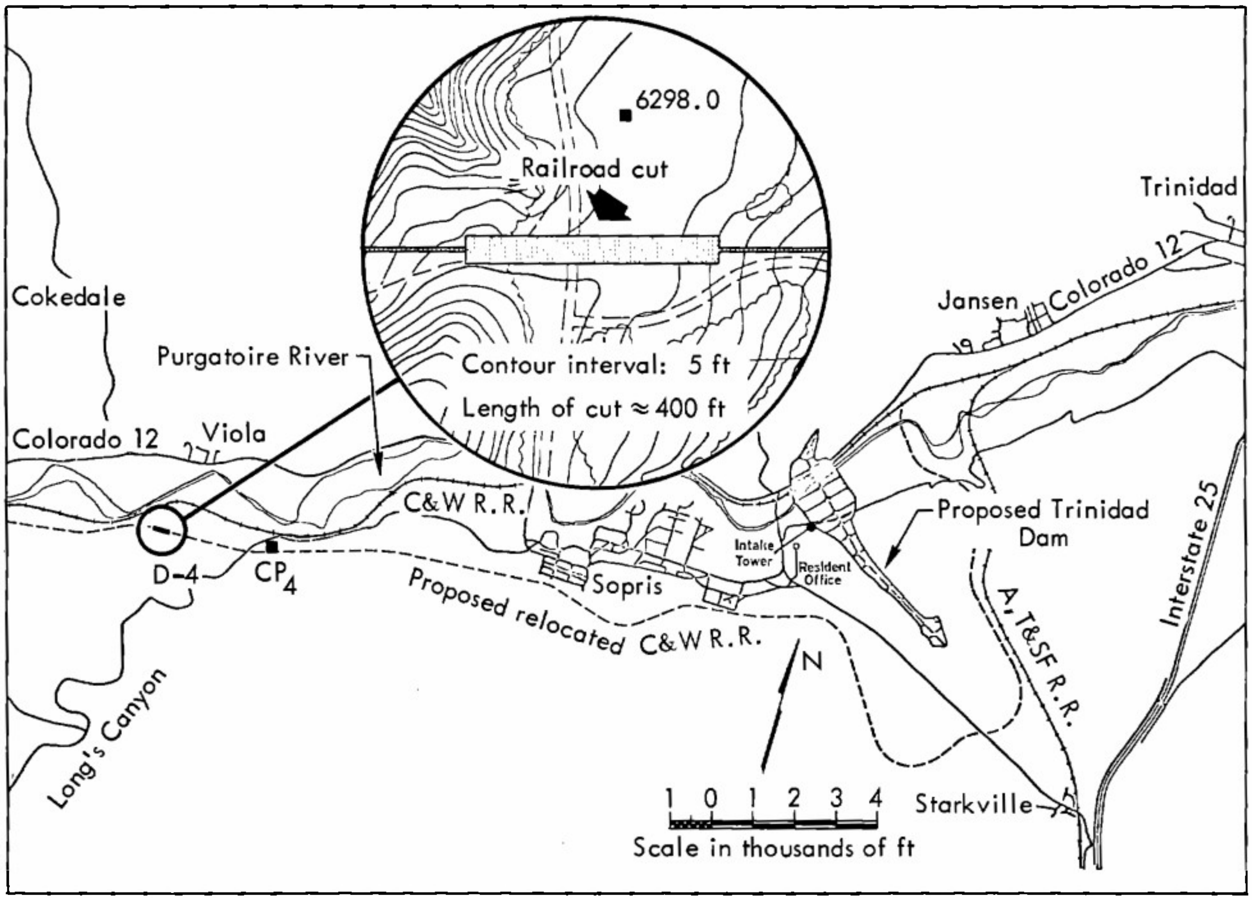


Fig. B16. Location map of Project Trinidad railway cut.

single row crater [$W_a/2$ (average)] and at the shallowest portion the separation was 1.70 $W_a/2$ (average). The charge spacing within each row varied from 0.90 to 1.25 times the average radius of a single charge crater. At the point of deepest cut, (between Stations 91+00 and 92+00 in Fig. B17) a single line of 3-in. diameter "pre-splitting" holes was drilled and lightly loaded with explosive at the predicted location of the apparent crater. The purpose of these holes was to control the up-hill slope in this section.

Previous work by the Nuclear Cratering Group and other investigators has shown that when two rows of charges are used, a delay between the firing of the rows results in a larger excavation than that obtained when both rows are fired simultaneously. Accordingly, a 150-msec delay was introduced between the detonations of the two rows: 9 msec between the pre-splitting charges and the northern (downhill) row and 141 msec between the northern and southern (uphill) rows. This design represented the best experience then available, including the recent experimental work at Trinidad.

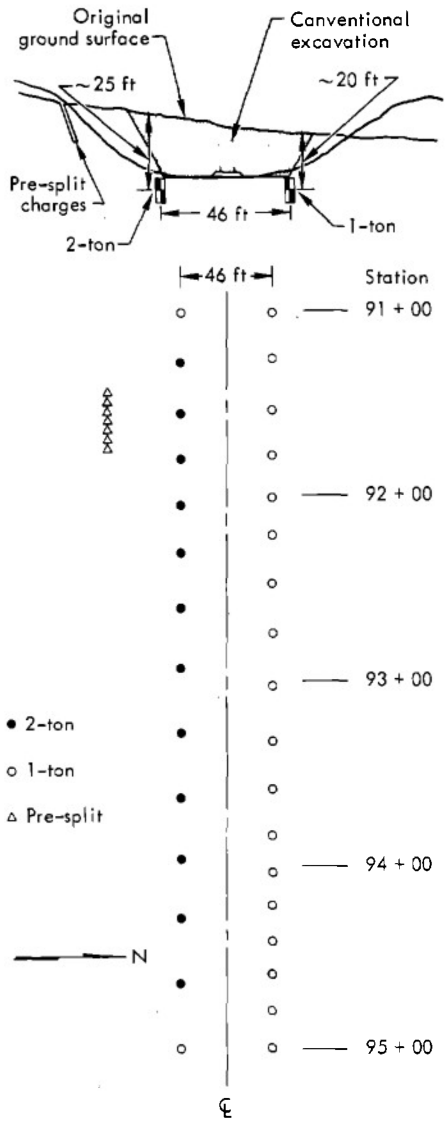


Fig. B17. Representative cross section and plan view of explosive charge array for experiment D4.

d. Emplacement Construction

Two methods were employed for drilling the emplacement holes for the explosive. The northern line of one-ton holes was drilled using an "underreamer." The procedure consisted of drilling an 18-in. pilot hole and expanding the bottom half of the hole to the required 36 in. with a special tool. The southern line of two-ton holes was drilled to depth at the full charge cavity diameter of 30 in. with a bucket auger. Figure B18 shows the bucket auger in operation.

e. Explosive and Explosive Emplacement

An aluminized ammonium nitrate slurry (20% aluminum content) was used because it was readily available at reasonable cost and had proven to be satisfactory in the previous tests at Trinidad. The selection was more a matter of convenience than the result of a detailed economic analysis because all of the explosives for the Trinidad experimental program were procured and emplacement, arming, and firing services were provided under one contract. The charge cavities were loaded by pumping the required amount of slurry directly into the emplacement holes from the mixing truck (Fig. B19). After the firing system was installed, each borehole was stemmed with a pit-run gravel. The sandy gravel was placed directly over the explosive, saturated, and carefully tamped as the holes were back-filled.

f. Detonation and Results

The D4 detonation took place on 16 December 1970. Figure B20 shows the explosion a few seconds after detonation as viewed from the Control Point, located approximately 2000 ft east of SGZ. Note the heavy surge of material near ground level being ejected to the right (north) in the photograph. The maximum range of ejecta thrown out by the explosion (except for dust-size particles) was estimated at 1400 ft.

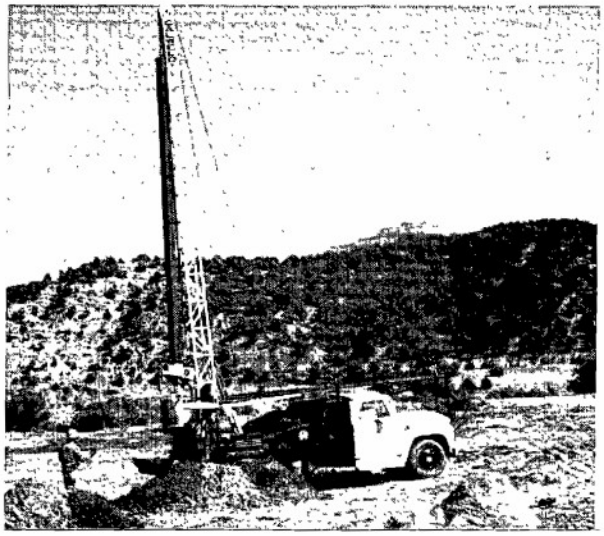


Fig. B18. Bucket auger used to drill 30-in. diameter emplacement holes for 2-ton charges.

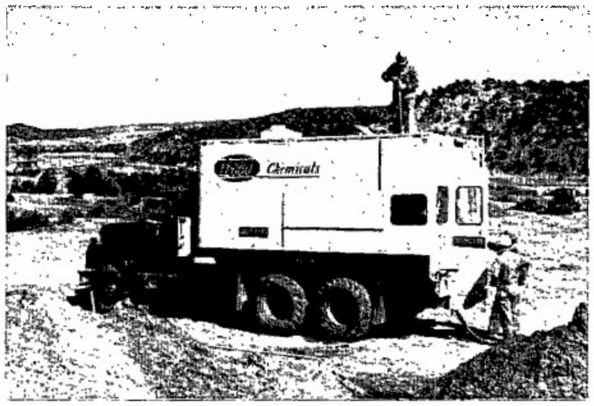


Fig. B19. Truck used to mix and pump slurry blasting agents directly into boreholes at Trinidad

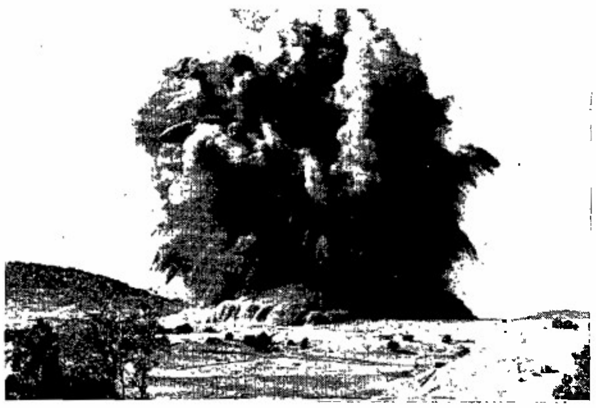


Fig. B20. Photograph of D4 railway cut detonation at Trinidad, Colorado, on 16 December 1970.

The detonation excavated approximately 18,000 yd^3 of material and produced a crater which very closely circumscribed the conventional design cross section (Fig. B21).

g. Summary

The D4 railway cut provided a realistic demonstration of the use of explosive excavation techniques on a construction project and an excellent field test of the design procedures presented in this report. A preliminary analysis of costs indicate that the cost of this excavation is less than that required for conventional blasting and earthmoving methods where the material from the cut is not required for fill. This is encouraging. Modification of the basic cratering technique may be required to allow explosive excavation to compete on a balanced cut and fill project. The next series of tests planned at the Trinidad site will be conducted to investigate promising modifications while still making use of large point charges and design procedures which are extensions of those contained in this report.

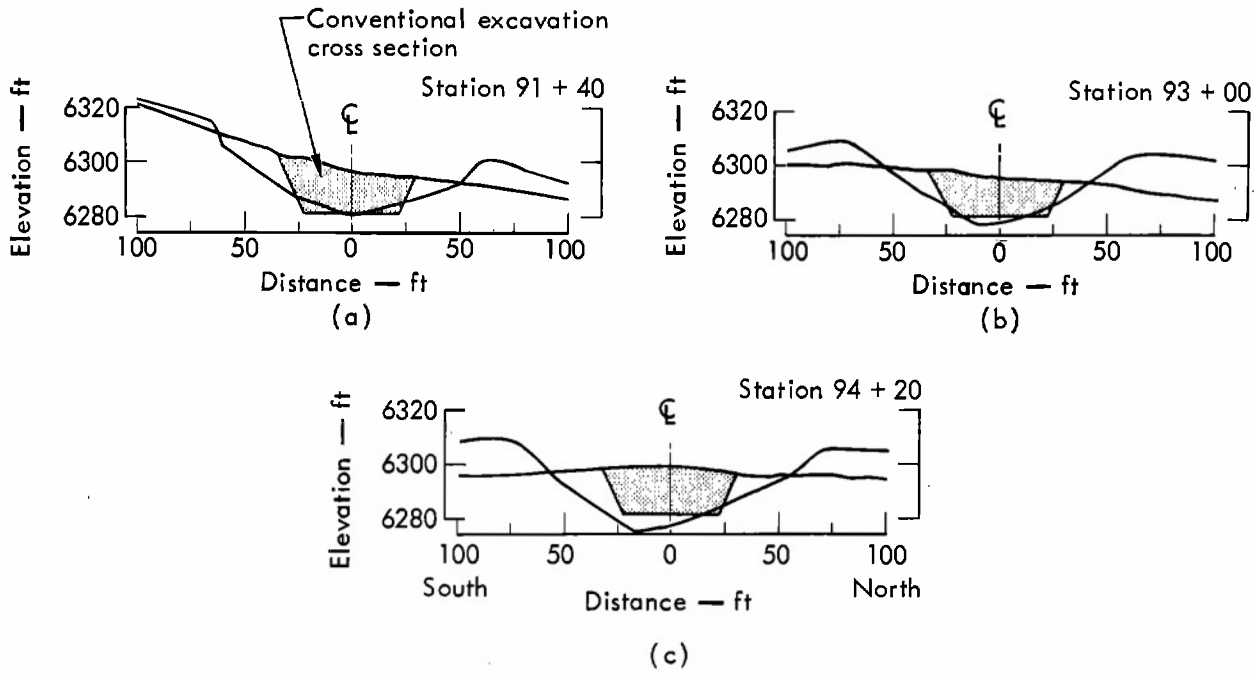


Fig. B21. Three cross sections of D4 crater showing comparison between cratered cut and a cut excavated by conventional techniques.

Reference

1. B. B. Redpath, Project Trinidad: Explosive Excavation Tests in Sandstone and Shale, U. S. Army Engineer Nuclear Cratering Group, Livermore, Calif., Rept. NCG-TR-34 (to be published).

Appendix C
Explosives Supplement

C.1 DETONATION VS DEFLAGRATION

Chemical explosives are oxygen-bearing compounds or mixtures which react violently (i.e., detonate) when subjected to sudden shock and heat and within microseconds are reduced to a gas of high temperature and intense pressure.

Pressures produced by a detonation can exceed a quarter of a million atmospheres and, under such circumstances, the stoutest rock or high-carbon steel behaves like a compressible fluid. A characteristic of explosive detonation is that the pressures generated render the strength of any known material negligible by comparison Temperatures on the other hand generally do not exceed 3000°C and are short-lived; thus changes of state in the surrounding medium rarely occur.

Once initiated, the detonation proceeds through the explosive as a shock wave which travels at a speed greater than the acoustic velocity of the undetonated explosive. Across this shock or detonation wave (which occupies a zone of molecular thickness), there exist large jumps in pressure, temperature, density, and particle velocity (the latter being in the same direction as the detonation wave velocity). The increase in pressure and temperature are sufficient to cause the explosive to vaporize, react within a narrow zone behind the wave, and liberate great quantities of energy.

The energy released by detonation comes from recombination of the atoms to form molecules with exothermic heats of formation. Maximum energy release occurs when there is sufficient oxygen for complete oxidation of the reactants but not so much as to provide an excess which acts as a diluent.

All explosives are combustibles and will burn without supplementary oxygen if subjected to sufficient heat to initiate combustion. Such combustion is termed deflagration. The velocity of a combustion front in deflagration is far below the acoustic velocities of the products and reactants so that pressures are uniform through the charge. If a large quantity of explosive under confinement begins deflagrating, pressures and temperatures can mount considerably until deflagration transforms to detonation with the consequent dramatic increase in pressure and violence. Explosives with low detonation pressures, such as ammonium nitrate, are particularly susceptible to detonation due to deflagration and have caused several tragic disasters (e.g., 1947 at Texas City, Texas, seven million pounds of ammonium nitrate on two ships in the harbor detonated accidentally, killing over 560 persons and injuring some 3000 more).[1] It is a wise practice to store explosives in isolated small quantities and in containers that will rupture at low pressures.

Any explosive has a "critical diameter" below which detonation cannot be maintained. The critical diameter is related to the thickness of the reaction zone which exists directly behind the detonation wave and is usually the same order of magnitude.

Confining the charge underground or in a strong jacket helps reduce the critical diameter since the lateral expansion of the gaseous products is thereby retarded.

At charge diameters slightly above critical diameter, detonation velocity and energy release are below values produced by a larger charge. At larger charge sizes, detonation velocity and energy release cease to be a function of charge dimension and a condition called "ideal detonation" ensues. Whenever detonation properties depend upon charge size, the detonation is termed "non-ideal." For centrally-initiated spherical charges, most explosives detonate ideally whenever the charge diameter exceeds about 8 in. (about 10 lb typically). However, some (e.g., lead azide) detonate ideally even when charge dimensions are only a few millimeters, while others (e.g., pure ammonium nitrate) are influenced by charge dimensions up to 1 ft.

C.2 DETONATION OF BURIED EXPLOSIVES

The energy released from a buried explosion manifests itself in three ways: (1) shock energy resulting from the transmission of the detonation wave into the surrounding medium, (2) potential energy of the high pressure gas bubble, and (3) kinetic energy of the detonation gas products.

Upon encountering the explosive-medium interface, the detonation wave is divided into two components, one transmitted and one reflected, whose characteristics depend upon relative acoustic impedances of the explosive and the medium.[2] Let I_e be the acoustic (or detonation) impedance of the explosive (i.e., the product of explosive bulk specific gravity and detonation velocity) and I_m the acoustic impedance of the medium (i.e., the product of medium bulk specific gravity and seismic velocity). Three cases can occur depending on whether I_m is greater than, equal to, or less than I_e. The three possibilities are shown in Fig. C1.[2] Maximum transmission of shock energy occurs when $I_m = I_e$. Since shock wave propagation is roughly spherically symmetrical even if the charge is not, the energy associated with it is radiated more or less evenly in all directions. One may assume that about half of this energy is largely wasted since it radiates downward. Of the half which is radiated upward, only that portion within the boundaries of the true crater may be considered useful from the standpoint of cratering. Since the latter rarely exceeds a right angle cone, less than one-third of this upward moving energy may be considered useful. Consequently, less than one-sixth of the total shock wave energy generated by the explosion is utilized for cratering.

An argument could be made for mismatching of explosive to rock impedance in order to reduce the amount of shock energy which is transmitted, and therefore largely wasted, by the shock wave. However, it can be shown that this is a futile effort because the ratio of transmitted to incident shock energy is rather insensitive to impedance ratio within the range of I_e/I_m between 0.2 and 5.0.

A counter argument can be put forward for matching impedances. Detonation impedance is directly related to detonation pressure, and acoustic impedance is a crude

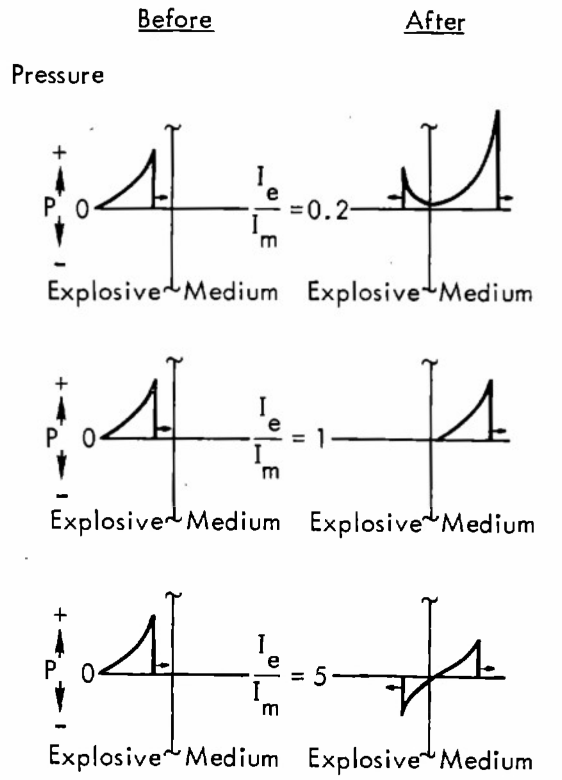


Fig. C1. Interaction of detonation wave with surrounding medium.

measure of the strength of the medium. Matching impedances then can at least roughly tailor shock wave pressure to the strength of the material.

Too low a detonation impedance will produce insufficient shock pressure for rock breakage. Too high a detonation impedance corresponds to a high detonation pressure and a large fraction of incident shock wave energy. In the latter case even if due to the impedance mismatch, transmission properties are poor, a large amount of energy will still be transmitted by the shock wave and wasted in excessive crushing and plastic deformation close to the charge. For this reason, explosive-to-rock impedance ratios greater than unity are to be avoided whenever possible.

The expanding gas bubble provides a more efficient energy source for cratering because it tends to expand toward the spalled free surface heaving the remaining material out of the way in the process. Therefore, this component of energy should be as large as possible. The manner in which the detonation gas products expand is also important. Gas pressures should be sustained for a long period but not so long as to release sizeable amounts of energy to the atmosphere at venting. Explosives which maximize the gas bubble energy are discussed in Chapter 4, Section 4.2c.

The kinetic energy of the detonation products is typically about 10% of the total energy.[3] It is associated with the motion of the product gases but, as a practical matter for understanding cratering detonations, can be considered as indistinguishable from the high pressure gas bubble pulse.

C.3 PRIMARY EXPLOSIVES

Primary explosives differ from other high explosives in their capacity for ideal detonation in milligram quantities. Critical diameters are virtually nonexistent. They are highly unstable substances extremely sensitive to ignition by heat, shock, and electrical discharge and should always be handled with extreme caution and stored in small quantities.

The three major primary explosives (lead azide, mercury fulminate, and lead styphnate) are usually employed in initiating devices such as blasting caps. Their properties are listed in Table C1. Being relatively insensitive to flame, lead azide is a poor primary explosive for fuzed initiators but is excellent for electric blasting caps.

Table C1. Some properties of primary explosives.[2,4]

Property	Lead azide	Mercury fulminate	Lead styphnate
Formula	PbN_6	$C_2N_2O_2Hg$	$PbC_6HN_3O_8$
State	Solid	Solid	Solid
Density (g/cm^3)	4.0	4.0	2.9
Detonation velocity (m/sec)	5180	5000	5200
Heat of detonation (cal/g)	367	427	457

Lead styphnate is used chiefly as an additive to lead azide to improve its flame sensitivity. Mercury fulminate, when mixed with 10 to 20% potassium chlorate, is used in fuzed blasting caps.

C.4 HIGH EXPLOSIVES

High explosives differ from primary explosives in three ways: (1) a definite critical diameter exists below which the explosive will not detonate, (2) electrostatic ignition is very difficult, and (3) much stronger shocks are required for detonation.

Most high explosives come from the carbon-hydrogen-oxygen-nitrogen (CHON) family and may be cast or molded into shape. TNT ($C_7H_5N_3O_6$) is one of the pure high explosives that can be cast into shape. Other solid explosives may be cast by using TNT as an additive. Some solid high explosives are molded into shape by plastic bonding; i.e., the explosive powder is mixed with a plastic and solvent. The solvent is removed by distillation causing the plastic to precipitate out of the explosive, forming a molding powder that can then be pressed into shape.

The normal state and principal uses of common pure high explosives are listed in Table C2. Common high explosive mixtures, their state, formulation, manufacture,

Table C2. Common pure high explosives.[1,4]

Common name (other names)	State	Principal uses
HMX (octagon)	Solid	Plastic bonded explosives
Nitroglycerine (NG)	Liquid	Dynamites
Nitromethane (NM)	Liquid	Cratering research explosive
Pelletol (TNT pellets)	Solid pellets	Mining
PETN	Solid	Detonators
RDX (hexagon, cyclonite)	Solid	Cast and plastic bonded explosive
Tetryl	Solid	Boosters
TNT	Solid	Cast explosives

and principal uses are listed in Table C3. Important properties of explosives are their density, detonation pressure, detonation wave velocity, heat of detonation or

Table C3. Common high explosive mixtures.[4]

Explosive	State	Formulation (wt %)	Manufacture	Principal uses
Composition B Grade A	Solid	36 TNT 63 RDX 1 wax	Cast	Boosters, military applications
Composition C-4	Putty	91 RDX 7.4 solvents 1.6 motor oil	Mix	Boosters, general purpose putty explosive
Cyclotol	Solid	25 TNT 75 RDX	Cast	Boosters, military applications
LX-04	Solid	85 HMX 15 Viton A	Plastic bonded	Boosters, military applications
Pentolite	Solid	50 TNT 50 PETN	Cast	Boosters, military applications

total energy, thermal stability, and impact sensitivity. These properties for many high explosives are listed in Table C4.

Most high explosives have excessive detonation pressures to be useful as cratering explosives. Only TNT and nitromethane have been extensively used as cratering charges. PETN, tetryl, pentolite and compositions B and C-4 are often employed in detonating cord, primary boosters, and as secondary boosters for cratering charges.

C.5 CLASSIFICATION OF EXPLOSIVES

a. Regulations Governing Explosives

It is beyond the scope of this appendix to list the detailed regulations and restrictions governing explosives. Several agencies such as the Association of American Railroads, the Institute of Makers of Explosives, and the American Trucking Association have developed their own rules (e.g., see Ref. 6), but all derive from or make use of the Code of Federal Regulations (CFR).[7] Maritime shipping is covered by CFR Title 46, Part 146, while overland transportation is in CFR Title 49, Parts 172-173. Bureau of Mines explosives classification tests are described in CFR Title 30, Part 15. Pertinent regulations are listed under "Applicable Publications" in Appendix F.

b. Classes of Explosives

Explosives normally used in cratering operations fall into four categories which, in order of hazard, are: Class A, B, or C explosive, and oxidizing materials. The more hazardous the material, the more restrictions surround it and, consequently, the

Table C4. Properties of high explosives.[1,4,5]

Explosive name	Density (g/cm^3)	Detonation pressure (kbar)	Detonation velocity (m/sec)	Heat of detonation[a] (cal/g)	Impact sensitivity[b] (cm)	Thermal stability[c] (cm^3/g)
HMX	1.89	387	9110	1330	30-40	0.07
NG	1.60	253	7700	1400	25	—
NM	1.13	125	6320	1126	—	—
Pelletol	1.00	54	4650	800	80	0.005
PETN	1.70	305	8300	1360	11	0.9-1.2
RDX	1.77	338	8640	1320	28	0.12-0.90
Tetryl	1.71	250	7850	1010	28	0.40
TNT	1.64	220	6930	1030	80	0.005
Composition B	1.72	295	7990	1180	45	0.05-0.16
Composition C-4	1.59	257	8040	1350	—	0.20
Cyclotol	1.76	316	8300	1250	33-71	0.25-0.94
LX-04	1.86	360	8460	1320	41-64	~0.3-0.5
Pentolite	1.67	248	7470	1280	35	3.0 at 100°C

[a]Calculated for gaseous products at density given.

[b]LRL drop weight machine (2.5 to 5.0 kg weight). Figures indicate height of weight for 50% probability of explosion. Less than 25 cm: very sensitive; 25 to 70 cm: moderately sensitive; >70 cm: relative insensitive.

[c]Cubic centimeters of gas evolved per gram after 48 hr at 120°C (less than 1 cm^3/g indicates high thermal stability).

more expensive it is to transport. Whenever possible, Class A explosives are to be avoided and oxidizing materials preferred.

Class A explosives are those which fall into one or more of the following categories.

(1) Can be caused to deflagrate (burn rapidly) by contact with sparks or flame.

(2) Can be detonated by means of a No. 8 blasting cap when unconfined.

(3) Can be detonated by contact with sparks or flame when unconfined.

(4) Can be detonated using the Bureau of Explosives Impact Apparatus* 50% of the time under a drop of 4 in., or 100% of the time under a drop of 10 in.

Class B explosives are propellant explosives which usually function by combustion instead of detonation. Typical examples of Class B explosives are some types of smokeless powders, liquid monopropellants, and solid rocket propellants.

Class C explosives are certain types of manufactured articles which contain Class A or B explosives or both as components, but in restricted quantities. For example, blasting caps in quantities of less than 1000 are Class C explosives.

*Eight-pound drop hammer tests.

Class A and B explosives must be labeled as such. No labels are required for Class C explosives.

Oxidizing materials are substances which decompose readily to yield oxygen when heated and may react violently with other combustible materials. Substances such as slurry blasting agents which contain ammonium nitrate but no Class A explosives are called nitrocarbonitrates and are classified as oxidizing materials.

Nitromethane is unique in that although it can be made to detonate, it may be shipped as an industrial solvent, because it is quite insensitive to sparks, shock, or flame.

The classifications of explosives discussed in this appendix are summarized in Table C5.

C.6 MEASUREMENT OF EXPLOSIVES PROPERTIES

The following variables should be known or determined as a part of the process of selecting a cratering explosive:

(1) Bulk density

(2) Detonation velocity

(3) Total available energy and gas bubble energy.

Usually the manufacturer provides this information concerning his explosives. The following paragraphs briefly describe how to obtain this data for certification purposes if it is otherwise unavailable.

Density can be measured routinely and techniques for doing so are not discussed here.

Detonation velocity can be measured in two simple ways: Dautriche test and time of arrival. The latter requires instrumentation such as a time interval counter or oscilloscope; the former does not. Both require a heavily confined cylindrical charge about 2 ft long and at least 4 in. in diameter. The charge may be detonated in air, underground, or underwater. Both tests are shown schematically in Fig. C2.

In the Dautriche test, a long length of detonating cord is inserted into the charge at two points a known distance, L_1, apart. The midpoint of the detonating cord is noted and the cord is taped to a length of lead or aluminum witness plate as shown in the figure. The detonation wave in the test charge will initiate each end of the detonating cord at different times so that the detonation waves in the cord will meet and leave a scar on the witness plate at a point removed from the midpoint. The detonation velocity, V_d, of the charge can then be found by the formula[2]:

$$V_d = V_p \, L_1/(2L_2) \qquad \text{(C1)}$$

where V_p is the detonation velocity of the detonating cord. For PETN detonating cord, $V_p = 6400$ m/sec.[8]

Table C5. Classification of explosives.[7]

Substance	Classification
Lead azide	Shipment in bulk quantities prohibited
Mercury fulminate	Shipment in bulk quantities prohibited
Lead styphnate	Shipment in bulk quantities prohibited
HMX	Explosive A
NG	Shipment in bulk quantities prohibited
NM	No explosive classification
PETN	Explosive A
RDX	Explosive A
Tetryl	Explosive A
TNT (and Pelletol)	Explosive A
Composition B, grade A	Explosive A
Composition C-4	Explosive A
Cyclotol	Explosive A
LX-04	Explosive A
Pentolite	Explosive A
AN	Oxidizing material
ANFO	Oxidizing material
AMATOL	Explosive A
Ammonol	Explosive A
Tritonal	Explosive A
AN slurries	Oxidizing material (NCN)[a]
AN slurries	Explosive C[b]
AN slurries	Explosive A[c]
AN-Al slurries	Oxidizing material (NCN)[a]
Blasting caps	Explosive A[d]
Blasting caps	Explosive C[e]

[a]No Explosive A component.

[b]Limited Explosive A component.

[c]Unlimited Explosive A component.

[d]More than 1000.

[e]1000 or less.

The time of arrival test is shown in the bottom half of Fig. C2. The detonation wave successively closes the contacts and the resulting pulses can be used to start and

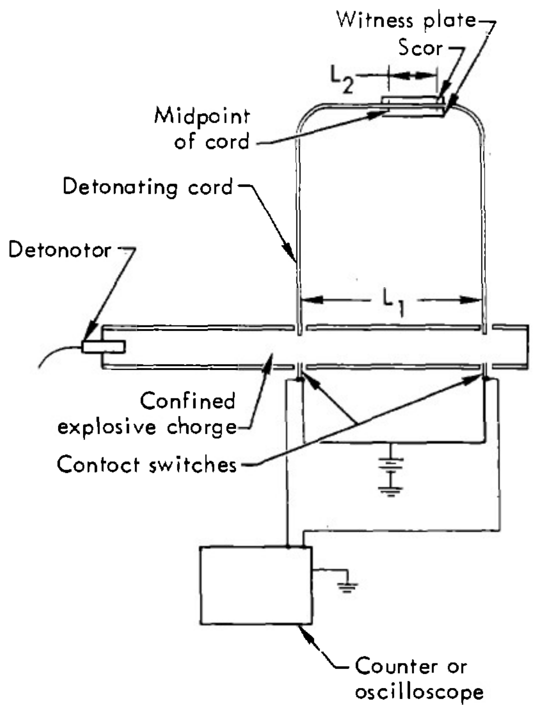


Fig. C2. Test setup for measuring detonation velocity (top: Dautriche test; bottom: time-of-arrival test).

stop a counter or trace two jumps on an oscilloscope. Detonation velocity is calculated by dividing the distance between contacts by the time interval between pulses.

Once density and detonation velocity are known, detonation pressure, P, in kilobars can be calculated by the semi-empirical formula[9]:

$$P = (485\rho - 175\rho^2)\left(\frac{V_d}{10{,}000}\right)^2, \qquad \text{(C2)}$$

where ρ is the explosive bulk specific gravity or density in g/cm^3 and V_d is the detonation velocity in m/sec.

Total heats of detonation can be measured by metal acceleration methods, calorimetry, calibrated underwater tests, and uncalibrated underwater tests.[1,5,10,11] Underwater testing is the only practical means of experimentally evaluating explosion energy when the explosive to be considered is new or unfamiliar or detonates nonideally at greater than 10-lb quantities. An underwater detonation radiates a shock pulse in all directions which is followed by a series of lower pressure pulses resulting from the oscillations of the gas bubble as it migrates to the surface.

Uncalibrated underwater testing is the easiest way to measure gas bubble energy. While times of arrival of the waves resulting from the detonation are measured with accuracy, pressures are not. It is the time of arrival of the first bubble pulse that is measured in the uncalibrated underwater test. Instrumentation is therefore limited to an inexpensive uncalibrated but rugged piezoelectric transducer and a single-channel oscilloscope with a sweep speed of at least 50 msec/cm. The test site can be any body of water, the dimensions of which depend upon the weight of explosive. For a 20-lb charge, 80-ft depth and 100-ft breadth are adequate. Accuracy, of course, depends upon instrumentation but assuming that the time measurement is accurate to 1%, gas bubble energy will be accurate within 3%.

A typical test layout is shown in the inset of Fig. C3. The pressure-time history at the transducer station is also shown in Fig. C3. All that is of interest in this trace for the uncalibrated test is the bubble period, T. This is the time it takes for the gas bubble to expand and contract once. Once the bubble period is known, the bubble energy can be readily computed by the formula[12]:

$$E_b = \left[5.5\ T^3(h + 2.31\ P_0)^{2.5} - \frac{QB}{2}\right] \Big/ Y, \qquad \text{(C3)}$$

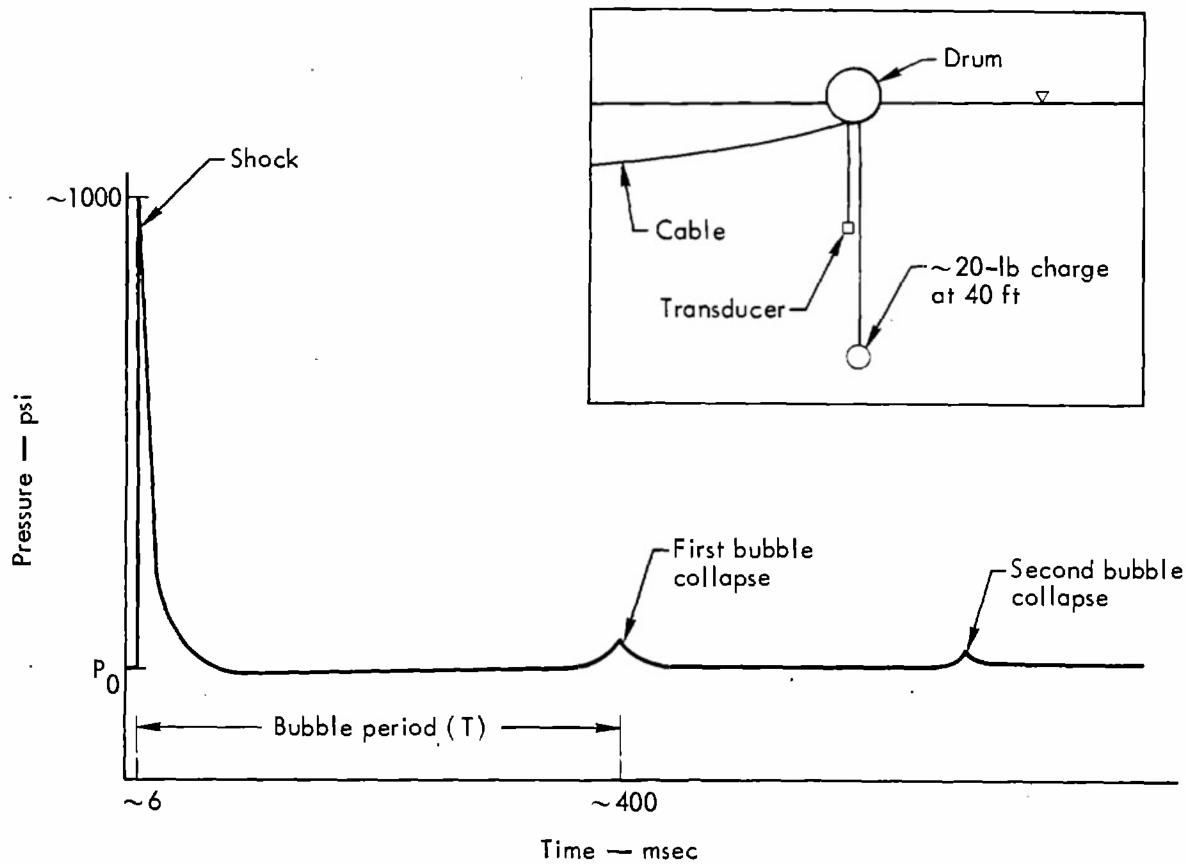


Fig. C3. Typical pressure trace from underwater test of type shown in insert diagram.

where E_b is the bubble energy in calories per gram, T is the bubble period in seconds, h is the depth of charge in feet of fresh water (if in sea water, multiply h by 1.03), P_0 is the local atmospheric pressure in psia, Y is the explosive weight — excluding booster — in pounds, B is the weight of booster in pounds, and Q is the heat of detonation of the booster explosive in cal/g (nominally 1000 cal/g).

Total heat of detonation is within 10% of two times E_b for nonmetallized explosives.[10,11] For aluminized explosives, the ratio of total energy to bubble energy varies from about 1.5 to 2.0 depending upon the ratio of aluminum to available oxygen. The proper multiplier is given in Fig. C4. The curve in the figure was developed experimentally and is accurate to within 10%. To use the figure, the number of gram-atoms of aluminum and available oxygen must be found. The following general formula accomplishes all this for various explosive mixtures. Weight percentages are used and, if a particular component does not occur in a mixture, simply substitute 0% in the formula. The formula is[13]:

$$Al/O = 0.0371\ (\%\ Al)/B \qquad (C4)$$

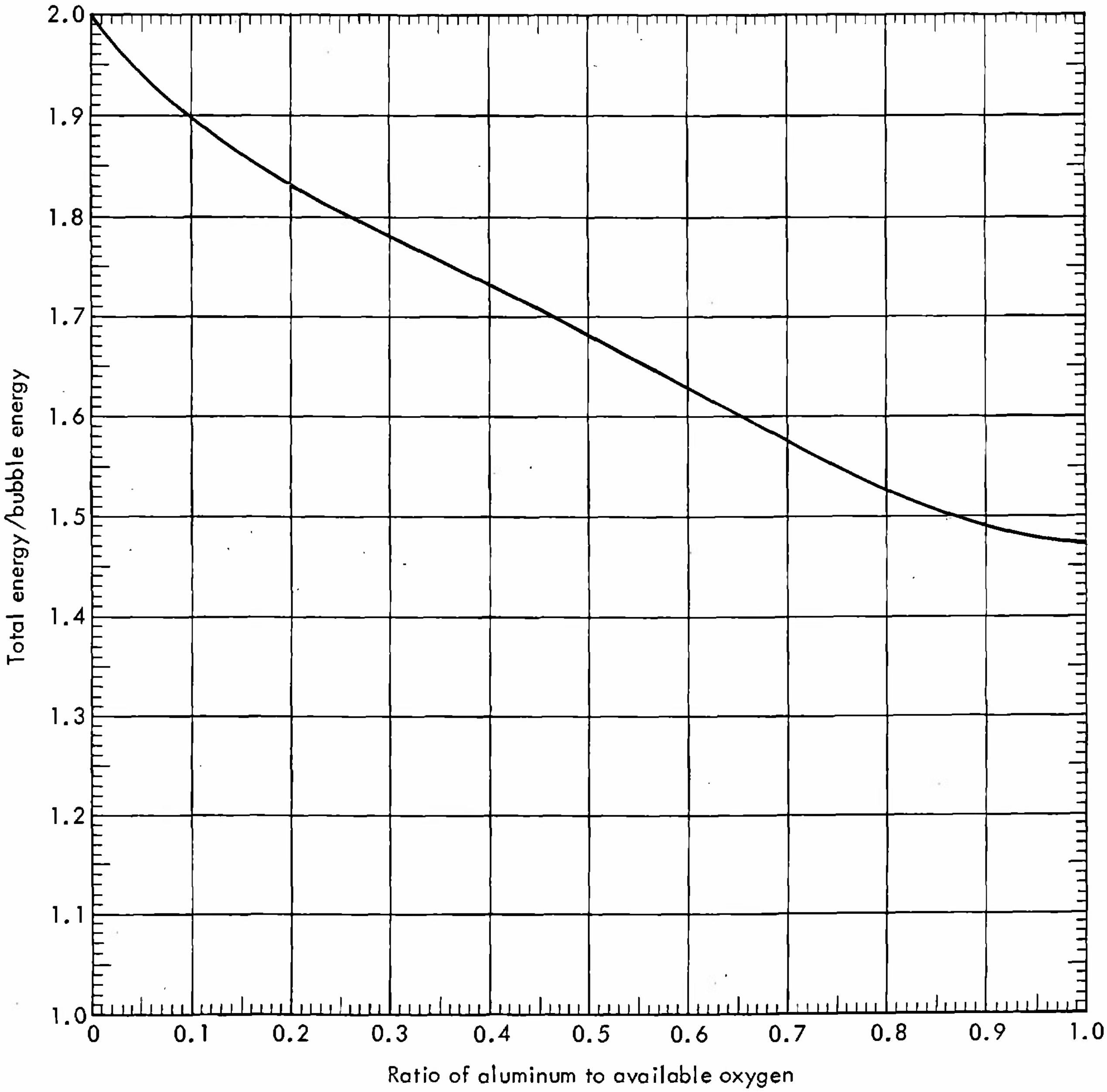


Fig. C4. Ratio of total energy to bubble energy (from Fig. 16).

where

$$
\begin{aligned}
B = {} & 0.0270\ (\%\ \mathrm{RDX}) + 0.0264\ (\%\ \mathrm{TNT}) \\
& + 0.0375\ (\%\ \mathrm{AN}) + 0.0270\ (\%\ \mathrm{HMX}) \\
& + 0.0328\ (\%\ \mathrm{NM}) + 0.0399\ (\%\ \mathrm{NG}) \\
& + 0.0279\ (\%\ \text{tetryl}) \\
& + 0.0380\ (\%\ \mathrm{PETN}) \\
& + 0.0555\ (\%\ \text{water}) \\
& + 0.0353\ (\%\ \text{sodium nitrate})
\end{aligned}
$$

(continued on next page)

$+ 0.0297$ (% potassium nitrate)
$+ 0.0341$ (% $NH_4Cl\ O_4$)
$+ 0.0326$ (% $N_aCl\ O_4$)

As an example, suppose a bubble period, T, of 0.40 sec was measured for a 20-lb charge with a 1-lb pentolite booster* suspended in 40 ft of fresh water: the composition of the charge was:

AN = 50%
Al = 30%
H_2O = 20%

The bubble energy is given by (1 atm = 33.9 ft of fresh water):

$$
\begin{aligned}
E_b &= [5.5\ (0.40)^3(40 + 33.9)^{2.5} - 1000(1)/2]/21 \\
&= [5.5\ (0.064)(46950) - 500]/21 \\
&= 763\ \mathrm{cal/g}.
\end{aligned}
$$

The aluminum-to-oxygen ratio is given by:

$$
\begin{aligned}
Al/O &= 0.0371(30)/[(0.0375)(50) + 0.0555(20)] \\
&= 0.373.
\end{aligned}
$$

From Fig. C4, the ratio of total energy to bubble energy is 1.74. The total heat of detonation is then (1.74)(763) = 1328 cal/g.

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*Assume Q = 1000 cal/g.

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Appendix D
Site Calibration

D.1 SCOPE

This appendix is a supplement to Chapter 5. It describes the use of field calibration tests, and methods of data reduction, to include details for the construction of the design charts which appear in Chapter 5.

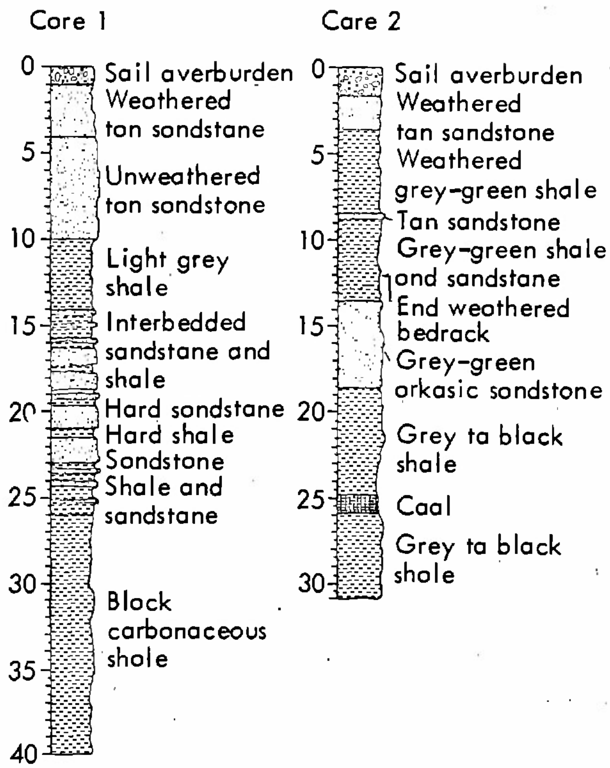
D.2 SITE CALIBRATION PROCEDURES

Detailed design procedures in current use are presented in Chapter 5. Crater dimension data which have been determined through experimentation for certain media are presented in Figs. 17 through 19. Occasionally the material to be cratered will be considerably different from those materials for which crater dimensions have been given in Chapter 5. In these instances it will be necessary to conduct single-charge calibration tests in order to obtain the necessary crater dimension data to complete the project design.

A calibration experiment designed to provide basic cratering data will generally consist of no less than three single-charge detonations. The optimum depth of burial can be initially estimated on the basis of the energy content of the explosive. Optimum depths are close to 1.9 ft/Mcal$^{1/3}$, and this depth should be bracketed by the calibration shots. For reasons of economy the charge weights should be as small as possible but they must be large enough to produce craters which can be measured reliably. In some instances, the project and site conditions may make it advisable to use charge weights on the order of those expected to be used in the project itself. The testing area should closely approximate the average site conditions to be encountered throughout the project, such as the amount of overburden or water cover and the site geology.

As an illustration of these procedures consider the following hypothetical problem. A project under study requires that several diversion channels be excavated through varying terrain. A preliminary geologic reconnaissance of the site reveals that the rock in the area is primarily dry, interbedded sandstone and shale with varying thicknesses of overburden as shown in Fig. D1. Because this material is layered, dry and weak, it varies considerably from that for which cratering data are available and site calibration data are needed.

From available topographic data and project requirements it is determined that the maximum depth of cut for any channel will be 20 ft, and that the average depth of cut will be 13 ft. In order to obtain a conservative approximation of the charge weights required for the project, an analysis of the deepest cut should be made using the design procedures of Chapter 5 and the crater dimension data for dry rock (Fig. 17). Such an analysis will show that the maximum single-charge weight required is 10 tons of TNT,



Notes: 1. Water table not evident.
2. Depths in feet.

Fig. D1. Area stratigraphy.

and that the single-charge weight required to excavate the average depth of cut is 2 tons of TNT. For this range of charge weights a series of 1-ton calibration shots would appear to be adequate. (If the original charge weight estimates had been from 20 to 100 tons, then a series of 5- to 10-ton calibration shots would be more advisable.)

The explosive to be used in the calibration series should be selected from a listing of those most likely to be used for the project (see Chapters 4 and 9. For the hypothetical project under consideration an aluminized (18 to 20%) ammonium nitrate slurry blasting agent is selected. The heat of detonation, as provided by the explosives contractor, is 1500 cal/g. For 1-ton calibration shots the optimum depth of burial is estimated as follows:

$$\text{Total explosive energy} = (\text{heat of detonation}) \times (\text{charge weight}) \tag{D1}$$

$$= (1500\ \text{cal/g}) \times (454\ \text{g/lb}) \times (2000\ \text{lb})$$

$$= 1.36 \times 10^9\ \text{cal.}$$

$$\text{Optimum DOB} \simeq (1.9\ \text{ft/Mcal}^{1/3}) \times (\text{total explosive energy})^{1/3} \tag{D2}$$

$$\simeq (1.9\ \text{ft/Mcal}^{1/3}) \times (1.36 \times 10^3\ \text{Mcal})^{1/3}$$

$$\simeq 21.1\ \text{ft.}$$

In order to define fully the cratering characteristics of the medium, a series of five shots is planned with depths of burial at 16, 18, 21, 23, and 28 ft. In addition to obtaining the optimum parameters for follow-on design of the project, such a series will provide practical experience in the tolerances to which drilling and charge emplacement criteria must be held.

Ground surveys should be made at each shot location both before and after detonation. These surveys will generally consist of two orthogonal profiles through the shot

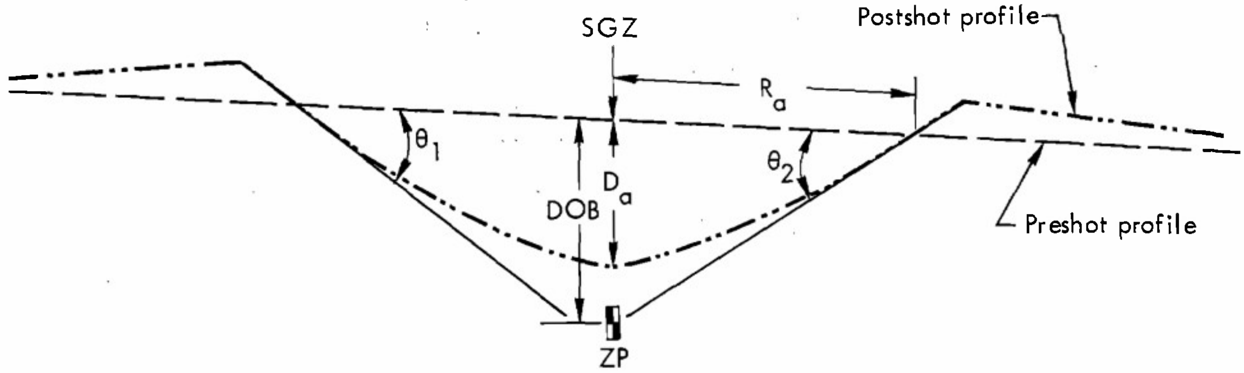


Fig. D2. Typical single-charge crater profile.

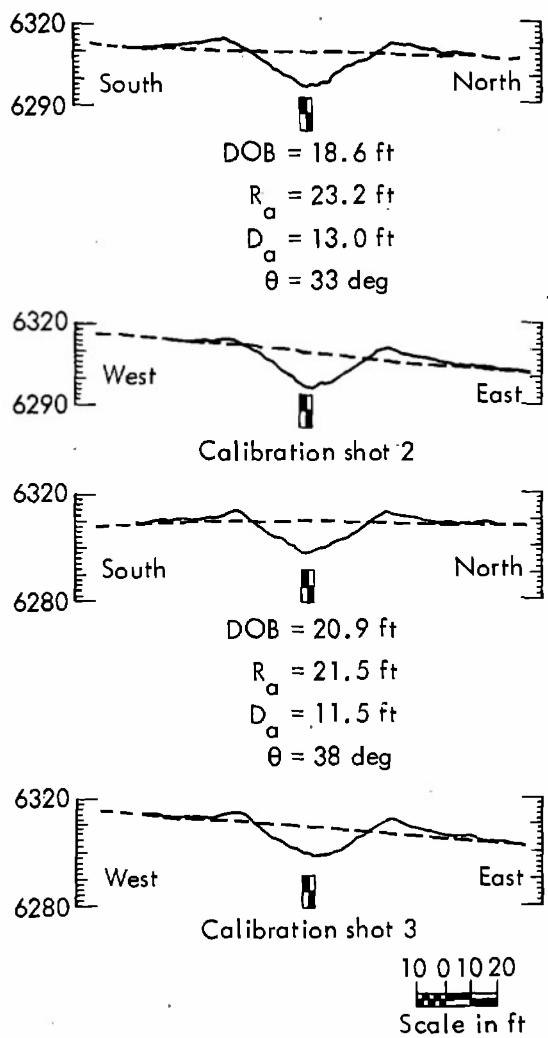


Fig. D3. Crater profiles from calibration shots 2 and 3.

point, the profiles extending out from SGZ to three to four times the expected crater radius. A typical single-charge crater profile is shown in Fig. D2. Both preshot and postshot profiles are plotted to scale, and applicable distances or angles are read directly from them. In cases in which two or more measurements are taken, average values are computed. Four parameters are of prime interest: actual depth of burial (DOB), apparent crater radius (R_a), apparent crater depth (D_a), and the fallback slope angle (θ) measured from the profiles at the original ground surface—see Section 8.2a (3). The results of two of the calibration shots are presented in Fig. D3. The values shown in Fig. D3 were obtained by averaging those values determined from the two profiles at each shot location.

It has been found from experimentation that crater geometry is best approximated by a hyperbolic cross section (see Fig. 20). Table D1 provides data which can be used to compute single-charge crater volume and, for row-charge crater volume computations, the cross-sectional area, all based upon the assumption of hyperbolic shape. Additionally, the table provides the parameters "b" and "p" for use in Eq. 17. All units in the table are normalized to the depth of the crater, thus, for a specific crater:

Table D1. Crater dimensions—hyperbolic crater geometry.

θ (deg)	R_a/D_a =	1.4	1.5	1.6	1.7	1.8	1.9	2.0	2.1	2.2	2.3	2.4	2.5
25	v =	0.	0.	0.	0.	0.	0.	0.	0.	5.19	5.90	6.64	7.41
	a =	0.	0.	0.	0.	0.	0.	0.	0.	2.25	2.43	2.60	2.77
	b =	0.	0.	0.	0.	0.	0.	0.	0.	0.03	0.08	0.13	0.18
	p =	0.	0.	0.	0.	0.	0.	0.	0.	0.98	0.93	0.90	0.87
26	v =	0.	0.	0.	0.	0.	0.	0.	4.73	5.40	6.11	6.85	7.62
	a =	0.	0.	0.	0.	0.	0.	0.	2.15	2.33	2.50	2.67	2.83
	b =	0.	0.	0.	0.	0.	0.	0.	0.02	0.08	0.13	0.18	0.24
	p =	0.	0.	0.	0.	0.	0.	0.	0.98	0.93	0.90	0.86	0.84
27	v =	0.	0.	0.	0.	0.	0.	4.27	4.91	5.59	6.29	7.03	7.80
	a =	0.	0.	0.	0.	0.	0.	2.03	2.22	2.39	2.56	2.72	2.88
	b =	0.	0.	0.	0.	0.	0.	0.02	0.07	0.13	0.19	0.25	0.31
	p =	0.	0.	0.	0.	0.	0.	0.98	0.94	0.90	0.86	0.83	0.81
28	v =	0.	0.	0.	0.	0.	3.82	4.43	5.07	5.75	6.46	7.20	7.97
	a =	0.	0.	0.	0.	0.	1.92	2.10	2.28	2.44	2.60	2.76	2.92
	b =	0.	0.	0.	0.	0.	0.01	0.07	0.12	0.18	0.25	0.31	0.38
	p =	0.	0.	0.	0.	0.	0.99	0.94	0.90	0.87	0.83	0.81	0.78
29	v =	0.	0.	0.	0.	0.	3.97	4.58	5.22	5.90	6.61	7.34	8.11
	a =	0.	0.	0.	0.	0.	1.98	2.16	2.33	2.49	2.65	2.80	2.95
	b =	0.	0.	0.	0.	0.	0.05	0.11	0.18	0.24	0.31	0.38	0.46
	p =	0.	0.	0.	0.	0.	0.95	0.91	0.87	0.84	0.81	0.78	0.76
30	v =	0.	0.	0.	0.	3.52	4.10	4.71	5.36	6.03	6.74	7.48	8.25
	a =	0.	0.	0.	0.	1.86	2.04	2.21	2.37	2.53	2.68	2.84	2.99
	b =	0.	0.	0.	0.	0.04	0.10	0.17	0.23	0.31	0.38	0.46	0.54
	p =	0.	0.	0.	0.	0.96	0.92	0.88	0.84	0.81	0.78	0.76	0.74
31	v =	0.	0.	0.	3.09	3.64	4.22	4.93	5.48	6.15	6.86	7.60	8.37
	a =	0.	0.	0.	1.73	1.91	2.08	2.25	2.41	2.56	2.72	2.87	3.02
	b =	0.	0.	0.	0.02	0.08	0.15	0.22	0.30	0.37	0.45	0.54	0.63
	p =	0.	0.	0.	0.98	0.93	0.88	0.85	0.81	0.79	0.76	0.74	0.72
32	v =	0.	0.	0.	3.20	3.75	4.33	4.94	5.59	6.26	6.97	7.71	8.48
	a =	0.	0.	0.	1.79	1.96	2.12	2.28	2.44	2.59	2.74	2.89	3.04
	b =	0.	0.	0.	0.06	0.13	0.20	0.28	0.36	0.44	0.53	0.62	0.72
	p =	0.	0.	0.	0.94	0.90	0.85	0.82	0.79	0.76	0.74	0.72	0.70
33	v =	0.	0.	2.78	3.30	3.85	4.43	5.04	5.69	6.36	7.07	7.81	8.58
	a =	0.	0.	1.85	1.83	2.00	2.16	2.32	2.47	2.62	2.77	2.92	3.06
	b =	0.	0.	0.04	0.11	0.18	0.26	0.34	0.43	0.52	0.62	0.71	0.82
	p =	0.	0.	0.96	0.91	0.87	0.83	0.80	0.77	0.75	0.72	0.71	0.69
34	v =	0.	2.38	2.87	3.39	3.94	4.52	5.13	5.78	6.45	7.16	7.90	8.67
	a =	0.	1.52	1.70	1.87	2.03	2.19	2.34	2.50	2.65	2.79	2.94	3.09
	b =	0.	0.01	0.08	0.16	0.24	0.32	0.41	0.50	0.60	0.70	0.81	0.92
	p =	0.	0.99	0.93	0.88	0.84	0.80	0.77	0.75	0.73	0.71	0.69	0.68
35	v =	0.	2.47	2.95	3.47	4.02	4.60	5.22	5.86	6.53	7.24	7.98	8.75
	a =	0.	1.56	1.74	1.90	2.06	2.22	2.37	2.52	2.67	2.82	2.96	3.10
	b =	0.	0.05	0.13	0.21	2.29	0.36	0.48	0.58	0.69	0.80	0.91	1.03
	p =	0.	0.95	0.90	0.85	0.81	0.78	0.75	0.73	0.71	0.69	0.68	0.66
36	v =	2.09	2.54	3.03	3.55	4.10	4.68	5.29	5.94	6.61	7.32	8.06	8.83
	a =	1.42	1.60	1.77	1.93	2.09	2.24	2.39	2.54	2.69	2.83	2.98	3.12
	b =	0.02	0.09	0.18	0.26	0.35	0.45	0.56	0.66	0.78	0.90	1.02	1.15
	p =	0.98	0.92	0.87	0.83	0.79	0.76	0.74	0.71	0.70	0.68	0.66	0.65
37	v =	2.16	2.61	3.10	3.62	4.17	4.75	5.36	6.00	6.68	7.39	8.13	8.90
	a =	1.46	1.64	1.80	1.96	2.11	2.27	2.41	2.56	2.71	2.85	2.99	3.14
	b =	0.06	0.14	0.23	0.32	0.42	0.52	0.64	0.75	0.87	1.00	1.14	1.27
	p =	0.95	0.89	0.84	0.80	0.77	0.74	0.72	0.70	0.68	0.67	0.65	0.64

Table D1 (continued)

θ (deg)	R_a/D_a =	1.4	1.5	1.6	1.7	1.8	1.9	2.0	2.1	2.2	2.3	2.4	2.5
38	v =	2.22	2.68	3.16	3.68	4.23	4.81	5.43	6.07	6.74	7.45	8.19	8.96
	a =	1.50	1.67	1.83	1.98	2.14	2.29	2.43	2.58	2.72	2.87	3.01	3.15
	b =	0.10	0.19	0.28	0.39	0.49	0.60	0.72	0.85	0.98	1.11	1.26	1.41
	p =	0.92	0.86	0.82	0.78	0.75	0.73	0.70	0.69	0.67	0.65	0.64	0.63
39	v =	2.28	2.74	3.22	3.74	4.29	4.87	5.48	6.13	6.80	7.51	8.25	9.02
	a =	1.53	1.69	1.85	2.01	2.16	2.30	2.45	2.60	2.74	2.88	3.02	3.16
	b =	0.14	0.24	0.34	0.45	0.56	0.68	0.81	0.95	1.09	1.23	1.39	1.55
	p =	0.89	0.84	0.80	0.76	0.74	0.71	0.69	0.67	0.66	0.64	0.63	0.62
40	v =	2.33	2.79	3.28	3.80	4.35	4.93	5.54	6.18	6.86	7.57	8.30	9.07
	a =	1.56	1.72	1.87	2.02	2.17	2.32	2.47	2.61	2.75	2.89	3.04	3.18
	b =	1.19	0.29	0.40	0.52	0.64	0.77	0.91	1.05	1.20	1.36	1.53	1.70
	p =	0.86	0.82	0.78	0.75	0.72	0.70	0.68	0.66	0.65	0.63	0.62	0.61
41	v =	2.39	2.84	3.33	3.85	4.40	4.98	5.59	6.23	6.91	7.62	8.35	9.12
	a =	1.58	1.74	1.89	2.04	2.19	2.34	2.48	2.62	2.77	2.91	3.05	3.19
	b =	0.24	0.35	0.47	0.59	0.72	0.86	1.01	1.17	1.33	1.50	1.68	1.86
	p =	0.84	0.79	0.76	0.73	0.70	0.68	0.67	0.65	0.64	0.63	0.61	0.61
42	v =	2.43	2.89	3.38	3.89	4.44	5.02	5.64	6.28	6.96	7.66	8.40	9.17
	a =	1.60	1.76	1.91	2.06	2.21	2.35	2.49	2.64	2.78	2.92	3.06	3.20
	b =	0.29	0.41	0.54	0.67	0.81	0.96	1.12	1.29	1.46	1.64	1.83	2.03
	p =	0.81	0.77	0.74	0.71	0.69	0.67	0.65	0.64	0.63	0.62	0.61	0.60
43	v =	2.48	2.93	3.42	3.94	4.49	5.07	5.68	6.32	7.00	7.71	8.45	9.22
	a =	1.62	1.78	1.93	2.07	2.22	2.36	2.51	2.65	2.79	2.93	3.07	3.21
	b =	0.35	0.48	0.61	0.76	0.91	1.07	1.24	1.42	1.60	1.80	2.00	2.22
	p =	0.79	0.76	0.72	0.70	0.68	0.66	0.64	0.63	0.62	0.61	0.60	0.59
44	v =	2.52	2.97	3.46	3.98	4.53	5.11	5.72	6.37	7.04	7.75	8.49	9.26
	a =	1.64	1.79	1.94	2.09	2.23	2.38	2.52	2.66	2.80	2.94	3.08	3.22
	b =	0.41	0.55	0.69	0.85	1.01	1.18	1.36	1.56	1.76	1.90	2.19	2.41
	p =	0.77	0.74	0.71	0.69	0.67	0.65	0.63	0.62	0.61	0.60	0.59	0.59
45	v =	2.56	3.01	3.50	4.02	4.57	5.15	5.76	6.40	7.08	7.79	8.52	9.29
	a =	1.66	1.81	1.96	2.10	2.24	2.39	2.53	2.67	2.81	2.95	3.09	3.22
	b =	0.48	0.62	0.78	0.94	1.12	1.30	1.50	1.70	1.92	2.14	2.38	2.62
	p =	0.76	0.72	0.70	0.67	0.65	0.64	0.63	0.61	0.60	0.59	0.59	0.58
46	v =	2.59	3.05	3.53	4.05	4.60	5.18	5.79	6.44	7.11	7.82	8.56	9.33
	a =	1.67	1.82	1.97	2.11	2.26	2.40	2.54	2.68	2.82	2.96	3.09	3.23
	b =	0.55	0.71	0.87	1.05	1.24	1.44	1.64	1.86	2.09	2.34	2.59	2.85
	p =	0.74	0.71	0.68	0.66	0.64	0.63	0.62	0.61	0.60	0.59	0.58	9.57
47	v =	2.62	3.08	3.57	4.08	4.63	5.22	5.83	6.47	7.15	7.85	8.59	9.36
	a =	1.69	1.84	1.98	2.12	2.27	2.41	2.55	2.69	2.83	2.96	3.10	3.24
	b =	0.63	0.79	0.97	1.16	1.36	1.58	1.80	2.04	2.28	2.54	2.81	3.09
	p =	0.72	0.69	0.67	0.65	0.63	0.62	0.61	0.60	0.59	0.58	0.58	0.57

$$\text{Volume} = vD_a^3 \tag{D3}$$

$$\text{Cross-sectional area} = aD_a^2 \tag{D4}$$

For example, the crater from calibration shot 3 has the following dimensions:

R_a = 21.5 ft
D_a = 11.5 ft
θ = 38 deg
R_a/D_a = 1.87

Values in the tables are computed for decimal increments of R_a/D_a and it is necessary to interpolate for v, a, b and p:

	θ (deg)	R_a/D_a	v	a	b	p
From Table D1:	38	1.8	4.23	2.14	0.49	0.75
	38	1.9	4.81	2.29	0.60	0.73
Interpolating:	38	1.87	4.64	2.25	0.57	0.74

The volume of the crater may now be computed:

$$\text{Volume} = (4.64) \times (11.5 \text{ ft})^3 = 7040 \text{ ft}^3$$

The cratering curves resulting from the calibration series are presented in Fig. D4. Volumes for calibration shots 1 through 3 were computed in the manner described above. The crater slope for calibration shot 4 was only 23 deg and Table D1 could not be used. Calibration shot 5 produced no crater. The criterion for the selection of the optimum depth of burial is that the crater volume be a maximum. This maximum is picked from the curve and will generally occur at a depth intermediate between those for which the crater radius and depth are a maximum. On the basis of Fig. D4 the maximum volume and consequent optimum dimensions were selected as:

$$V_{max} = 8200 \text{ ft}^3$$

$$\text{DOB} = 18.0 \text{ ft}$$

$$R_a = 23.4 \text{ ft}$$

$$D_a = 13.2 \text{ ft}$$

Because 1-ton charges were used, these are also the "scaled" optimum parameters in $\text{ft/ton}^{0.3}$ for this explosive. To convert these values to a TNT base the appropriate "cratering effectiveness" value from Table 9 must be applied. Since a 20% Al-AN slurry blasting agent is 1.5 to 1.7 times more effective for cratering than TNT, it follows that each ton of slurry is equivalent to 1.5 to 1.7; say, 1.6 tons of TNT. Thus each of the above values (DOB, R_a and D_a) must be divided by the factor $(1.6)^{0.3}$ to obtain the "scaled" optimum parameters (dob, r_a and d_a) in $\text{ft/ton}^{0.3}$ for TNT. These values are:

$$\text{dob} = 16 \text{ ft/ton}^{0.3} \text{ of TNT or equivalent}$$

$$r_a = 20 \text{ ft/ton}^{0.3} \text{ of TNT or equivalent}$$

$$d_a = 11 \text{ ft/ton}^{0.3} \text{ of TNT or equivalent}$$

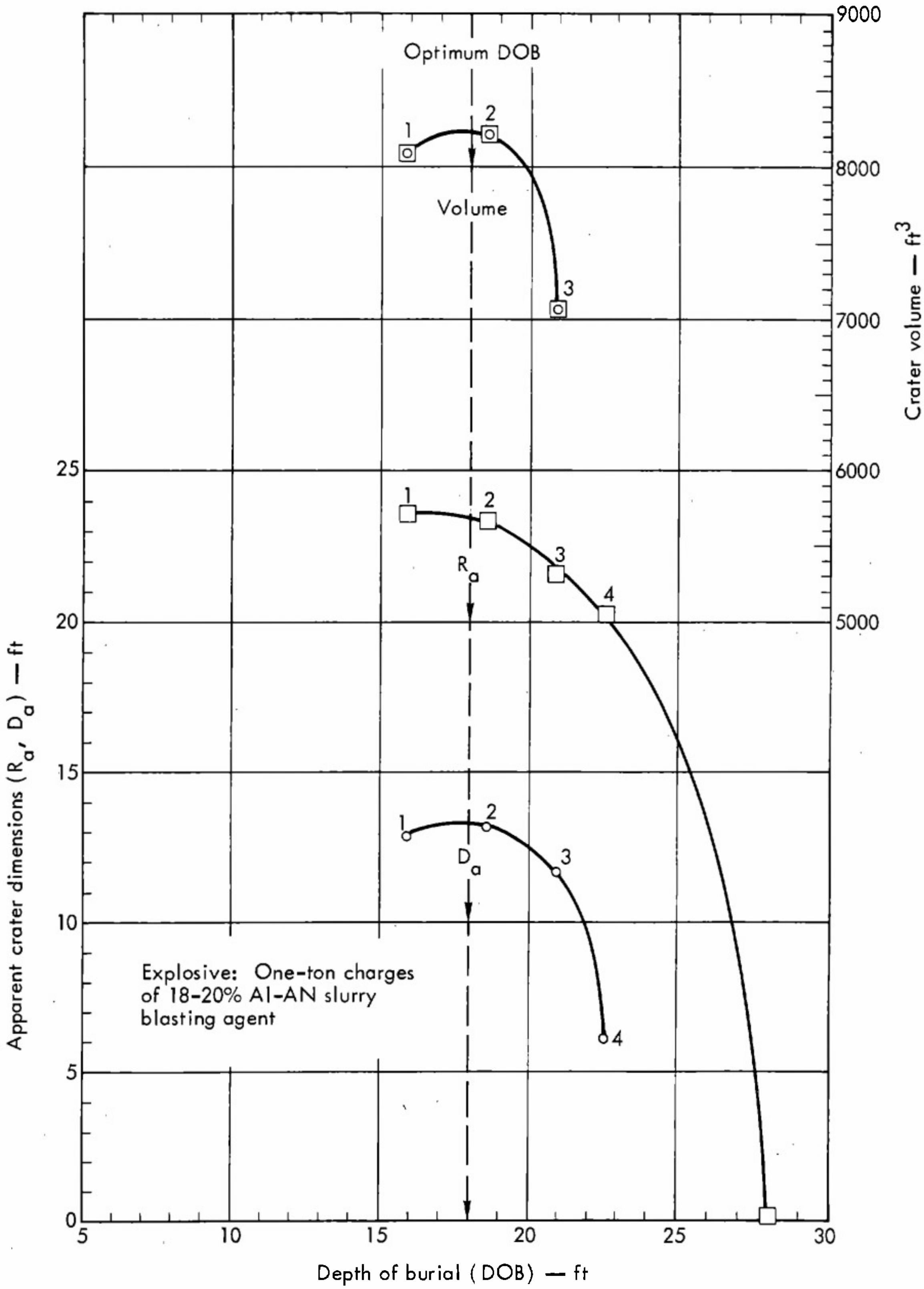


Fig. D4. Cratering curves from calibration series.

Since the profiles for calibration shot 2 closely approximate optimum conditions, the average crater slope from shot 2 can be used with the above values to obtain the parameters "b" and "p" for use in Eq. (17). Thus, the remaining parameters associated with optimum conditions are selected as:

$$\theta = 33 \text{ deg}$$

$$\frac{r_a}{d_a} = 1.8$$

$$v = 3.85$$

$$a = 2.00$$

$$b = 0.18$$

$$1 = 0.87$$

D.3 CRATER DIMENSION DESIGN CHARTS

Among the most useful design charts for explosive excavation projects is a plot of crater dimension data in the form illustrated in Figs. 17 through 19. Such a plot is easily constructed from the optimum parameters obtained from the calibration shots. Since crater dimensions scale as the charge weight raised to the exponent 0.3, a logarithmic plot of crater dimensions vs charge weight will result in linear curves. A practical chart for most applications can be constructed with 3-cycle × 3-cycle logarithmic graph paper with the crater dimension scale varying from 1 to 1000 ft, the charge weight scale varying from 0.1 to 100 tons, and the volume scale varying from 100 to 100,000 yd^3. The equations for the curves are given by:

$$\text{Radius} = (r_a) \times (\text{charge weight})^{0.3}, \tag{D5}$$

$$\text{Depth} = (d_a) \times (\text{charge weight})^{0.3}, \tag{D6}$$

$$\text{Volume} = (Vd_a^3/27) \times (\text{charge weight})^{0.9}\ yd^3, \tag{D7}$$

where r_a and d_a are the "scaled" optimum crater dimensions and Vd_a^3 is the "scaled" crater volume in cubic feet associated with optimum conditions. The depth of burial scale is similarly obtained from the equation:

$$\text{Depth of burial} = (\text{dob}) \times (\text{charge weight})^{0.3}, \tag{D8}$$

where dob is the "scaled" optimum depth of burial.

The completed design chart for the project under consideration is shown in Fig. D5.

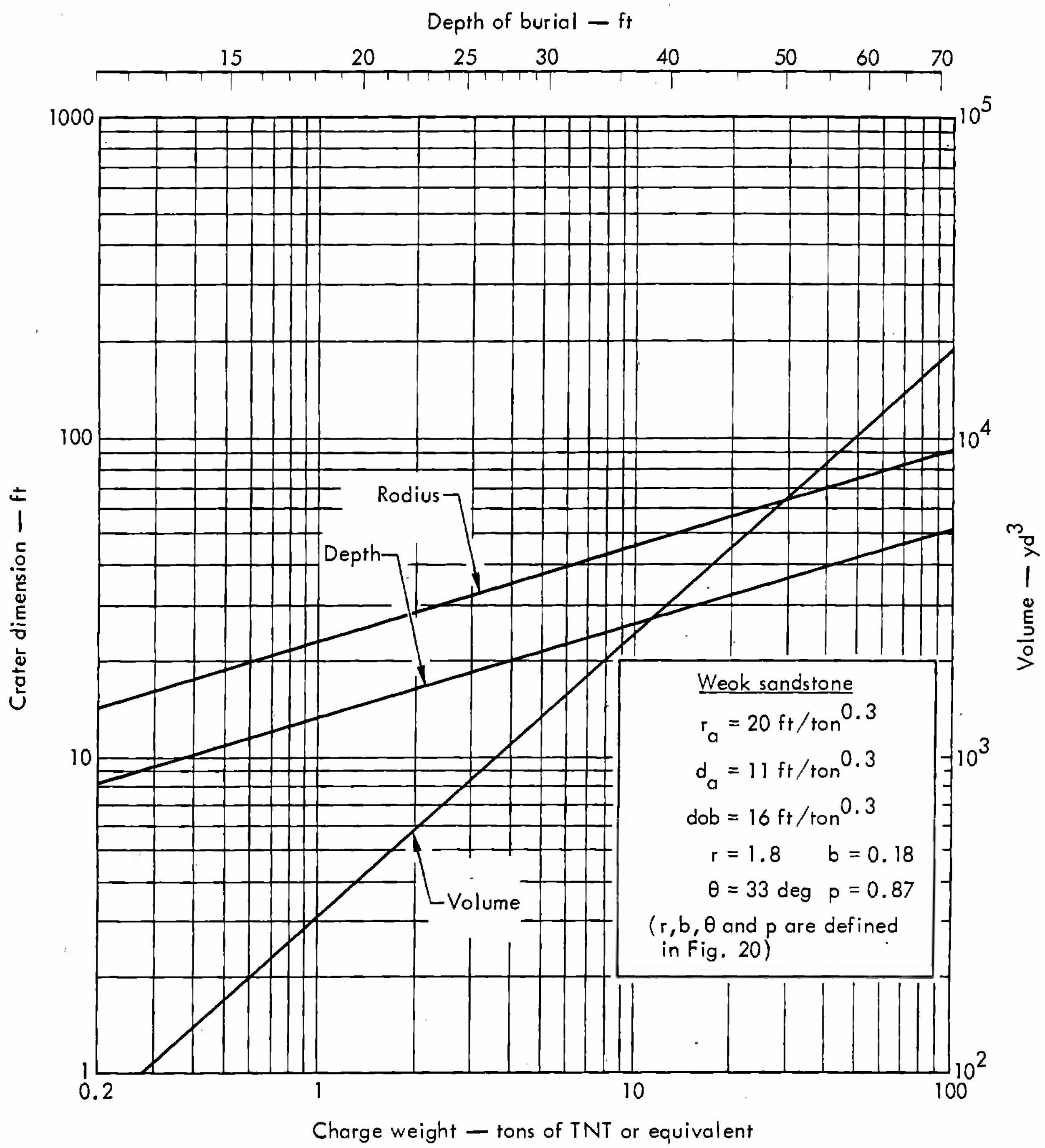


Fig. D5. Crater dimension data developed from calibration shots.

Appendix E
Expedient Design

E.1 SCOPE

This appendix is a supplement to Chapter 5. Its purpose is twofold. First, it provides details concerning the development and construction of special dimensionless crater design charts not covered in Chapter 5, but which have considerable potential for facilitating the design of certain types of excavation projects. Second, it uses the dimensionless charts and other design curves as the basis for an initial approach toward simplification of the row-charge crater design procedures in Chapter 5.

E.2 CRATER CROSS SECTIONS IN DIMENSIONLESS FORM

The typical hyperbolic cross section of a row-charge crater can be plotted in a convenient dimensionless form by using one of the characteristic dimensions (depth, D_{ar}, or half-width, $W_a/2$, associated with the optimum crater) to normalize linear measurements. Charts prepared in this manner are particularly useful for calculating the size of the crater necessary to achieve a specified width at a certain depth below a specified datum as for a navigation prism in a channel excavation.

The hyperbolic cross section of a row-charge crater is shown in Fig. E1. Using the same convention introduced in Fig. 20, the equation for the hyperbola is given by:

$$y^2 - (\tan^2\theta)x^2 = B^2, \tag{E1}$$

where (refer to Fig. E1)

$x = L/2$ ($L/2$ varies within the range 0 to $W_a/2$)

$y = B + D_{ar} - D$

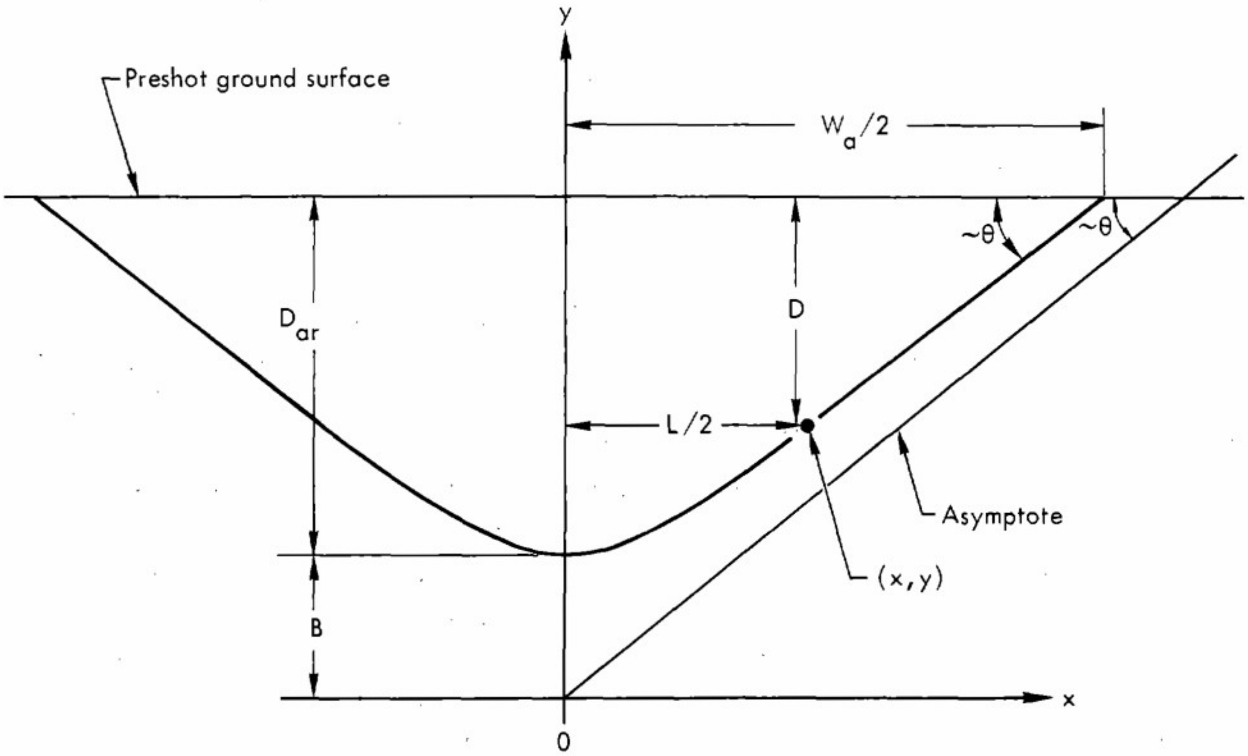
θ = fallback slope angle (assumed to be the slope of the asymptotes of the hyperbola)

$B = (b \cdot D_{ar})$

$$b = \frac{1}{2}\left[\left(\frac{W_a/2}{D_{ar}}\right)^2 \tan^2\theta - 1\right]$$

D_{ar} = characteristic optimum depth of crater

$W_a/2$ = characteristic optimum half-width of row crater.



Equation of hyperbola:

$$y^2 - (\tan^2\theta)\,x^2 = B^2$$

Fig. E1. Typical hyperbolic row-crater cross section.

If the indicated substitutions are made for x, y, and B, Eq. (E1) can be put in the form:

$$\frac{D}{D_{ar}} = b + 1 - \left[b^2 + \left(\frac{L/2}{D_{ar}}\right)^2 \tan^2\theta\right]^{1/2}, \tag{E2}$$

where the variables x and y are expressed in terms of L/2 and D, parameters which define the hyperbola by specifying the half-width and depth at any point along the crater profile and which have been normalized by the characteristic depth, D_{ar}. [Optimum values of D_{ar}, $W_a/2$ and θ for a particular medium would be used in Eq. (E2)].

A plot of $\frac{L/2}{D_{ar}}$ vs D/D_{ar} can be made; however it is more useful to normalize to the half-width, $W_a/2$, rather than the depth, D_{ar}. The simplest way of doing this is to note that the ratio, $\frac{D_{ar}}{W_a/2}$, is a constant for a particular medium* and thus:

*$D_{ar} = ed_a Y^{0.3}$; $W_a/2 = er_a Y^{0.3}$. Thus the ratio $\left(\frac{D_{ar}}{W_a/2}\right) = \frac{d_a}{r_a}$ and is a function only of the medium.

$$\left(\frac{L/2}{W_a/2}\right) = \left(\frac{L/2}{D_{ar}}\right)\left(\frac{D_{ar}}{W_a/2}\right), \tag{E3}$$

and

$$\left(\frac{D}{W_a/2}\right) = \left(\frac{D}{D_{ar}}\right)\left(\frac{D_{ar}}{W_a/2}\right). \tag{E4}$$

Equations (E3) and (E4) can be used to plot the "optimum" hyperbolic row-charge crater cross section in dimensionless form, where the dimensions L/2 and D are normalized by the row crater half-width, $W_a/2$.

With certain additional assumptions, dimensionless plots of the crater cross sections resulting from the detonation of two parallel rows of charges may be constructed. If the excavation between the rows is assumed to result in a relatively flat bottom and the outside boundaries retain the hyperbolic shape, then the double row-charge crater is seen as a single row-charge crater which has been cut in two and the two parts pushed apart a distance equal to the separation between the rows of charges. This separation is assumed to be 1.5 $(W_a/2)$ for optimum cratering performance (see Section 5.4).

Construction of these dimensionless plots is illustrated for the dry rock medium discussed in Chapters 2 and 5 having the following characteristic parameters for 1-ton TNT charges:

$$W_a/2 = 20 \text{ ft}$$

$$D_{ar} = 10 \text{ ft}$$

$$\theta = 37 \text{ deg}$$

$$b = 0.635$$

Table E1 shows the computations for the points needed to plot the dimensionless curves for a single row and for a double row of charges in dry rock. Columns 7 and 8 are used to plot the single row curve in Fig. E2. Columns 7 and 9 are used to plot the double row curve in Fig. E3. Each value $\left(\frac{L/2}{D}\right)$ in Figs. E2 and E3 is a fixed ratio of the abscissa to the ordinate and thus is analogous to the slope of an associated radial line.

Curves for the dry soil and saturated clay shale media discussed in Chapters 2 and 5 have been developed in a similar manner and are also drawn in Figs. E2 and E3. It should be noted that additional plots need not be constructed unless a project medium is considerably different from those for which curves are presented in Figs. E2 and E3. In such a case, parameters would be obtained by site calibration procedures (see Appendix D) and the necessary curves would be developed and constructed as shown in this section.

Table E1. Calculations for dimensionless plots of craters in dry rock.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)[a]
Assumed $\left(\frac{L/2}{D_{ar}}\right)$	$\left(\frac{L/2}{D_{ar}}\right)^2$	$\tan^2\theta \times \left(\frac{L/2}{D_{ar}}\right)^2$	$\tan^2\theta \times \left(\frac{L/2}{D_{ar}}\right)^2 + b^2$	$\sqrt{\tan^2\theta \left(\frac{L/2}{D_{ar}}\right)^2 + b^2}$	$\frac{D}{D_{ar}} = b + 1 - \sqrt{\tan^2\theta \left(\frac{L/2}{D_{ar}}\right)^2 + b^2}$	$\frac{D}{W_a/2} = \left(\frac{D}{D_{ar}}\right) \times \left(\frac{D_{ar}}{W_a/2}\right)$	$\frac{L/2}{W_a/2} = \left(\frac{L/2}{D_{ar}}\right) \times \left(\frac{D_{ar}}{W_a/2}\right)$	$\frac{S'}{2} + \frac{L/2}{W_a/2}$
0.00	0.000	0.000	0.410	0.640	1.000	0.500	0.000	0.750
0.25	0.063	0.035	0.446	0.667	0.973	0.486	0.125	0.875
0.50	0.250	0.142	0.552	0.743	0.897	0.448	0.250	1.000
0.75	0.563	0.320	0.730	0.854	0.786	0.393	0.375	1.125
1.00	1.000	0.568	0.970	0.985	0.655	0.327	0.500	1.250
1.25	1.563	0.888	1.300	1.140	0.500	0.250	0.625	1.375
1.50	2.250	1.280	1.690	1.300	0.340	0.170	0.750	1.500
1.75	3.063	1.740	2.150	1.470	0.170	0.085	0.875	1.625
2.00	4.000	2.270	2.680	1.640	0.000	0.000	1.000	1.750

[a]$S' = 1.5$ for optimum performance of a double row of charges.

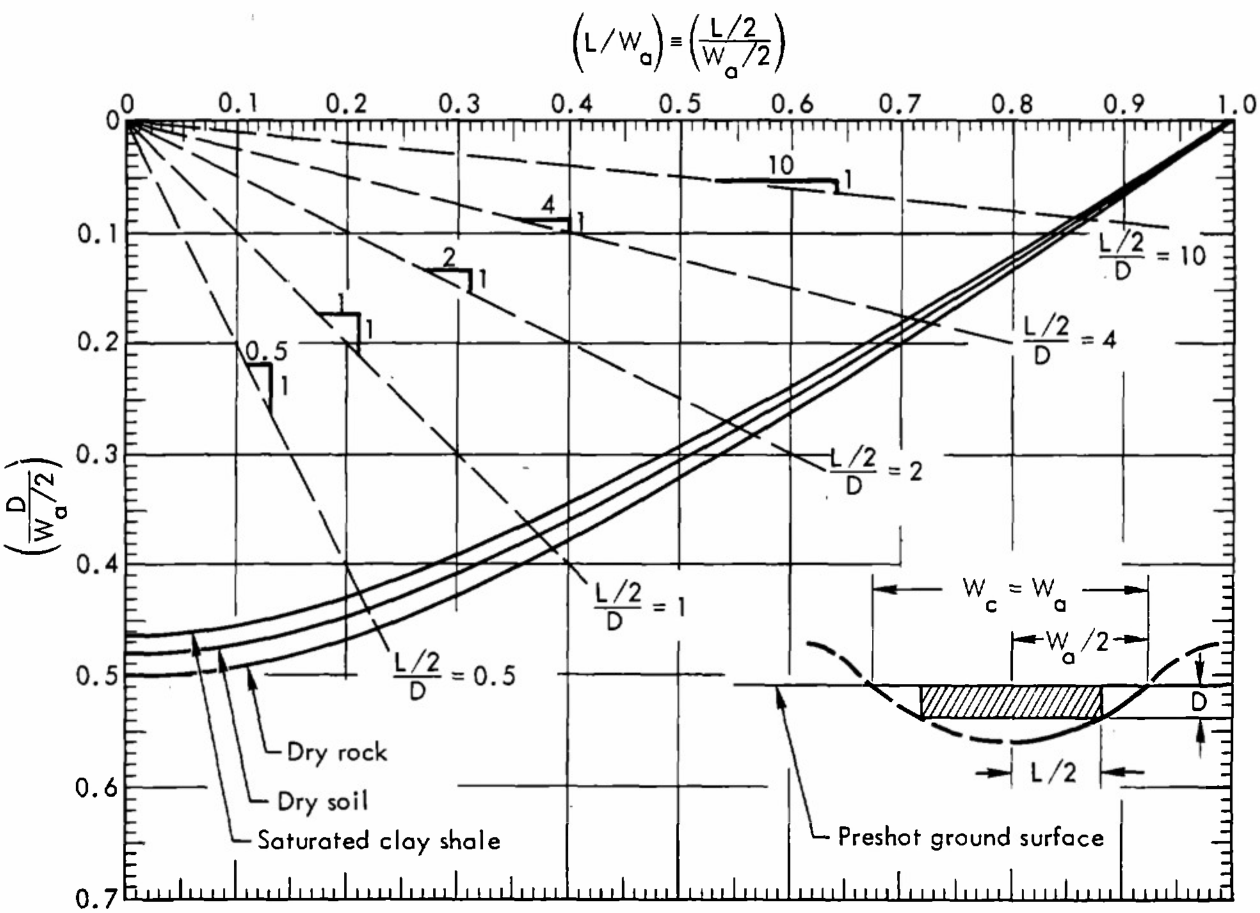


Fig. E2. Dimensionless plots of hyperbolic crater geometry, single row of charges.

E.3 EXPEDIENT DESIGN OF CERTAIN EXCAVATIONS

The foregoing discussion concerning the preparation of dimensionless charts has led to an investigation of the possibility of extension and further simplification of the procedures presented in Chapter 5. Work has been done to find ways to reduce

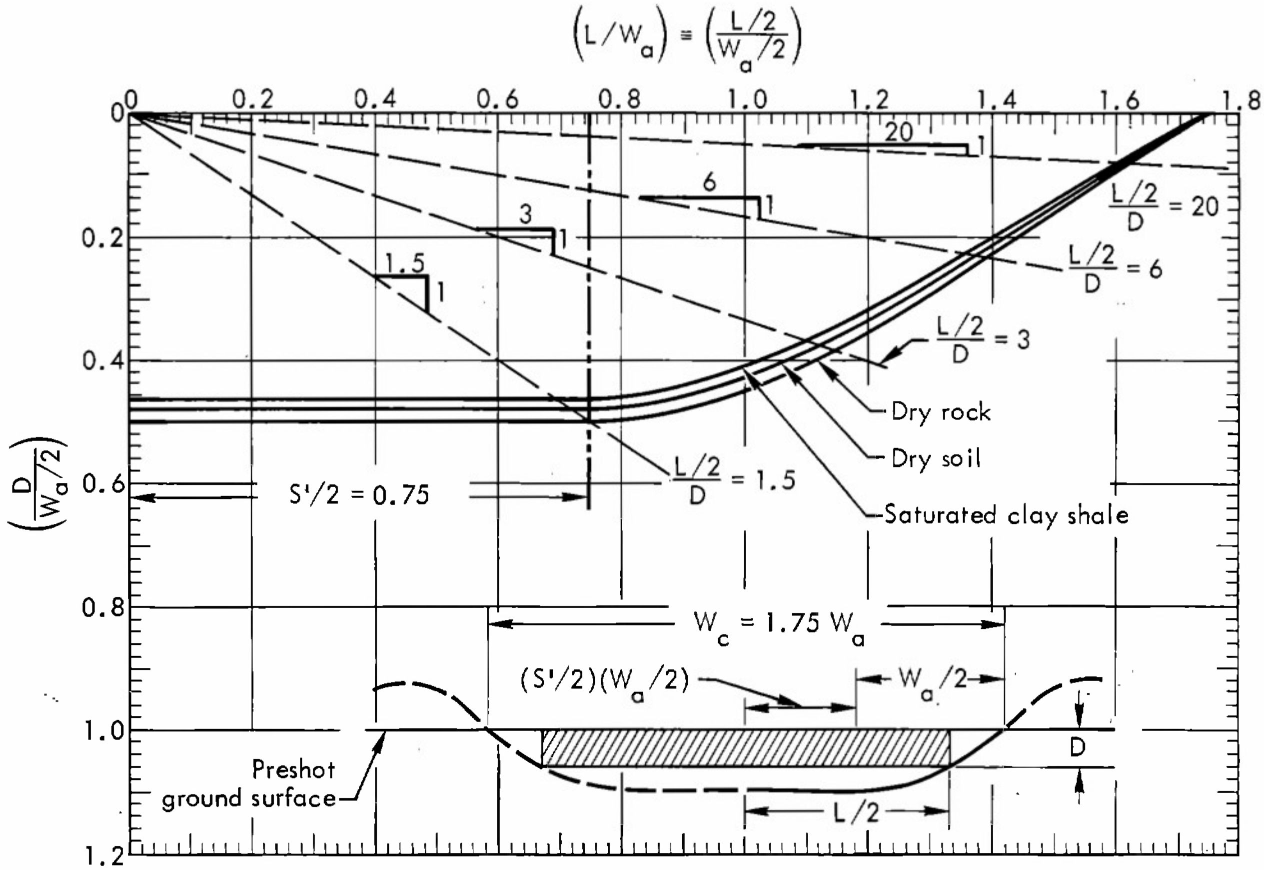


Fig. E3. Dimensionless plots of hyperbolic crater geometry, double row of charges.

time-consuming calculations. The dimensionless plots of the previous section are an example of progress toward this end. Additional work has been done to simplify the selection of charge locations in a row, eliminating the need to calculate an average spacing before placing the charge (as required by the procedures in Chapter 5). Finally, work has been done to consolidate design procedures into a single, tabular, step-by-step program utilizing dimensionless crater design charts and graphical plots to eliminate or simplify repetitive calculations. As of this writing, a systemmatic approach to design to replace the procedures in Chapter 5 is in a very early stage of development. The results of recent efforts are described below. It is expected that future editions of this report will reflect further progress toward this end.

Step-by-step procedures leading to the specification of charge weights, spacings, and depths of burial are outlined in Table E2. The design relationships in the table were developed from the relationships found in Chapters 2 and 5. To understand these procedures it is first necessary to recognize that there are three main variables in any explosive excavation design: charge weight (Y), in-row spacing (S), and enhancement (e). (Between-row spacing is assumed to be fixed at 1.5 ($W_a/2$) for a double row of charges.) Then it follows that by fixing one of these variables the other two can be determined in terms of project requirements. The procedures in Table E2 allow either

Table E2. Expedient excavation design procedures.

Step number (repeated?)	Procedure I[a] (L and D specified)		Procedure II[a] (D_{ar} specified)	
	Constant Y	Constant e	Constant Y	Constant e
1 (yes)	Determine L and D for cross section	Determine L and D for each cross section	Determine D_{ar} for each cross section	Determine D_{ar} for each cross section
2 (yes)	Determine L/W_a from Fig. E2 or Fig. E3	Determine L/W_a from Fig. E2 or Fig. E3	N/A	N/A
3 (no)	Determine Y_{max} (see Chapter 10)	Determine Y_{max} (see Chapter 10)	Determine Y_{max} (see Chapter 10)	Determine Y_{max} (see Chapter 10)
4 (no)	Select $Y = Y_{max}$ (use throughout design procedure)	Select enhancement $e = \left[\frac{L}{2r_a\,(L/W_a)}\right] Y_{max}^{-0.3}$ (use throughout design procedure)	Select $Y = Y_{max}$ (use throughout design procedure)	Select enhancement $e = \left[\frac{D_{ar}}{d_a}\right] Y_{max}^{-0.3}$ (use throughout design procedure)
5 (no)	Construct design curves of equations in Steps 6 to 11 (see text for example curves)	Construct design curves of equations in Steps 6 to 11 (see text for example curves)	Construct design curves of equations in Steps 6 to 11 (see text for example curves)	Construct design curves of equations in Steps 6 to 11 (see text for example curves)
6 (yes)	Determine in-row spacing $S = \left[\frac{5.6 r_a^3 Y^{0.9}}{L^2}\right] (L/W_a)^2$ (read from design curves)	Determine in-row spacing $S = \left[\frac{0.7L}{e^3}\right] (L/W_a)^{-1}$ (read from design curves)	Determine in-row spacing $S = \left[1.4 r_a d_a^2 Y^{0.9}\right] D_{ar}^{-2}$ (read from design curves)	Determine in-row spacing $S = \left[\frac{1.4 r_a}{d_a e^3}\right] D_{ar}$ (read from design curves)
7 (yes)	Check enhancement $e = \left[\frac{L}{2r_a Y^{0.3}}\right] (L/W_a)^{-1}$ (if $e < 1$, return to Step 5 and complete design using constant enhancement with $e = 1$) (read from design curves)	Determine charge size $Y = \left[\frac{L}{2r_a e}\right]^{3.33} (L/W_a)^{-3.33}$ (read from design curves)	Check enhancement $e = \left[\frac{1}{d_a Y^{0.3}}\right] D_{ar}$ (if $e < 1$, return to Step 5 and complete design using constant enhancement with $e = 1$) (read from design curves)	Determine charge size $Y = \left[\frac{1}{e d_a}\right]^{3.33} D_{ar}^{3.33}$ (read from design curves)
8 (yes)	Determine depth of burial $DOB = \left[\frac{(dob)L}{2r_a}\right] (L/W_a)^{-1}$ (read from design curves)	Determine depth of burial $DOB = \left[\frac{(dob)L}{2r_a}\right] (L/W_a)^{-1}$ (read from design curves)	Determine depth of burial $DOB = \left[\frac{dob}{d_a}\right] D_{ar}$ (read from design curves)	Determine depth of burial $DOB = \left[\frac{dob}{d_a}\right] D_{ar}$ (read from design curves)
9 (yes)	Determine between-row spacing $BRS = [0.75L]\,(L/W_a)^{-1}$ (double row only) (read from design curves)	Determine between-row spacing $BRS = [0.75L]\,(L/W_a)^{-1}$ (double row only) (read from design curves)	Determine between-row spacing $BRS = \left[\frac{1.5 r_a}{d_a}\right] D_{ar}$ (double row only) (read from design curves)	Determine between-row spacing $BRS = \left[\frac{1.5 r_a}{d_a}\right] D_{ar}$ (double row only) (read from design curves)
10 (yes)	Predict crater depth $D_{ar} = \left[\frac{d_a L}{2r_a}\right] (L/W_a)^{-1}$ (read from design curves)	Predict crater depth $D_{ar} = \left[\frac{d_a L}{2r_a}\right] (L/W_a)^{-1}$ (read from design curves)	N/A	N/A
11 (yes)	Predict crater width $W_c = \left[\left(1 + \frac{S'}{2}\right) L\right] (L/W_a)^{-1}$ where: $S' = 0$ for single row; $S' = 1.5$ for double row (read from design curves)	Predict crater width $W_c = \left[\left(1 + \frac{S'}{2}\right) L\right] (L/W_a)^{-1}$ where: $S' = 0$ for single row; $S' = 1.5$ for double row (read from design curves)	Predict crater width $W_c = \left[(2 + S')\,\frac{r_a}{d_a}\right] D_{ar}$ where: $S' = 0$ for single row; $S' = 1.5$ for double row (read from design curves)	Predict crater width $W_c = \left[(2 + S')\,\frac{r_a}{d_a}\right] D_{ar}$ where: $S' = 0$ for single row; $S' = 1.5$ for double row (read from design curves)
12 (yes)	Determine effective spacing $S_e = S\left[1 \pm \frac{\%\ slope}{200}\right]$ where: (+) = proceeding upslope; (-) = proceeding downslope; % slope = local av slope	Determine effective spacing $S_e = S\left[1 \pm \frac{\%\ slope}{200}\right]$ where: (+) = proceeding upslope; (-) = proceeding downslope; % slope = local av slope	Determine effective spacing $S_e = S\left[1 \pm \frac{\%\ slope}{200}\right]$ where: (+) = proceeding upslope; (-) = proceeding downslope; % slope = local av slope	Determine effective spacing $S_e = S\left[1 \pm \frac{\%\ slope}{200}\right]$ where: (+) = proceeding upslope; (-) = proceeding downslope; % slope = local av slope

Table E2 (continued)

Step number (repeated?)	Procedure I[a] (L and D specified)		Procedure II[a] (D_{ar} specified)	
	Constant Y	Constant e	Constant Y	Constant e
13 (yes)	Consider cross section corresponding to maximum depth of cut for first charge(s). Place charge(s) Y_1 at DOB_1 on this cross section. Move a distance $(S_e)_1$ in either direction along the centerline of the excavation from charge(s) Y_1 to consider the cross section for the second charge(s) Y_2. Place charge(s) Y_2 at DOB_2 on this cross section. Move a distance $(S_e)_2$ in the same direction from charge(s) Y_2 to consider the cross section for the next charge(s) Y_3. Repeat this procedure until the design is completed over the entire length of the excavation.			

[a]Cost analyses show that a single row of charges should be used rather than a double row if charge weight and spacing considerations permit.

charge weight or enhancement to be specified and held constant throughout the design. Procedure I is used when a width of cut (L) at a certain depth (D) is specified; Procedure II is used when only a depth of excavation is specified. Both procedures are illustrated in the following section.

E.4 EXAMPLE PROBLEMS

a. Use of Procedure I

Consider the navigation prism example problem presented in Section 5.3 and represented by the profile in Fig. 23. A portion of that profile is shown in Fig. E4. It is

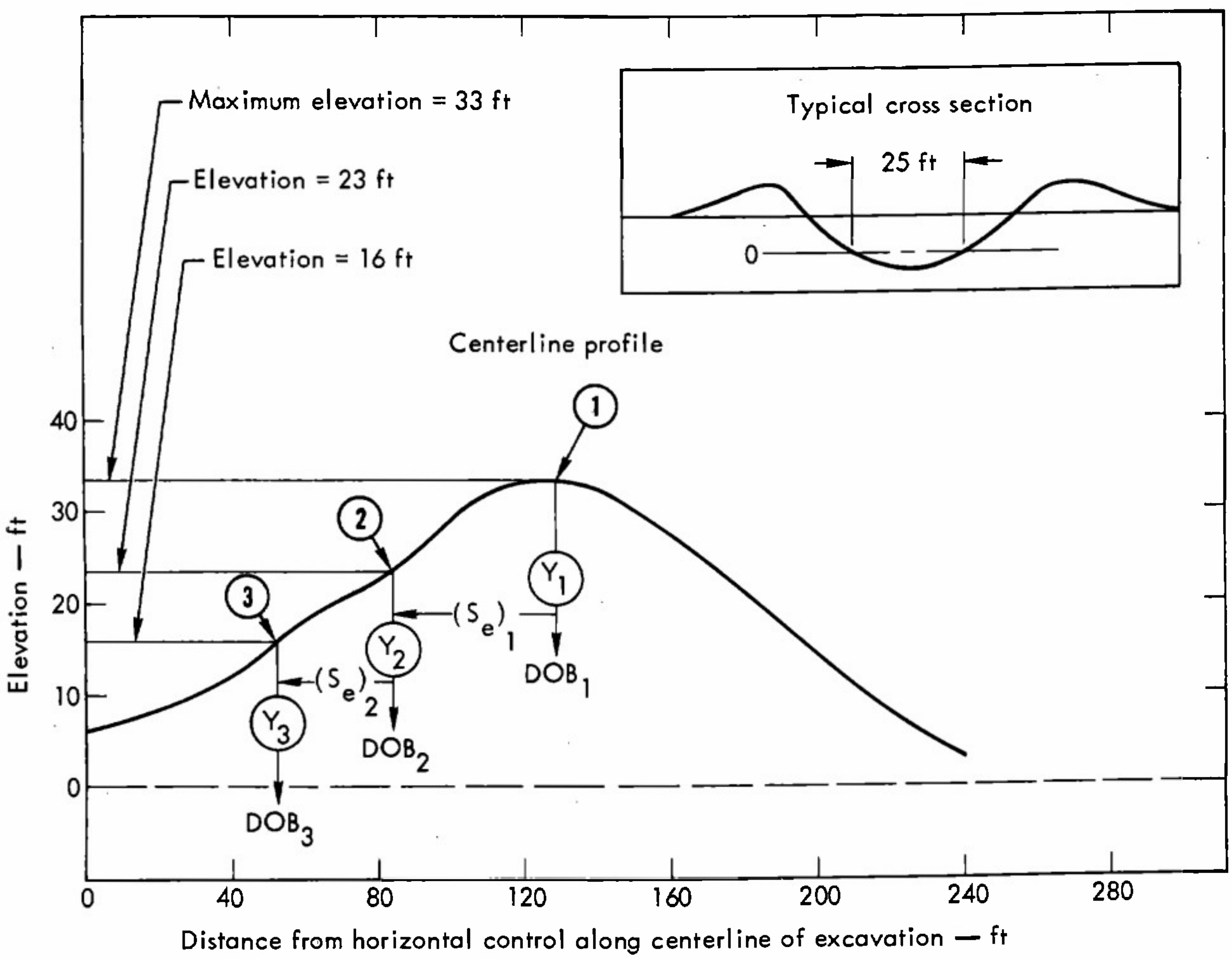


Fig. E4. Row-charge crater with navigation prism through varying terrain (reference, Fig. 23).

desired to excavate a channel through dry rock such that the channel width at elevation zero will be 25 ft. The highest elevation along the profile is 33 ft, and a maximum charge size of 30 tons of TNT or equivalent is to be used.

The method of constant enhancement will be used. Begin the design at the cross section for which the maximum cut must be made, in this case at the point of maximum elevation. The numbers enclosed in circles refer to the cross section under consideration; i.e., first, second, etc.

1 Step 1: $L = 25$ ft at $D_1 = 33$ ft (from Fig. E4)

$$\left(\frac{L/2}{D_1}\right) = 0.378$$

Step 2: $L/W_a = 0.180$ (from Fig. E2)

Step 3: $Y_{max} = 30$ tons of TNT or equivalent

Step 4: $r_a = 20 \text{ ft/ton}^{0.3}$

$$e = \left[\frac{L}{2R_a\ (L/W_a)}\right] Y_{max}^{-0.3} = 1.25$$

Step 5: Using the given or determined values for L, e, r_a, d_a, and S', the equations in Steps 6 through 11 reduce to

$$S = 8.96\ (L/W_a)^{-1}$$

$$Y = 0.10\ (L/W_a)^{-3.33}$$

$$DOB = 11.25\ (L/W_a)^{-1}$$

$$BRS = 0 \text{ (between-row spacing not applicable)}$$

$$D_{ar} = 6.25\ (L/W_a)^{-1}$$

$$W_c = 25\ (L/W_a)^{-1}$$

These equations are best plotted on logarithmic graph paper as illustrated in Fig. E5.

Step 6: $S_1 = 50$ ft (from Fig. E5)

Step 7: $Y_1 = 30$ tons (from Fig. E5)

Step 8: $DOB_1 = 62$ ft (from Fig. E5)

Step 9: Not applicable

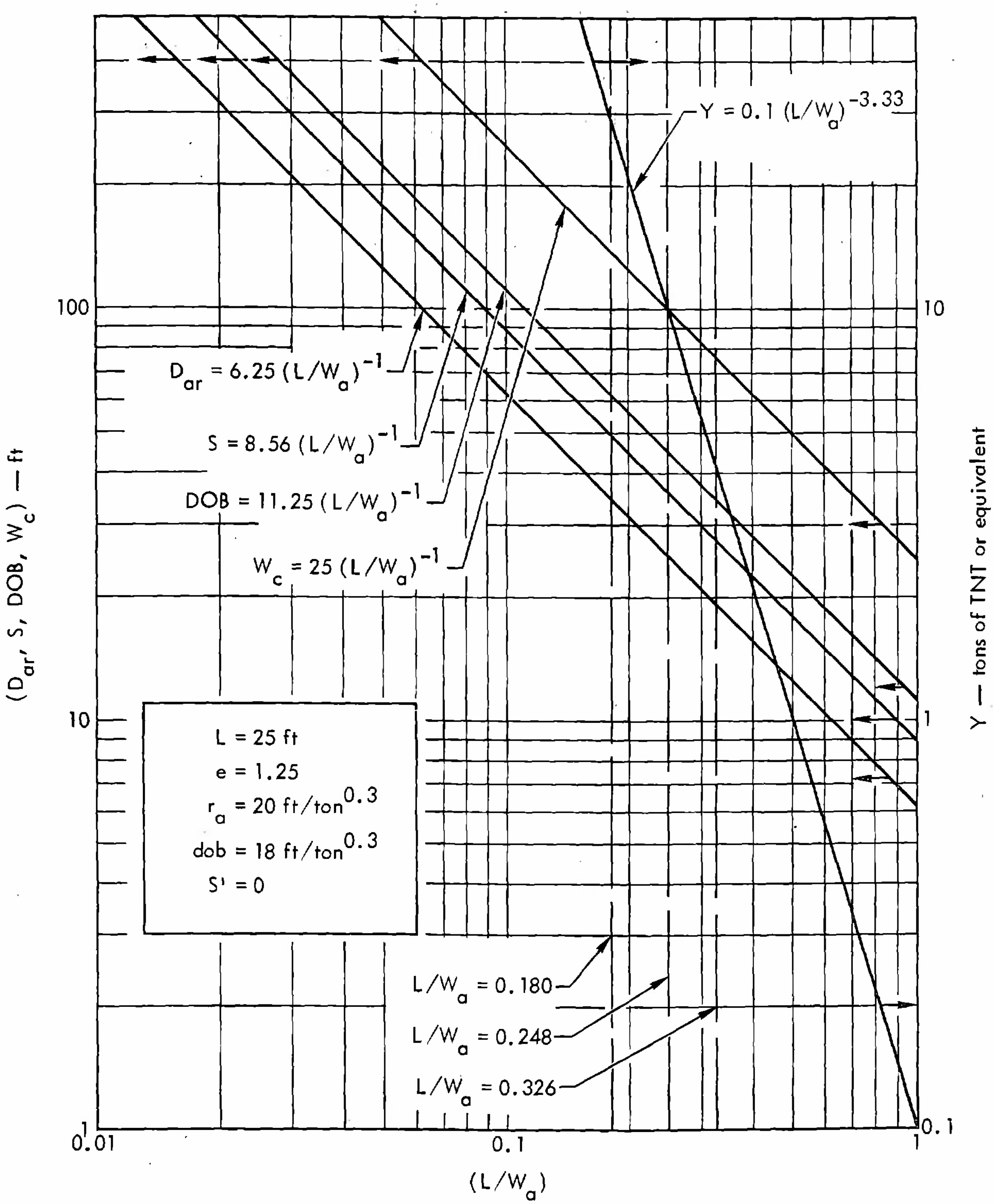


Fig. E5. Design curves for navigation prism example problem.

Step 10: D_{ar} = 35 ft (from Fig. E5)

Step 11: W_c = 140 ft (from Fig. E5)

Step 12: % slope = 23

$$(S_e)_1 = 50\left(1 - \frac{23}{200}\right) = 44 \text{ ft}$$

Step 13: Place charge Y_1 under the point of maximum elevation at a depth of burial DOB_1. Move a distance $(S_e)_1$ to either side and repeat applicable Steps 1 through 13 (see Fig. E4).

(2) Step 1: At a point 44 ft to the left of the previous charge the elevation is 23 ft and thus

$$L = 25 \text{ ft at } D_2 = 23 \text{ ft (from Fig. E4)}$$

$$\left(\frac{L/2}{D_2}\right) = 0.543$$

Step 2: $L/W_a = 0.248$ (from Fig. E2)

Step 3: Not repeated

Step 4: Not repeated

Step 5: Not repeated

Step 6: $S_2 = 36$ ft (from Fig. E5)

Step 7: $Y_2 = 10.5$ tons (from Fig. E5)

Step 8: $DOB_2 = 45$ ft (from Fig. E5)

Step 9: Not applicable

Step 10: $D_{ar} = 25$ ft (from Fig. E5)

Step 11: $W_c = 100$ ft (from Fig. E5)

Step 12: % slope = 24

$$(S_e)_2 = 36\left(1 - \frac{24}{200}\right) = 32 \text{ ft}$$

Step 13: Place charge Y_2 at a distance $(S_e)_1$ from charge Y_1 and at a depth of burial DOB_2. Charge Y_2 will thus be 44 ft to the left of charge Y_1. Move a distance $(S_e)_2$ to the left of charge Y_2 and repeat applicable Steps 1 through 13 (see Fig. E4).

(3) Step 1: At a point 32 ft to the left of the previous charge the elevation is 16 ft and thus

$$L = 25 \text{ ft at } D_3 = 16 \text{ ft (from Fig. E4)}$$

$$\left(\frac{L/2}{D_3}\right) = 0.781$$

Step 2: $L/W_a = 0.326$ (from Fig. E2)

Step 3: Not repeated

Step 4: Not repeated

Step 5: Not repeated

Step 6: $S_3 = 27$ ft (from Fig. E5)

Step 7: $Y_3 = 4.2$ tons (from Fig. E5)

Step 8: $DOB_3 = 35$ ft (from Fig. E5)

Step 9: Not applicable

Step 10: $D_{ar} = 19$ ft (from Fig. E5)

Step 11: $W_c = 76$ ft (from Fig. E5)

Step 12: % slope = 22

$$(S_e)_3 = 27 \left(1 - \frac{22}{200}\right) = 24 \text{ ft}$$

Step 13: Place charge Y_3 at a distance $(S_e)_2$ from charge Y_a and at a depth of burial DOB_3. Charge Y_3 will thus be 32 ft to the left of charge Y_2. Move a distance $(S_e)_3$ to the left of charge Y_3 and repeat applicable Steps 1 through 13 (see Fig. E4).

Continue the procedure until the left limit of the required excavation is reached. Then complete the design for the right portion of the cut in the same manner.

A comparison between this example and the corresponding example in Chapter 5 reveals that the repetitive square root calculations in the latter have been eliminated by the use of the dimensionless design charts. It was necessary to construct a set of graphs for the solution of the example above; however, once constructed they were applicable throughout, and a net savings in time and effort are certain to accrue in any realistic design problem. The method of spacing introduced here is revolutionary and somewhat unproved. It is more conservative than the approach presented in Chapter 5, and the resultant yields are slightly larger as a consequence. Its primary advantages lie in the fact that a given charge is placed on the cross section from which it was determined and this process is accomplished in a single step. The major disadvantages of these design procedures are the necessity for uniform terrain and the inflexibility to changes in basic parameters or design considerations during the design process.

b. Use of Procedure II

Consider the cutoff channel design problem presented in Section 10.5 and represented by the profile in Fig. 45. A portion of that profile is shown in Fig. E6. It is

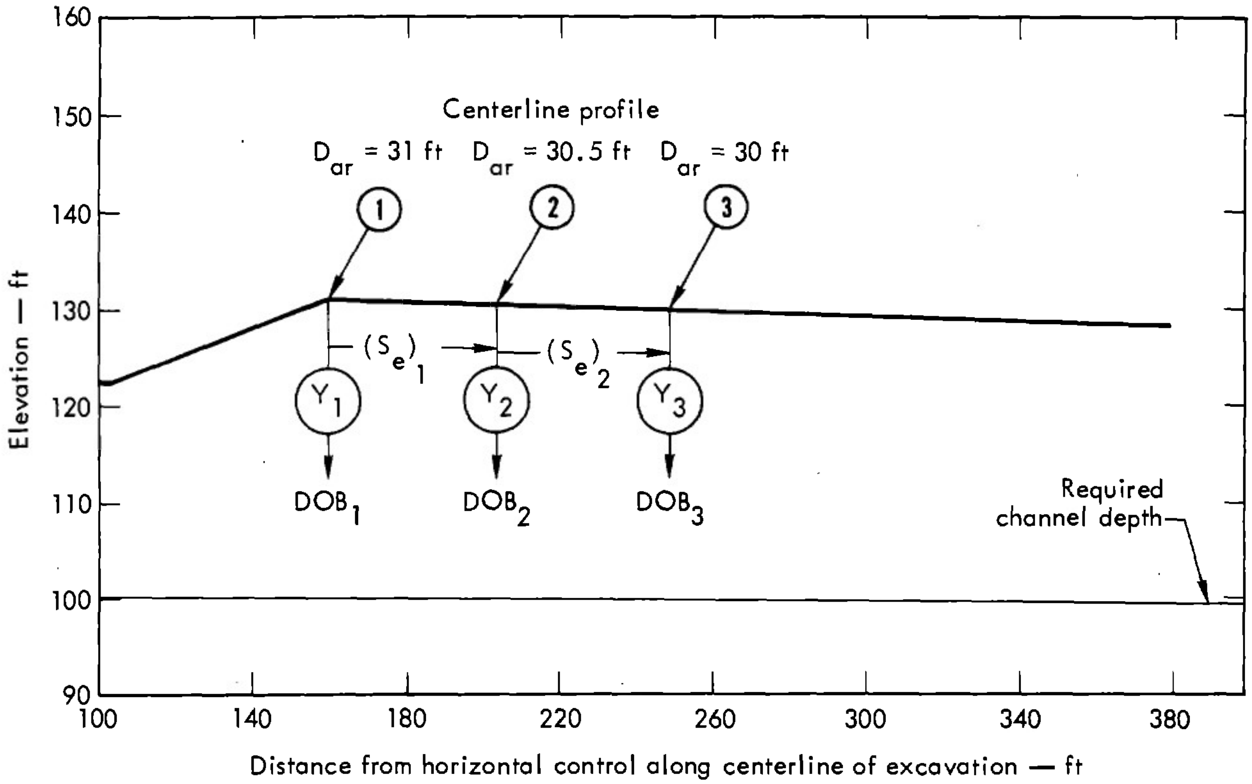


Fig. E6. Cutoff channel profile (reference, Fig. 45).

desired to excavate a 23-ft deep cutoff channel in dry rock. The maximum depth of cut will be 31 ft and a maximum charge size of 20 tons of TNT is to be used.

The method of constant charge weight will be used. Begin the design at the cross section for which the maximum cut must be made. The numbers enclosed in circles refer to the cross section under consideration; i.e., first, second, etc.

(1) Step 1: D_{ar} = 31 ft (from Fig. E6)

Step 2: Not applicable

Step 3: Y_{max} = 20 tons of TNT

Step 4: Y = 20 tons

Step 5: Using the given or determined values for Y, r_a, d_a, dob, and S', the equations in Steps 6 through 11 reduce to

$$S = 4.14 \times 10^4 D_{ar}^{-2}$$

$$e = 4.07 \times 10^{-2} D_{ar}$$

$$DOB = 1.8 D_{ar}$$

BRS = 0 (between row-spacing not applicable)

$$W_c = 4 D_{ar}$$

These equations are best plotted on logarithmic graph paper as illustrated in Fig. E7.

Step 6: $S_1 = 43$ ft (from Fig. E7)

Step 7: $e = 1.3$ (from Fig. E7)

Step 8: $DOB_1 = 56$ ft (from Fig. E7)

Step 9: Not applicable

Step 10: Not applicable

Step 11: $W_c = 125$ ft (from Fig. E7)

Step 12: % slope = 1

$$(S_e)_1 = 43\left(1 - \frac{1}{200}\right) = 43 \text{ ft}$$

Step 13: Place charge Y_1 under the point of maximum cut at a depth of burial DOB_1. Move a distance $(S_e)_1$ to either side and repeat applicable Steps 1 through 13 (see Fig. E6).

(2) Step 1: At a point 43 ft to the right of the previous charge the depth of cut is

$D_{ar} = 30.5$ ft (from Fig. E6)

Step 2: Not applicable

Step 3: Not repeated

Step 4: Not repeated

Step 5: Not repeated

Step 6: $S_2 = 45$ ft (from Fig. E7)

Step 7: $e = 1.25$ (from Fig. E7)

Step 8: $DOB_2 = 55$ ft (from Fig. E7)

Step 9: Not applicable

Step 10: Not applicable

Step 11: $W_c = 122$ ft (from Fig. E7)

Step 12: % slope = 1

$$(S_e)_2 = 45\left(1 - \frac{1}{200}\right) = 45 \text{ ft}$$

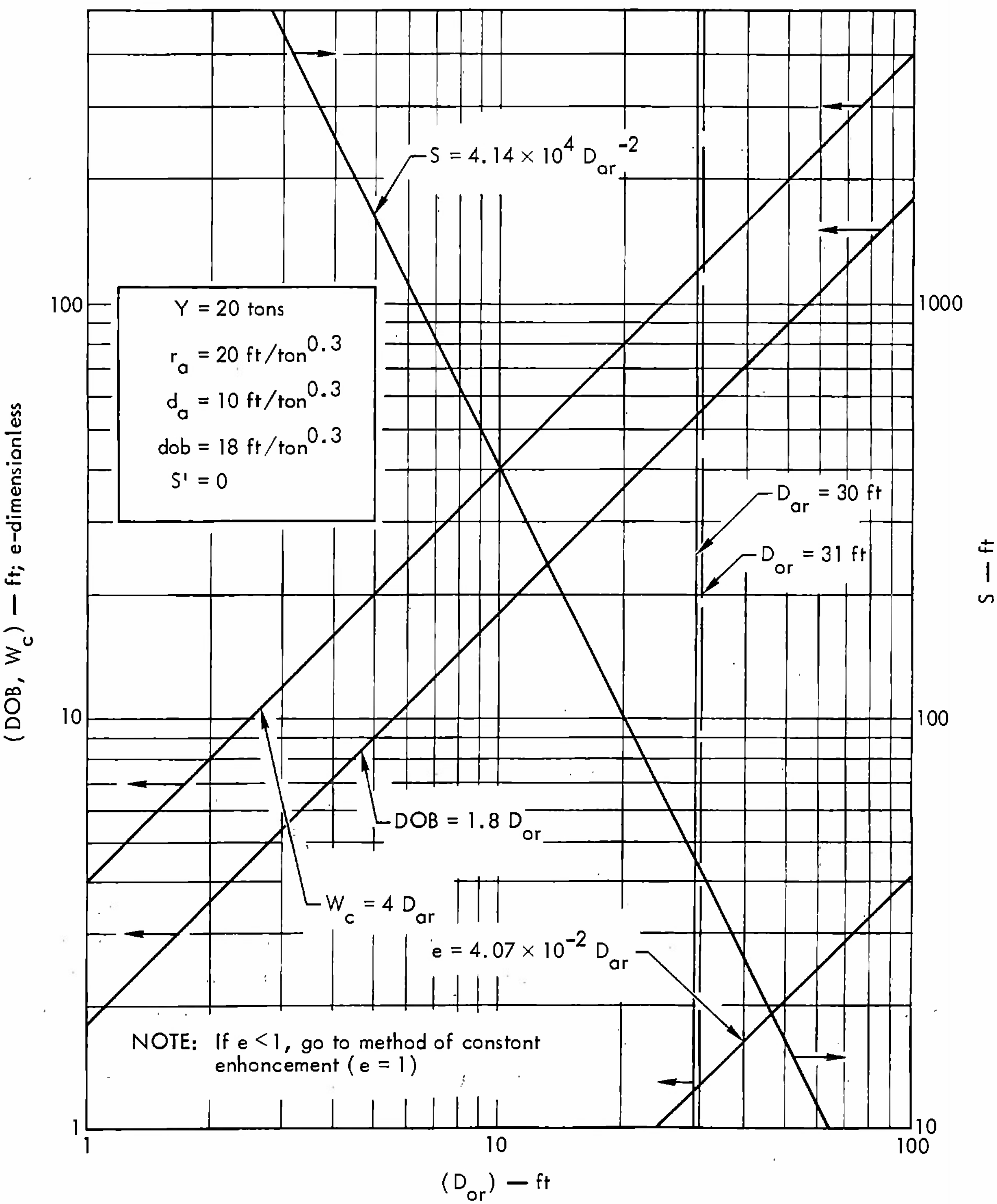


Fig. E7. Design curves for cutoff channel design problem.

Step 13: Place charge Y_2 at a distance $(S_e)_1$ from charge Y_1 and at a depth of burial DOB_2. Charge Y_2 will thus be 43 ft to the right of charge Y_1. Move a distance $(S_e)_2$ to the right of charge Y_2 and repeat applicable Steps 1 through 13 (see Fig. E6).

(3) Step 1: At a point 45 ft to the right of the previous charge the depth of cut is

D_{ar} = 30 ft (from Fig. E6)

Step 2: Not applicable

Step 3: Not repeated

Step 4: Not repeated

Step 5: Not repeated

Step 6: S_3 = 46 ft (from Fig. E7)

Step 7: e = 1.2 (from Fig. E7)

Step 8: DOB_3 = 54 ft

Step 9: Not applicable

Step 10: Not applicable

Step 11: W_c = 120 ft

Step 12: % slope = 1

$$(S_e)_3 = 45\left(1 - \frac{1}{200}\right) = 45 \text{ ft}$$

Step 13: Place charge Y_3 at a distance $(S_e)_2$ from charge Y_2 and at a depth of burial DOB_3. Charge Y_3 will thus be 45 ft to the right of charge Y_2. Move a distance $(S_e)_3$ to the right of charge Y_3 and repeat applicable Steps 1 through 13 (see Fig. E6).

Continue the above procedure until the right limit of the required excavation is reached. Then complete the design for the left portion of the cut in the same manner.

A comparison between this example and the corresponding example in Chapter 10 reveals no clearcut conclusion as to which procedure is easier. The above example requires the construction of a set of graphs to solve relatively simple equations. The primary advantages of the procedures used here are the systematic approach and the use of the new method of spacing. Work to simplify design procedures is continuing as of this writing.

Appendix F
Contract Guide Specifications

F.1 SCOPE

This appendix is a supplement to Chapter 7, and provides proposed guide specifications which can be used in contracting for explosive excavation projects. These specifications require the contractor to provide explosive procurement, charge emplacement, and arming and firing services in accordance with an excavation design. The format of the proposed specifications is that currently used by the Corps of Engineers in civil works construction.

F.2 GUIDE SPECIFICATIONS FOR EXPLOSIVE EXCAVATION PROJECTS

NOTES ON USE OF THIS GUIDE SPECIFICATION

1. This guide specification is to be used in the preparation of contract specifications in accordance with the ER-1110-2-1200. It will not be made a part of a contract merely by reference; pertinent portions will be copied verbatim into the contract documents.

2. Paragraph 2: The listed designations for publications are those in effect when the guide specification was being prepared. These designations will be changed, as required, to those in effect on the date of invitation for bids; and the nomenclature, types, grades, classes, etc., referenced in the guide will be checked for conformance to the latest revision or amendment. To minimize the possibility of error, the letter suffixes, amendments, and dates indicating specific issues will be omitted elsewhere in the specification. It is essential, therefore, that the list of applicable publications be retained in the contract specifications.

3. Paragraph 4: Fill in the blank.

4. Paragraph 6: After the type (or types) of blasting agent has been selected and the properties of the design explosive determined, the appropriate paragraphs should be numbered and the blanks filled in. If the specifications are opened up to allow a range of blasting agents to be used, a paragraph should be added to the specifications or notes on the contract drawings

to enable the Contractor to determine the correct charge weights for any substitute blasting agent.

5. Paragraph 6._ Slurry: If the charges will be emplaced in a dry environment delete the second parenthetical expression. If the charges will be emplaced in a wet environment, delete the first parenthetical expression.

6. Paragraph 6._.2.1 Bulk Density: If the slurry blasting agent will be emplaced under water, delete the parentheses and fill in the blank with the appropriate pressure for the anticipated head. If the charge will be emplaced in the dry, delete the parenthetical expression.

7. Paragraph 9: Fill in the blank.

SECTION __
EXPLOSIVE EXCAVATION

1. GENERAL: The Contractor shall perform all construction necessary to emplace the charges at the locations shown on the contract drawings, and shall furnish the specified explosives and required accessories, and shall place, prime and fire the explosive charges.

2. APPLICABLE PUBLICATIONS: The following publications of the issues listed below, but referred to thereafter by basic designation only, form a part of this specification to the extent indicated by the references thereto:

2.1 Corps of Engineers Manual:

EM 385-1-1, dated 1 March 1967	General Safety Requirements

2.2 Code of Federal Regulations:

CFR Title 46, Part 146, dated 1 January 1970	Shipping
CFR Title 26, Part 181, dated 15 January 1971	Commerce in Explosives: Definitions, Administrative Provisions, Licenses and Permits, Conduct of Operations, Records and Reports, Exemptions, Storage, Explosives Listing

2.3 Other publications:

National Fire Protection Association (NFPA) Code No. 495, dated 1970	Manufacture, Transportation, Storage and Use of Explosives and Blasting Agents
Institute of Makers of Explosives (IME) Publication No. 6, Approved 27 May 1968	Recommended Industrial Standards

3. CONTRACTOR'S EQUIPMENT AND METHODS: Before charge emplacement construction is started, the Contractor shall furnish to the Contracting Officer a complete list of major equipment which he proposes to use on the work, together with a description of the methods he proposes to use to perform the work.

4. CHARGE EMPLACEMENT CONSTRUCTION: Emplacement holes shall be excavated to the depth and at the location shown on the contract drawings. Construction shall be by drilling, with or without underreaming, or by other approved methods. The length-to-diameter ratio of the charges shall not

exceed __ and shall contain the weight of blasting agent specified to be furnished by the Contractor. The center of the charge shall be at the depth shown on the contract drawings for length-to-diameter ratios less than three. For length-to-diameter ratios greater then three, the lower third point of the charge shall be at the location shown on the contract drawings. The Contractor shall furnish and install any temporary liner necessary to insure the integrity of the hole until the explosives have been placed. Holes shall be completely stemmed with wet coarse sand or free running gravelly sand with a maximum particle size of 2 inches, after loading of the blasting agent and booster installation; or at the option of the Contractor the holes may be stemmed in advance and the charge loaded through a 4-inch maximum diameter combination fill and vent line. The stemming material shall be placed in lifts and tamped. If the blasting agent used will not support the stemming material without intermixing, or the Contractor elects to stem before emplacing the blasting agent, the Contractor shall submit a design for a stemming support system to the Contracting Officer for approval.

5. DRILLING LOG: An up-to-date and accurate written log shall be maintained at all times by the Contractor. The Contractor shall record the type of material, characteristics of strata encountered, the time required to drill each 5-foot interval, type of bit used and the time when the bit is changed, and any additional information that may be helpful in interpreting the drilling time log. All measurements for depths shall be referenced to existing ground surface at the drilling site. The Contractor shall verify depth measurements with the Contracting Officer.

6. BLASTING AGENT(S):

6._ Ammonium Nitrate and Fuel Oil: The AN/FO blasting agent shall consist of a homogeneous mixture of prilled ammonium nitrate and No. 2 fuel oil.

6. _ . 1 Emplacement: The blasting agent shall be capable of emplacement by pumping or tremie into the charge cavity through a minimum 4-inch diameter combination fill and vent line or open access hole.

6. _ . 2 Performance Specifications of the AN/FO Blasting Agent are as follows:

6. _ . 2. 1 Bulk Density: The density of the formulated mixture shall be a minimum of __ grams per cubic centimeter at one atmosphere. At least five independent samples shall be taken and the average, minimum and maximum density values reported. After emplacement the density of the blasting agent shall be within 5 percent of the average density as determined experimentally.

6. _ . 2. 2 Detonation Velocity: Shall be between _ and __ meters per second in a 6-inch diameter unconfined charge at 50°F. maximum. Measurement of detonation velocity shall be by Dautriche tests or time of arrival with measuring stations at least 0.5 meters apart. The first measuring station shall be at least 0.5 meters from the point of initiation. The charge may be detonated in air, underwater, or underground. At least five tests shall be run and the average, minimum and maximum values reported.

6. _ . 3 Underwater Gas Bubble Energy: Shall be between _ and __ gram-calories per gram at 50°F. maximum. This shall be measured by underwater test as follows: An approximately spherical lightly confined charge of approximately 20 pounds shall be suspended in water at a depth of about 30 feet and detonated from the center using a booster as specified in paragraph: "Boosters". The body of water must be at least 150 feet across and 60 feet deep. If a test pond of smaller dimensions is used it shall be normalized by detonating an equal weight of TNT in the location planned for the test samples. The measured T for the samples shall be corrected by the ratio of measured and known T's for TNT. A suitable pressure transducer with no more than 0.5-millisecond rise time shall be placed 15 to 30 feet from the charge and connected to an oscilloscope, or similar recording device, with a sweep speed accuracy of one percent at a sweep speed of 50 milliseconds per centimeter. The bubble period, defined as the time interval between the shock arrival time and the arrival

time of the peak of the first bubble pulse, shall be measured with an accuracy of at least one percent. Underwater gas bubble energy shall then be calculated by the formula:

$$E_b = \left[5.5T^3 (h + 2.31 P_o)^{2.5} - QB/2 \right] /W$$

where, E_b = underwater gas bubble energy, cal/gm

T = bubble period, seconds

h = depth of charge, feet of fresh water (if sea water, multiply h by 1.03)

P_o = local atmospheric pressure, PSIA

Q = heat of detonation of the booster explosive, cal/gm

B = weight of booster, pounds

W = net charge weight, pounds (excluding primers and boosters)

At least five underwater tests shall be performed. The average, minimum and maximum underwater gas bubble energies shall be reported.

6. Slurry: The slurry blasting agent may contain the following components: Ammonium and/or sodium nitrate and/or perchlorate, water, solvents, aluminum granules, powder, or flakes (particle size shall not exceed 0.5 millimeter or 40 mesh in its smallest dimension), gelling or cross linking agents, stabilizing agents as required for storage and water resistance, and sensitizers as required for complete detonation using the specified boosters. The blasting agent components and the resulting mixture shall fall under only the following classifications (CFR Title 46, Part 146.04-5): Explosive B, oxidizing material, hazardous article, inflammable liquid, and inflammable solid. Additives classified as explosives A by the CFR Title 46, Part 146.20-7 are not permitted except by the express permission of the Contracting Officer. The slurry shall be homogeneous throughout the charge and shall be capable of gelling to prevent leakage and to support the stemming material.

It shall be capable of complete detonation using the booster system specified in paragraph: "Boosters", after being (emplaced) (immersed in water) for a period of __ days at __ $^{\circ}F$. The blasting agent, its components, and its detonation products shall be sufficiently nontoxic and nonirritant so as not to present any undue hazard to personnel engaged in explosives handling and emplacement. Special safety equipment for personnel must not be required.

6._.1 Emplacement: The slurry may be delivered to the job site either in bulk or prepackaged. The slurry shall be capable of emplacement by pumping or tremie into the charge cavity through a minimum 4-inch diameter combination fill and vent line or open access hole. The discharge end of the delivery hose or tremie shall be kept below the rising pool level of the explosives at all times. The quantity of blasting agent shall be measured by count of the packaged materials, by weighing, or by metering at the option of the Contractor. The measurement unit shall be 2,000-pound tons. If the material is weighed or metered the measuring system shall be calibrated in the presence of a representative of the Contracting Officer. The tolerance of the measuring system shall be within 5 percent of the value indicated on the recording device.

6._.2 Performance Specifications of the Slurry Blasting Agent are as follows:

6._.2.1 Bulk Density: The density of the formulated mixture shall be a minimum of __ grams per cubic centimeter at one atmosphere. At least five independent samples shall be taken and the average, minimum and maximum density values reported. (The average density of the blasting agent shall also be reported for a pressure of __ atmospheres.) The Contractor shall make a minimum of one density measurement per emplacement hole to determine the density of the blasting agent being emplaced.

6. _ . 2. 2 Detonation Velocity: Shall be between _ and __ meters per second in a 6-inch diameter unconfined charge at $50^{\circ}F$. maximum. Measurement of detonation velocity shall be by Dautriche tests or time of arrival with measuring stations at least 0.5 meters apart. The first measuring station shall be at least 0.5 meters from the point of initiation. The charge may be detonated in air, underwater, or underground. At least five tests shall be run and the average, minimum and maximum values reported.

6. _ . 2. 3 Underwater Gas Bubble Energy: Shall be between _ and __ gram-calories per gram at $50^{\circ}F$. maximum. This shall be measured by underwater test as follows: An approximately spherical lightly confined charge of approximately 20 pounds shall be suspended in water at a depth of about 30 feet and detonated from the center using a booster as specified in paragraph: "Boosters". The body of water must be at least 150 feet across and 60 feet deep. If a test pond of smaller dimensions is used it shall be normalized by detonating an equal weight of TNT in the location planned for the test samples. The measured T for the samples shall be corrected by the ratio of measured and known T's for TNT. A suitable pressure transducer with no more than 0.5-millisecond rise time shall be placed 15 to 30 feet from the charge and connected to an oscilloscope, or similar recording device, with a sweep speed accuracy of one percent at a sweep speed of 50 milliseconds per centimeter. The bubble period, defined as the time interval between the shock arrival time and the arrival time of the peak of the first bubble pulse, shall be measured with an accuracy of at least one percent. Underwater gas bubble energy shall then be calculated by the formula:

$$E_b = \left[5.5T^3 (h + 2.31 P_o)^{2.5} - QB/2 \right] /W$$

where, E_b = underwater gas bubble energy, cal/gm

T = bubble period, seconds

h = depth of charge, feet of fresh water (if in sea water, multiply h by 1.03)

P_o = local atmospheric pressure, PSIA

Q = heat of detonation of the booster explosive, cal/gm

B = weight of booster, pounds

W = net charge weight, pounds (excluding primers and boosters)

At least five underwater tests shall be performed. The average, minimum and maximum underwater gas bubble energies shall be reported.

6._.2.4 Ratio of Aluminum to Available Oxygen: The number of gram-atoms of aluminum and available oxygen shall be computed and reported as a ratio. The following general formula shall be used for this purpose. Weight percentages are used and, if a particular component does not occur in a mixture, simply substitute zero percent in the formula. The formula is:

$$Al/O = 0.0371\ (\%\ Al)/B$$

where,

$$\begin{aligned} B = {} & 0.0270\ (\%\ RDX) + 0.0264\ (\%\ TNT) \\ & + 0.0375\ (\%\ AN) + 0.0270\ (\%\ HMX) \\ & + 0.0328\ (\%\ NM) + 0.0399\ (\%\ NG) \\ & + 0.0279\ (\%\ Tetryl) \\ & + 0.0380\ (\%\ PETN) \\ & + 0.0555\ (\%\ Water) \\ & + 0.0353\ (\%\ Sodium\ Nitrate) \\ & + 0.0297\ (\%\ Potassium\ Nitrate) \\ & + 0.0341\ (\%\ NH_4ClO_4) \\ & + 0.0326\ (\%\ NaClO_4) \end{aligned}$$

7. BOOSTERS: Shall be high strength, water resistant, non-nitroglycerin explosive, such as 50/50 pentolite, or the equivalent. Boosters shall have through-holes to accept plastic reinforced detonating cord. The bubble energy of the booster material shall be not less than 450 calories per gram at 50°F, using the detonating cord specified in paragraph: "Detonating Cord"

for initiation. The bubble energy may be measured by underwater test or an approved equivalent. A continuous column booster extending for a minimum of three-fourths the height of the explosive column shall be installed approximately in the center of each charge. The height of the explosive charge shall be determined by field measurements. The continuous booster column may be assembled by stacking individual units or as one continuous column primed its full length by detonating cord, but in any case the booster and any individual increment thereof must be capable of complete initiation of the specified blasting agent in the emplacement environment. The booster column must readily pass through a 4-inch pipe and be weighted or otherwise supported so as to remain in position during explosives emplacement operations. The booster shall be of sensitivity capable of being initiated with 50-grain reinforced or 54-grain plastic reinforced detonating cord. The Contractor shall submit a complete description of the booster system and the proposed method of installation to the Contracting Officer for approval.

8. ACCESSORIES:

8.1 Detonating Cord: PETN core Primacord as manufactured by the Ensign-Bickford Co., or equivalent. All detonator cord ends shall be sealed prior to emplacement to prevent water intrusion.

8.2 Blasting Caps:

8.2.1 Instantaneous electric, No. 6 or better.

8.2.2 Delayed electric, No. 6 or better. The time delay accuracy shall be within 10 percent.

9. EMPLACEMENT, PRIMING AND FIRING: The Contractor shall furnish and install all items of material and equipment necessary for emplacement, priming and firing. The Contractor shall submit detailed operations, and safety and security plans for emplacement, priming and firing to the Contracting

Officer for approval. The operations plan should include, but not be limited to, a detailed layout showing all runs of detonating cord by type and size, detonator type, locations and method of installation, and booster requirements. When delayed detonations are required a detailed drawing shall be included which incorporates the delay caps as an integral part of the booster system. The safety and security plan should describe in detail the methods the Contractor proposes to use to secure the exclusion area during emplacement, priming and firing. The outer boundary of the exclusion area shall be a minimum of ___ feet from the center of the proposed detonation. The approval of the operation and safety and security plans by the Contracting Officer shall not be construed as a complete check, but will indicate only that the general method of emplacement, priming and firing is satisfactory. Approval of such plans will not relieve the Contractor of the responsibility for the satisfactory detonation of all charges.

9.1 Safety and Security Officer: The Contractor shall designate a safety and security officer to supervise the Contractor's safety and security program and to coordinate activities with the Contracting Officer or his representative. The Contractor shall furnish to the Contracting Officer a copy of a letter of direction to the Contractor's Safety and Security Officer, outlining his duties and responsibilities, and signed by a responsible officer of the firm.

9.2 Explosives Engineer: During emplacement, priming and firing of the explosives the Contractor shall provide an explosives engineer of proven experience and ability in blasting operations, to supervise the Contractor's operations and to coordinate activities with the Contracting Officer or his representative. A minimum of 30 calendar days prior to commencement of emplacement operations the Contractor shall furnish the Contracting Officer with the bona fides of the explosives engineer to include but not be limited to past experience, training, and education. The acceptability of the explosive engineer is subject to the approval of the Contracting Officer.

9.3 Detonation Schedule: In addition to the progress schedule submitted in accordance with the general provisions of this contract, the Contractor shall verify in writing the proposed detonation dates to the Contracting Officer within 15 calendar days of the planned detonation(s). The Contractor shall verbally confirm readiness for and request approval for the Contracting Officer to proceed with a planned detonation 24 hours before each event. Actual detonation may be accomplished only when the Contracting Officer has determined that the safe firing criteria, as outlined in the safety and security plan, have been met. If a detonation is delayed by the Contracting Officer for reasons outside the purview of the Contractor's responsibility, after the proposed detonation is ready and the safe firing criteria have been met, he will be paid for delays in excess of 4 hours.

9.4 Firing: The Contractor shall provide a control point capable of housing his firing equipment and crew and the Contracting Officer or his representative during shot time. The control point shall be at a convenient location outside of the exclusion area which provides a clear line of sight between the control point and ground zero. The Contractor shall submit to the Contracting Officer, for approval, drawings showing the location, access thereto and type of construction of the proposed control point. Before the first detonation a minimum of one systems dry run shall be conducted. The dry run shall consist of duplicating the detonating cord and electric cap layout proposed for the detonation and successfully firing it. Before each succeeding detonation a system check of the firing unit shall be made using the same number of electric caps as proposed for use on the planned detonation. The actual firing shall be accomplished only on specific approval and in the presence of the Contracting Officer or his representative. The firing of charges shall be accomplished with a Contractor-furnished capacitor discharge unit (CDU) of sufficient size to detonate the maximum number of electric caps required within 0.1 seconds of zero time switch closure. The unit shall have a key-operated master switch that makes the unit inoperable and shorts the output terminals when the key is removed. The CDU may be installed near ground zero if it is remotely controlled from the control point.

10. TRANSPORTING AND STORAGE: In addition to full compliance with Section XXV, Blasting, of General Safety Requirements, EM 385-1-1, the Contractor shall follow the recommendations of NFPA Code No. 495 and IME Publication No. 6 when transporting and handling blasting agents. The Contractor shall be responsible for making all arrangements and shall comply with all applicable Federal, State and local laws and regulations for the transporting and handling of the explosives. The Contractor shall submit to the Contracting Officer, for approval, drawings showing the location, access thereto, and type of construction of the proposed storage magazines for blasting agents, explosives, cap house, detonating cord and "make up shack." The explosive storage magazines and other facilities may be located on Government lands if satisfactory locations can be found and are approved by the Contracting Officer; or the Contractor, at his option, through private negotiations, may locate explosive magazines outside Government lands. The Contractor shall provide and maintain access to the explosive storage areas at his own expense.

11. TESTS: All required test results shall be furnished prior to explosive delivery to the test site except for those tests required to be performed on-site. Samples of explosives and other related materials may be required to be taken by the Contractor at the site and delivered to the Contracting Officer or his representative at the site.

12. LOG: A record of all explosives emplaced shall be made and furnished the Contracting Officer before each detonation. This shall include the length and position of boosters, quantity of explosives emplaced, elevation of top and bottom of explosives, and results of any required density measurements.

13. WAYBILLS shall be submitted to the Contracting Officer during the progress of the work. Before the final payment is allowed the Contractor shall file with the Contracting Officer certified waybills for all explosives.

14. MEASUREMENT AND PAYMENT:

14.1 Mobilizing and Demobilizing shall consist of bringing to the site all equipment, materials, and supplies necessary for the work, and for erecting and/or installing equipment. Mobilization and Demobilization will be paid for at the contract price for such work. The Contractor will be paid 60 percent of the contract price after mobilization and 40 percent of the contract price after demobilization.

14.2 Charge Emplacement Construction: Construction for charge emplacement will be measured for payment by the nominal charge size, as shown on the contract drawings, including stemming of the charge. Payment will be at the contract unit price for the item in the Bid Schedule, which payment shall constitute full compensation for construction and stemming of the charge.

14.3 Blasting Agent(s): As herein before specified the quantity of blasting agent(s) shall be measured by count of the packaged materials, by weighing, or by metering, at the option of the Contractor. The measurement unit shall be 2,000-pound tons. Payment will be at the respective contract unit prices per ton for the item in the Bid Schedule, which payment shall constitute full compensation for furnishing, placing, priming, firing, and for incidentals necessary to complete the work required by this section of the specifications.

14.4 Standby time will be measured for payment on the basis of the number of hours of delay, as defined in paragraph "Detonation Schedule", in actual work time after the first 4 hours of such delay. Payment will be made at the contract price per hour for the item in the Bid Schedule, but will be limited to 8-hours in any one work day. Such payment shall constitute full compensation to the Contractor for all costs in connection with the delay.